G3SoilWorks
GEOLOGY · GEOTECH · GROUNDWATER

Preliminary Engineering Geologic / Geotechnical Evaluation

Proposed AC Marriott Residence Inn Dual Brand
3466 Mission Inn Avenue
Riverside, California 92501

Prepared for:

Mr. Angel Orozco

Greens Group, Inc.
9289 Research Drive
Irvine, California 92618

Prepared by:

G3SoilWorks, Inc.

Project No. 1-1167-A
March 31, 2020

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Attachments:

Mr. Angel Orozco
Greens Group, Inc.
9289 Research Drive
Irvine, California 92618

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Subject: **PRELIMINARY ENGINEERING GEOLOGIC / GEOTECHNICAL EVALUATION**
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Riverside, California 92501

References: See attached List of Selected References

Dear Mr. Orozco,

In accordance with your request and authorization, G3SoilWorks, Inc. (G3), has prepared this Preliminary Engineering Geologic / Geotechnical Investigation Report for the proposed AC Marriott Residence Inn located at 3466 Mission Inn Avenue in Riverside, California. This report presents our findings developed through desktop evaluation, review of historic information including vintage topographic maps, aerial photos, geologic maps, subsurface exploration, geomorphic analyses, and our local experience with other nearby projects of a similar nature. Also presented are our geotechnical recommendations for site development as it pertains to the currently proposed design concepts presented to us by Greens Development and architectural plan sets by AXIS / GFA Architecture + Design.

PURPOSE / INTENT

The purpose of our work was to evaluate the engineering geologic / geotechnical aspects of the site and provide recommendations for construction of the proposed development. This report is intended to provide an engineering geologic / geotechnical synopsis of the current site conditions and recommendations for development as they relate to the proposed construction.

SCOPE OF WORK

The following scope of work was performed to evaluate the engineering geologic / subsurface conditions present at the site, develop appropriate geotechnical recommendations for the proposed development / construction, and prepare this report summarizing our findings and recommendations:

- Discussion / correspondence with the project architect (AXIS / GFA Architecture + Design);

- Review of preliminary architectural design plans of the proposed hotel building (Reference 1);
- Review of recent topographic survey maps of the existing property;
- Review and analysis of historic information including vintage topographic maps, aerial photos, and published geologic maps / reports;
- Formulation of a preliminary subsurface investigation program, development of boring layout and cordoning off existing permit parking areas in advance, and coordination with Underground Services Alert (USA DigAlert);
- Field subsurface exploration included the advancement of three (3) 8-inch diameter hollow stem auger borings (B-1 through B-3, respectively) and related sampling to depths ranging from 51.5 to 76.5 feet below existing ground surface;
- Laboratory testing of soil samples obtained during subsurface exploration;
- Review of preliminary design architectural plans, published and proprietary geologic / geotechnical information, geologic mapping / reports, etc., to develop design and construction recommendations;
- Geologic hazards assessment including seismic hazards and similar;
- Development of geotechnical recommendations for site development including geotechnical criteria for shoring, foundations, and tiebacks in support of the planned underground excavation work and planned parking structure; and
- Preparation of this preliminary engineering geologic / geotechnical report.

SITE DESCRIPTION

Site Vicinity / General Layout / Nearby Structures

The site is located in the Downtown area of Riverside, California, at the south corner of Lemon Street and Mission Inn Avenue (Figures 1 and 2, attached). Currently, the site consists of an open permit-parking area and fenced-in paved lot approximately 125 feet wide by 200 feet long. Downtown Riverside is characterized by densely urbanized city-block development with several nearby historic buildings, multi-story high rises, and paved streets / avenues. Mission Inn Avenue and Lemon Street bound the site to the northeast and northwest, respectively. A historic building understood to be the former Downtown Fire Station 1 bounds the site to the southeast and a through-going alleyway between Lemon Street and Lime Street bounds the site to the southwest and separates the historic fire station building from the modern Downtown Fire Station 1 and adjacent Life Arts Center. The Riverside Municipal Auditorium & Soldier’s Memorial Building and Riverside Art Museum are located northeast of the site across Mission Inn Avenue.

PROJECT DESCRIPTION / PROPOSED DEVELOPMENT

According to the provided architectural plan set (Reference 1), the project consists of an 8-story hotel with four (4) sub-levels for underground parking. Current site elevation (El.) is 861± feet above sea level; proposed building heights range from approximately 40 feet below existing ground surface (lowest basement level – El. 821± feet), 84.92 feet above ground surface (roof level – El. 946± feet), and 94.33 feet above ground surface (penthouse level – El. 956± feet). Construction of the underground parking structure and hotel building will require excavations on the order 40 feet below existing ground surface. Shoring will be required to support excavation sidewalls and adjacent properties.

GEOLOGY

The following sections provide a general overview of the regional geology, tectonics, and associated seismic hazards in southern California. Site-specific information regarding the engineering geologic conditions considered in this evaluation is presented in the Engineering Geology section of this report.

Regional Geologic Setting / Tectonics

Southern California is a region of active tectonics, faulting, and seismicity (earthquakes) associated with right lateral movements between the Pacific and North American tectonic crustal plates of the earth. These right lateral movements, with the Pacific plate moving northwestward relative to the North American plate, have developed over the past ~20 million years of geologic time along a major plate boundary referred to as the San Andreas transform fault. Currently, and throughout recent geologic time, right lateral motion along the Pacific-North American plate boundary occurs at a rate of approximately 2± inches/year (50 mm/yr) of which the San Andreas Fault Zone (SAFZ) accommodates nearly one-half of this relative motion with an average slip rate of 0.9± inches/year (22 mm/yr). The remaining plate motion is taken up by a complex system of faults that compose the southern San Andreas fault system illustrated in Figure 3. Active faults of the southern San Andreas system accommodate relative plate motion through “stick-slip” behavior that can manifest as infrequent, large earthquakes producing anywhere from inches to several feet, or tens of feet of slip in a single earthquake; cumulative displacements averaged over geologic time, resulting from multiple earthquakes (slip-events), equate to the long-term average estimated slip rates for individual fault lines depicted on Figure 3. One of these fault lines, the San Jacinto Fault Zone (SJFZ), is located approximately 7 miles northeast of the site.

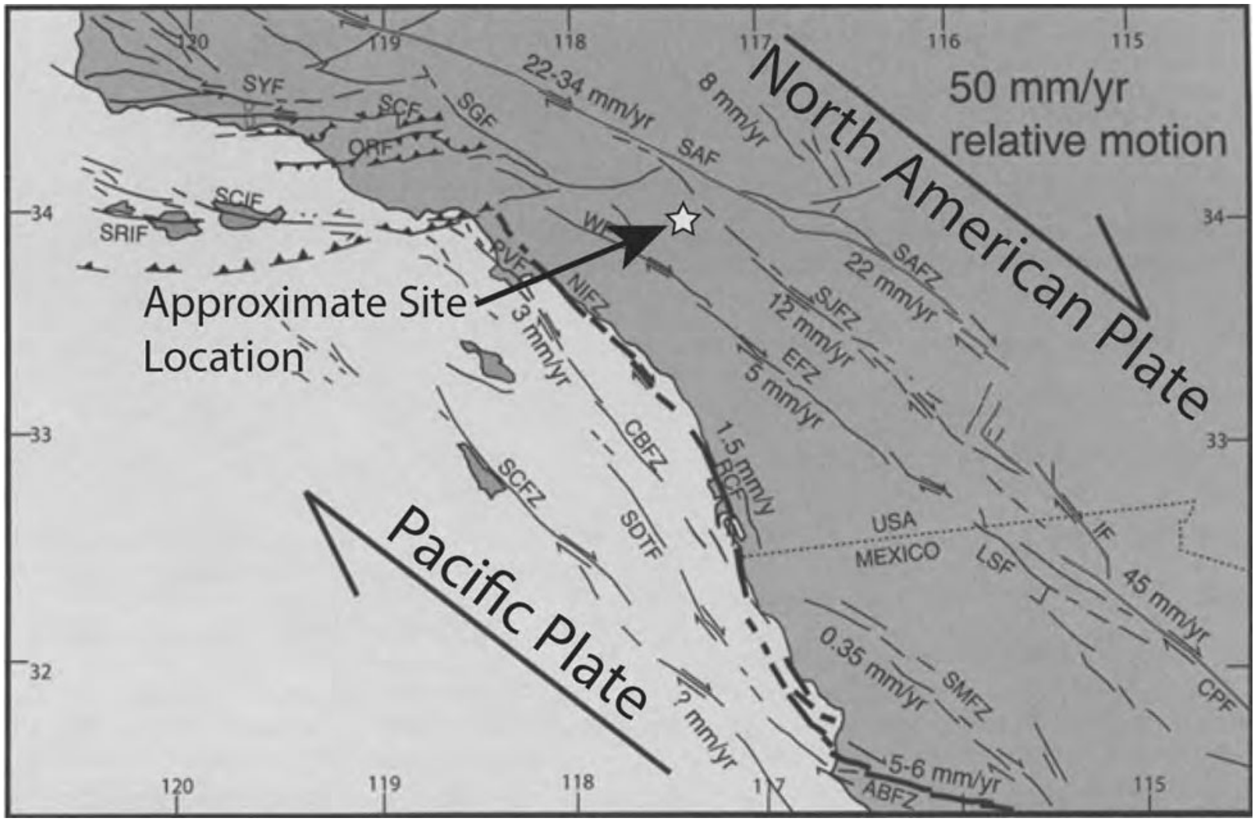

Figure 3. Illustrative Regional Tectonic Map (modified from Grant and Rockwell, 2002 – Reference 2) depicting relative plate motion between the Pacific and North American tectonic plates and average slip rates along selected fault lines (shown in mm/year; 1 inch = 25.4 mm). As described above, the San Andreas Fault Zone (SAFZ; red line) accounts for nearly half of this relative motion with the remainder distributed along a complex system of faults that include the San Jacinto Fault Zone (SJFZ) located approximately 7 miles northeast of the project site.

As illustrated below, long-term displacements along the various fault lines of the southern San Andreas system has deformed the southern California landscape and broken it into individual crustal blocks that, in part, compose the major geomorphic provinces of California. The project site is located within the Peninsular Ranges geomorphic province of southern California, and is characterized by northwest-trending, fault-bounded mountain blocks that include the Santa Ana Mountains, Perris Peneplain, and San Jacinto / Santa Rosa Mountains. Along the boundaries of these mountain blocks, right-lateral faulting and associated seismicity mirroring that of the San Andreas occurs along the northwest-trending Elsinore (EFZ) and San Jacinto (SJFZ) fault zones. The project site is located in northern Riverside County near the SJFZ, just south of the "Big Bend" of the SAFZ.

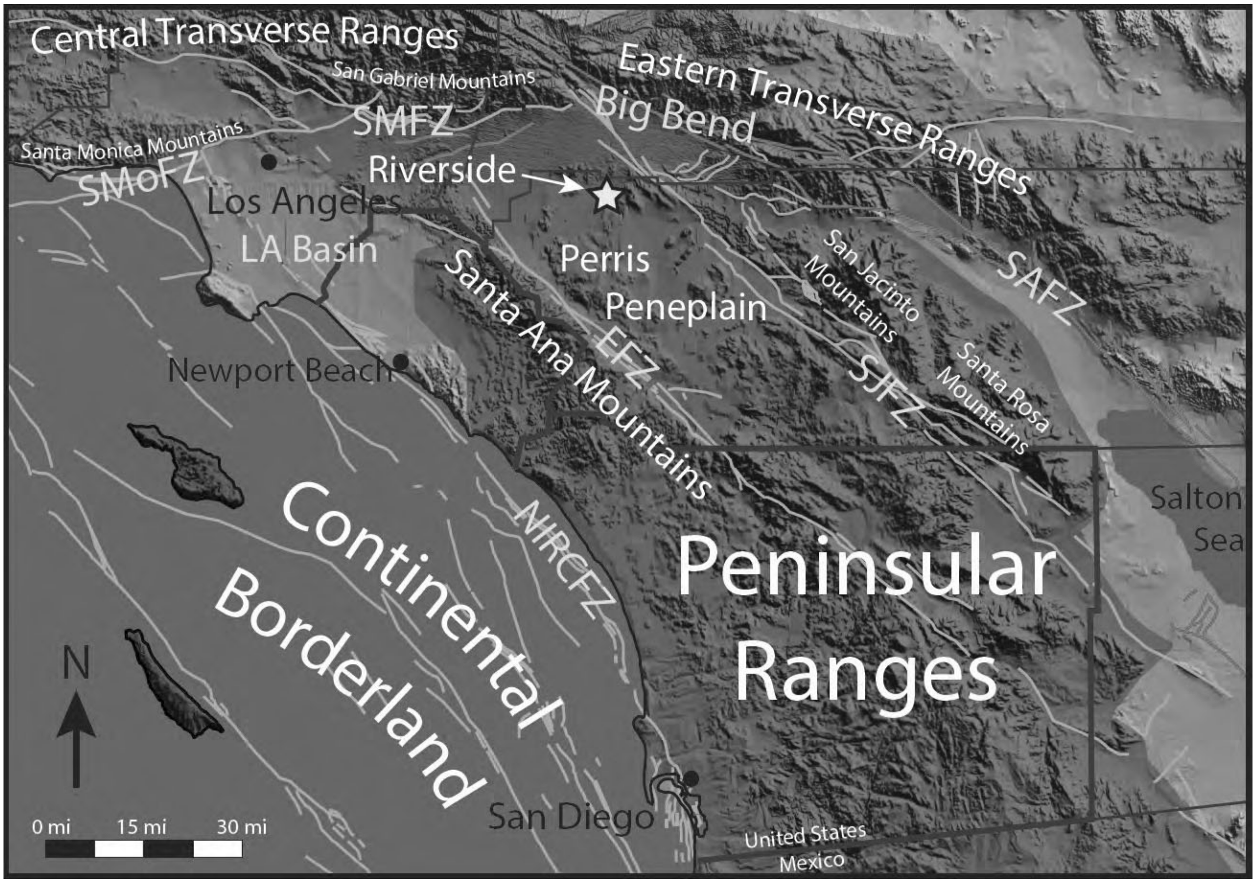

Figure 4. Map of Local Geomorphic Provinces, Regional Fault Lines, and Related Land Forms including: the Peninsular Ranges (orange), Continental Borderland (blue), Transverse Ranges (green) and LA Basin sub-province; the San Andreas (SAFZ; red line), Newport-Inglewood / Rose Canyon (NIRCFZ; orange line), San Jacinto (SJFZ), Elsinore (EFZ), Santa Monica (SMoFZ), and Sierra Madre (SMFZ) fault zones (orange lines). The project site (yellow star) is located in the northern Peninsular Ranges geomorphic province. County lines are shown in blue with the project site located in northwestern Riverside County.

Local Geologic Setting / Geomorphology

The City of Riverside and site area are located in the Santa Ana River Valley, 1.7 miles easterly of the Santa Ana River itself (El. 750± feet). Topographically, the City of Riverside and site area (El. 861± feet) occupy a broad, topographic "low" / alluvial fan surface between the Box Springs Mountains (3.5 miles east of the site) and Mount Rubidoux (1.4 miles west of the site; see Figure 5, attached). Here, the geomorphology is defined by a broad, gently sloping alluvial fan / valley fill complex derived from the crystalline / igneous plutonic rocks of the Box Springs Mountains and upland areas east of Riverside. Geologic mapping by Dibblee and Minch (References 3 and 4; see Figure 6) indicates that the site area is underlain by older surficial sediments of weakly indurated sand and minor gravel (Quaternary old alluvium, Qoa), representing Pleistocene-age alluvial fan deposits derived from the local mountains and foothills. Plutonic rocks of the Peninsular Ranges (crystalline granitoid rocks including quartz diorite / tonalite, Qdi / Qdx) compose the local mountains / foothills and represent the source terrain for the alluvial sediments underlying the site. Over the past 10- to 20-thousand years or more of geologic time, weathered mineral soils / sediments eroded from the local bedrock terranes have accumulated in the "low" formed between Mount Rubidoux and the distal fan surfaces emanating from the Box Springs Mountains to the east. The infilling of this "low" has created a topographically elevated fan surface that sits 100+ feet above the current Santa Ana River drainage, with granitoid rock of Mount Rubidoux bounding this fan surface to the west. As a result, soils underlying the project site are comprised of thinly bedded horizons and anastomosing lenses of ancient alluvium (likely Pleistocene-age) derived from the local mountains and associated granitoid rocks.

ENGINEERING GEOLOGY

Subsurface Exploration and Laboratory Testing

In order to evaluate the site's engineering geologic / subsurface conditions and develop geotechnical recommendations for design, a total of three (3) exploratory borings were advanced using an 8-inch diameter, hollow-stem auger drill rig (see Figure 7 – Boring Location Map). Soil borings B-1, B-2, and B-3 were advanced to depths of 51.5, 76.5, and 51.5 feet, respectively. Driven samples were collected using alternating SPT / Cal-Modified sampling methods at 5-foot intervals for B-1 and B-3; samples for B-2 were taken at 2.5-foot intervals from 0-50 feet and 5-foot intervals from 50-75 feet, respectively. Soil samples were logged by our field geologist, packaged, and transported to our laboratory for testing. Logs of geotechnical soil borings are included in Appendix A.

Representative soil samples were tested for geotechnical properties including moisture / density, maximum density / optimum moisture, consolidation / settlement potential, shear strength, etc. Laboratory test results are included in Appendix B.

Site Geology / Subsurface Conditions

The site itself is paved and surrounding vicinity heavily urbanized. However, based on the results of our subsurface investigation and testing, the site is underlain by locally derived artificial fills (afu) and Quaternary old alluvium (Qoa) comprised of thinly bedded horizons / anastomosing lenses of silty sands and sandy silts with localized horizons / lenses of poorly-graded sand, sandy clay, and silty clay. Native alluvial soils contain a large percentage of low to very low plasticity / non-plastic fines and micaceous fine sand. SPT and Cal-Modified blow counts indicate that alluvial soils are in a relatively dense configuration (medium-dense to very dense / stiff to very stiff and locally hard / indurated). No groundwater was encountered to maximum depths drilled to 76.5 feet.

Geologic Units and Occurrence

<u>Artificial Fill Undocumented – (afu)</u>

- Depths of Occurrence: likely no more than **3-5 feet** below ground surface;
- Soil Types: **Sandy Silt with Clay** and localized zone of **Silty Clay** (B-3 only);
- Color: **dark yellowish brown** (10YR 4/4) and **very dark gray** (10YR 3/1), respectively;

- Moisture: **moist to very moist**;
- Density / Consistency: **medium dense** and approximately **firm to stiff**, respectively;
- Plasticity: **low** and **low to medium**, respectively (apparent, based on field classification);
- Expansion Potential: probably **low** to locally **medium** (based on field classification and laboratory testing); and
- Other features: very dark gray silty clay in B-3 contained brick fragments.

Quaternary Old Alluvium – (Qoa)

Quaternary old alluvium was encountered to maximum depths explored (51.5-76.5 feet) and primarily consisted of highly micaceous silty sand / sandy silt mixtures with variable clay content and low to very low plasticity. Lenses / horizons of predominately fine- to medium-grained, poorly-graded sand composed primarily of sub-rounded to sub-angular mineral grains (quartz and feldspar derived from local granitoid rocks) was encountered at approximate depths of 35-45 feet, 35-45 feet, 25-35 feet in borings B-1, B-2, and B-3, respectively. At depths of 45-55 feet, interbedded horizons / lenses of clayey sand and sandy clay were encountered in boring B-2. Hard / highly indurated silty clay and sandy clay soils were encountered at depths of 55 feet; horizons / lenses of very stiff, silty clay with low to medium apparent plasticity were encountered at 66.5 and 70 feet. General soil types / descriptions are summarized below:

- Soil Types: **Silty Sand** / **Sandy Silt**, **Poorly-Graded Sand**, and **Silty Clay** / **Sandy Clay**;
- Color: **brown** to **dark yellowish brown**, **olive brown**, and **strong brown**;
- Moisture: **moist**, **slightly moist**, and **very moist**, respectively;
- Density / Consistency: **medium-dense** to **very dense / stiff to hard**;
- Expansion Potential: generally **low** to **very low** and **low** to **medium** where clayey (based on field classification and laboratory test results)
- Other Features: highly micaceous fine sand, low to very low plasticity fines, and non-plastic silt abundant throughout.

Hydrogeology / Groundwater

Hydrogeologic Setting

Downtown Riverside and the project site are within the bounds of the Upper Santa Ana Valley Groundwater Basin, Riverside-Arlington Sub-Basin. According to the California Department of Water Resources (DWR) Groundwater Bulletin 118 (DWR, last updated 2/27/04), the Riverside-Arlington Sub-Basin is understood to be bound by the Box Springs Mountains to the southeast, Arlington Mountain to the south, La Sierra Heights and Mount Rubidoux to the northwest, and the Jurupa Mountains to the north; the Rialto-Colton fault (a splay of the San Jacinto Fault Zone) bounds the sub-basin to the northeast. In general, groundwater associated with this sub-basin is primarily occurs in association with the Quaternary age alluvial deposits (sand, gravel, silt, and clay) of the Santa Ana River and its tributaries. Near the City of Riverside, the upper 50 feet of deposits are reported to be "principally clay" and, therefore, are poor conductors of groundwater. This reported occurrence of clayey / low permeability soils in the upper 50 feet of deposits near Riverside is generally consistent with the findings of our subsurface investigation, except for the presence of interbedded silty sands / sandy silts with variable clay content and localized lenses of relatively "clean" sand that occur throughout the overall section. Sand bodies encountered at depth are anticipated to be of limited aerial extent and generally confined by fine-grained / low permeability capping soils that comprise the upper 25-35 feet of the subsurface.

Subsurface Exploration

Groundwater was not encountered in our exploratory borings to a maximum depth of 76.5 feet in boring B-2. The lack of observed groundwater is attributed to the site's elevation 100± feet above the nearest regional water body (i.e., the Santa Ana River) and presence of fine-grained / low permeability soils which form confining layers (aquicludes / aquitards) near the surface and at depth. As described above, alluvial soils underlying the site are dominated by the presence of thinly bedded / discontinuous horizons of predominately fine-grained sandy silts / silty sands with variable clay content – low permeability soils which are poor conductors of groundwater. Lenses of predominately poorly-graded sand were encountered at depths ranging from 25-45 feet. Low soil moistures, presence of generally uniform / moderately oxidized soil colors (7.5-10YR), and lack of mottling / gleying (soil discoloration) also support the general absence of groundwater at the depths explored.

Review of DWR Water Data Library and Geotracker

A review of DWR's Water Data Library (Table A) for two (2) nearby wells (Fox Metro and Clearwater) indicate that groundwater levels measured near the site from 2011-2019 range from Elevation (El.) 763-770 for Fox Metro well (1,400-1,500± northwest of the site) to El. 766-776 for the Clearwater well (5,800± feet southeast of the site). Additional well data available on Geotracker was also reviewed for 3304 14th Street (CHEVRON #96984, LUST Cleanup Site, T0606599140, El. 860-864± feet, 2,800± feet south-southwest of site) and 3315 14th Street (MOBIL #18-D3H, LUST Cleanup Site, T0606500480, El. 861± feet, 2,600± feet south-southwest of site) indicate groundwater depths 86-99± feet below ground surface (El. 763-774± feet). Based on the current site elevation (861± feet), bottom elevation for the proposed underground parking structure (821± feet), and above described well data, a vertical separation of 47-51± feet is apparent between shallowest recorded groundwater and base of proposed parking structure.

Table A. Local Well / Groundwater Data form DWR Water Data Library (http://wdl.water.ca.gov/waterdatalibrary/)

State Well Number:	02S05W23F001S	02S05W25F001S
Local Well ID:	Fox metro	Clearwater
Site Code:	339840N1173750W001	339690N1173590W001
Latitude (NAD83):	33.984000	33.969000
Longitude (NAD83):	-117.375	-117.359
Groundwater Basin (code):	Riverside-Arlington (8-002.03)	Riverside-Arlington (8-002.03)
Well Use:	Observation	Unknown
Well Status:	Inactive	Inactive
Ground Surface Elevation (NAVD88 feet):	843.000	916.560
Total Depth of Well (feet)	N/A	388
Perforated Interval Depths (feet)	N/A	144-218
Distance from Project Site	1400-1500± feet NW	5,800± feet SE
Range of Groundwater Depths (feet)	73.53-79.38*	140.78-149.76**
Range of Groundwater Elevations (NAVD88 feet)	763.62-769.47*	766.80-775.78**

Note: (*) Range of depths / elevations recorded from 10/31/2011 through 3/8/2016;
(**) Range of depths / elevations recorded from 10/31/2011 through 10/15/2019.

Groundwater / Subsurface Water Considerations

Given the above, the proposed excavation is not anticipated to intercept groundwater. However, it should be noted that transient water, in the form of localized seepage and/or surface water intrusion along the excavation sidewalls, may develop as excavations proceed and the area is subjected to wet weather conditions. Such occurrences should be accounted for in the planning, design, and construction of temporary shoring systems and the proposed underground parking structure. It has been our experience that such structures, even in the absence of groundwater, can be susceptible to surface / subsurface water phenomena that can adversely affect the performance of underground / basement waterproofing, drainage, and similar. For this reason, we provide general recommendations for drainage, waterproofing, and similar in the Recommendations section of this report.

SEISMIC HAZARDS

Seismicity and Local Faulting

The project site is in an area of high seismicity associated with the nearby Elsinore, San Jacinto, and San Andreas fault zones (EFZ, SJFZ, and SAFZ, respectively), representing major / regionally active faults of the southern San Andreas fault system. According to the California Geological Survey Earthquake Zones of Required Investigation website application (EQ Zapp:

https://maps.conservation.ca.gov/cgs/EQZApp/app/), the nearest zoned active faults include the EFZ (15.5± miles southwest of the site), SJFZ (7 miles northeast of the site), and SAFZ (14 miles northeast of the site). According the Southern California Earthquake Data Center (SCEDC; https://scedc.caltech.edu/significant/fault-index.html#e), major surface ruptures along the EFZ and SJFZ occur approximately every 250 and 100-300 years on average, respectively, with probable earthquake magnitudes (Mw) estimated at 6.5-7.5. The SAFZ has a complex history of earthquakes / surface rupture events, with the most recent large earthquake being the 1857 Fort Tejon earthquake (Mw 7.8-7.9) that occurred along its central Mojave segment northwest of Cajon Pass. Prior to that, the last major rupture on the southern SAFZ (extending from Cajon Pass southward to the Salton Sea) is estimated to have occurred about 300 years ago between 1719 and 1733 (Reference 5). Probable magnitudes for the central and southern SAFZ are estimated at Mw 6.8-8.0, occurring every 140-180 years on average; the open interval for the southern SAFZ is estimated at over 300 years (nearly twice its average estimated recurrence interval), suggesting that it is overdue for a large earthquake / surface rupturing event.

Surface Rupture and Strong Ground Motion

Based on our review of published geologic / regulatory maps for the area including the Riverside County Fault Zones Mapping Portal, and our understanding of the local geology / tectonics of the area, the site is not considered to be at risk of surface rupture from a known active fault. The nearest active fault, zoned by the State of California / California Geological Survey, is the SJFZ (San Bernardino Section) located 7 miles northeast of the site. The site will undoubtedly experience strong ground motion as a result of regional seismicity associated with EFZ, SJFZ, SAFZ, and other active faults of the southern San Andreas fault system. Presented in Appendix E are the 2016 CBC (ASCE 7-16) Seismic Design Parameters for the subject site. Note that Risk Category III has been assigned based on occupancy and structure type (high occupancy, multi-story structure); Site Class D has been assigned based on the findings of our subsurface investigation, demonstrating high SPT / Cal-Modified blow counts (ranging between 15-50 blows/foot and greater). Also note that 1-second period ground motion parameters are omitted (*null) and should be evaluated accordingly by the project structural engineer per ASCE 7-16, Section 11.4.8.

Liquefaction

Liquefaction is phenomenon involving a loss of shear strength / bearing capacity of saturated, cohesionless sediments subjected to strong ground motion. Most liquefaction hazards are associated with poorly consolidated / low density, saturated or nearly saturated, non-cohesive, sandy and silty soils. The California Geological Survey has not published an Earthquake Zones of Required Investigation for the subject site, but the Riverside County Mapping Portal shows the site to be in Zone 104 – an area of low susceptibility to liquefaction. This is consistent with the findings of our subsurface investigation, demonstrating a lack of groundwater and presence of relatively dense soils with relatively high SPT / Cal-Modified blow counts (commonly exceeding 30 blows/foot), and cohesive / fine-grained soils. The relative age of on-site soils (pre-Holocene/Pleistocene) is also considered a favorable condition.

In view of the above, the potential for site liquefaction is considered very low to nil.

Earthquake-Induced Landslides

The California Geological Survey has not published an Earthquake-Induced Zones of Required Investigation for the City of Riverside, nor does the County of Riverside offer similar maps for the area. No slopes other than temporary construction cuts are proposed for the project and the site / vicinity is generally flat-lying / gently sloping. Based on the geomorphology of the area, landslide features are not anticipated. However, the proposed excavation will include 40± feet high vertical grade cuts, requiring the design / installation of temporary shoring and permanent basement-level retaining walls. Geotechnical criteria for shoring and basement retaining walls are included in the Recommendations section of this report to provide appropriate factors of safety against earthquake-induced instability.

The potential for large-scale earthquake-induced landsliding at the subject site is considered very low to nil provided that the recommendations included in this report are appropriately implemented to the satisfaction of this office.

Tsunami

The site is approximately 861 feet above sea level, 78 miles inland of the Pacific Ocean, and separated from the coast by the Santa Ana Mountains and coastal plain of Orange County. Risk of tsunami impacting the site from known tectonic / submarine landslide sources in the Pacific Ocean is considered nil.

Seiche

Seiche is defined as a standing wave oscillation effect generated in a closed or semi-closed body of water caused by wind, tidal current, and earthquake, (i.e., “sloshing”). Seiche potential is highest in large, deep, steep-sided reservoirs or water bodies. Based on the planned structure elevations and structure daylight elevations on the slope, the potential for seiche effects to the planned development is considered low to nil. However, seiche effects for planned pools should be accounted for in design, as sloshing of pool water may be expected during a large seismic event. Potential seiche effects should be mitigated to prevent uncontrolled runoff into to the future planned hotel appurtenances.

RECOMMENDATIONS

General

Based upon our field exploration, visual observations, laboratory testing, and review of the information provided, the following recommendations are provided. From a geotechnical standpoint, it is our opinion that the proposed construction is feasible, provided that the recommendations provided herein and the City / County of Riverside grading requirements are incorporated in the design and implemented during construction.

Geologic and Geotechnical Engineering Considerations

Based on our findings, it is our opinion that the construction of the proposed development is feasible from a geologic / geotechnical standpoint, provided that recognition of, consideration for, and accommodation of the conditions present at this site are accounted for in design and construction:

- The presence of existing fill materials to depths on the order of 5 feet below existing site grades, based on our observations.
- Soil disturbance resulting from the demolition and removal of existing improvements, utilities and landscape areas.
- The proposed development will include four (4) subterranean parking levels, and temporary shoring system will be necessary to accommodate excavations to depths on the order of 40 feet below existing site grade;
- Excavations for the proposed development will be proximal to property lines and bordering Mission Inn Avenue and Lemon Street.
- Designated historical buildings are located adjacent to the west and southern limits of the proposed development and may be impacted by noise, dust, and vibration - as well as ground effects translated to them by the subject work (i.e., shoring).
- Possibility of localized / transient seepage / groundwater and related moisture / vapor hazards are present for slabs and walls in contact with site soils.
- Noise and vibration during construction efforts.
- Settlement due to foundation and improvement loading.
- Earthquakes have occurred in Southern California and will, undoubtedly occur in the future, and high round accelerations during seismic shaking may be experienced at the site. Therefore, the proposed structure should be designed and constructed to the prevailing standards and seismic design requirements, as appropriate.

With regards to the above and other related considerations, the reader is referred to the "good words" section of this report for more information and critical insight on the various aspects of work described herein.

<u>Pre-Construction Monitoring</u>

The planned excavation for the subterranean parking levels are anticipated to extend to depths on the order of 40 feet below existing site grades. Pre-construction baseline monitoring, as recommended below, should be performed prior to site grading and excavation operations.

a) Prior to site excavation, adequate and proper instrumentation (e.g., extensiometer, survey points, etc.) should be considered for adjacent structures, improvements, or surface grades to document and monitor conditions. Photo-documentation of the condition of existing surrounding improvements prior to excavation is also recommended. The documentation should be periodically conducted during and following completion of construction.

b) Ground-transmitted vibrations and high sound levels should be anticipated, particularly during drilled soldier pile installation and basement level excavations. Site vibration monitoring should be performed in accordance with the U.S. Bureau of Mining Standard RI8507 or other methods suitable for the intent.

c) A preliminary study, in advance of demolition and construction should be performed to assess the sensitivities and characters of the surrounding developments / structures, evaluate proposed contractor methods and equipment / approach to develop vibration and noise thresholds, and assign appropriate risk management protocols and criteria to mitigate. Pre-construction sound and vibration levels should be monitored and documented during times and conditions that will be similar to those when construction work will take place to establish a baseline.

d) A construction monitoring and mitigation plan should be developed and appropriately implemented to document existing conditions, establish baselines for noise and vibration, pre-construction mitigative measures in construction monitoring and mitigation, and long-term considerations.

Site Grading

Presented below are general grading recommendations relative to site clearing, preparation, excavation, etc. which should be included as part of the grading operations, as appropriate.

<u>Site Clearing and Preparation</u>

a) Site preparation and grading should be made under the observation of the Project Geotechnical Engineer or Geologist, Project Engineering Geologist, and / or this firm's field representative, under their supervision.

b) Proper measures should be implemented during the performance of remedial / precise grading work to protect the work site, particularly excavated areas, from flooding, ponding and inundation due to poor or improper temporary surface drainage. During periods of impending inclement weather, temporary provisions should be made to adequately direct surface drainage, from all sources, away from and off the work site and to provide adequate pumps and sumps to handle any flow into the excavations.

c) Prior to the start of the required earthwork and grading, all vegetation, surface trash, debris and other deleterious materials should be removed from areas of planned grading and wasted away from the site. Vegetation removal should include root-balls and attendant root systems.

d) Utility laterals / stub outs should be located prior to grading and flagged.

e) Removal of existing structures / improvements should include foundations, concrete flatwork, and any remaining buried obstructions. Concrete fragments and construction debris from site demolition operations should be disposed of off-site.

f) Any pipelines or conduits encountered within the zone of planned development that are designated for abandonment should be removed from the construction area and ends cut and plugged according to the applicable Code requirements and/or evaluated by this office on a case by case basis, but not less than 10 feet outside the perimeter of the proposed construction area, or as property line considerations dictate. Non-reinforced concrete or clay pipes may be crushed in-place and incorporated in the fill.

g) Alternately, deep hollow lines may be left in place, provided they are filled with concrete, slurry or similar. No filled line should be permitted closer than 2 feet from the bottom of footings.

h) Local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

Remedial Grading

- Shallow existing fill soils to depths on the order of 2 to 5 feet were encountered at the exploration boring locations drilled during the field exploration phase. During site excavations for subterranean grading, it is anticipated that these fill soils will be completely removed from the limits of proposed development. However, should proposed development/ ancillary construction extend beyond the limits of subterranean level construction, remaining fill soils should be removed full depth within the limits of planned development, laterally to a distance of 5 feet beyond structure limits (where feasible), and replaced with engineered compacted fill.
- Excavation at the lowermost subterranean level is anticipated to expose competent native soils. To provide more uniform and acceptable slab on grade support, it is recommended the exposed subgrade across the basement be recompacted to a one-foot depth to at least 90 percent relative compaction.
- Any surface or subsurface obstructions, or questionable materials encountered during grading should be brought immediately to the attention of the Project Geotechnical Engineer.

After approval of the over-excavations discussed above, and prior to placement of any compacted engineered fill materials, the exposed bottoms should be scarified to a depth of approximately six to eight inches, moisture conditioned to approximately 1 to 3 percentage points above the optimum moisture contents and compacted to at least 90 percent relative compaction (ASTM: D1557) with suitable compaction equipment. The Project Geotechnical Consultant should evaluate and approve the acceptability of the compacted excavation bottom, prior to placement of new fill materials.

Excavation Procedures

Temporary excavations in site soils 4 feet or deeper should be shored or sloped in accordance with Cal OSHA requirements. Special construction techniques, such as slot cutting, may be utilized if excavations are greater than 4 feet vertical and site constraints preclude use of temporary slope cuts.

Excavations located along property lines and adjacent to existing structures (i.e. buildings, walls, fences, etc.) should not be permitted within two (2) feet from the existing foundations. Temporary slopes, if utilized, should be no steeper than 1:1 (horizontal: vertical) gradient. A representative of this firm should be present on-site during excavations to verify acceptability of temporary slopes. Acceptability will be dependent upon the soil conditions encountered, construction procedures and schedule. Excavations should not be permitted to dry out or deteriorate.

<u>Fill Placement, Moisture Conditioning and Compaction</u>

Prior to the placement of additional fill soils, if required, to achieve precise grade elevations, the exposed soils deemed suitable to receive new fills should be scarified 6 to 8 inches in depth and re-compacted to at least 90 percent relative compaction and at approximately 2-5 percentage points wet of optimum moisture contents (depending upon the soil type).

Fill materials should be placed in loose lifts not exceeding thickness that can be adequately and thoroughly processed and compacted by the equipment and methods utilized. These materials should be processed by blending and moisture conditioning to 2-5 percentage points above optimum moisture content, depending on the soil type, and compacted to at least 90 percent relative compaction based on the laboratory maximum dry density assigned – to the satisfaction of the Project Geotechnical Consultant. All grading should be performed under the observation and testing of the Project Geotechnical Consultant or their representative.

<u>Fill Materials</u>

Fill materials should consist of clean onsite or imported soils and should be free of vegetation, hazardous materials, construction debris, rocks greater than 6-inches maximum dimension, and any other organic or deleterious materials.

Import soils, if required, should consist of materials similar to or better than the onsite soils and should be approved by a representative of this firm, at the borrow site, at least 48 hours prior to importing.

All import materials are subject to evaluation and acceptance prior to being brought onsite. All import soils require a chain of custody and identification of original source. In order to provide adequate time for such assessment, at least 72-hours' notice (three business days) is required. Any import soils which are found not to the satisfaction of this office are subject to rejection and will need to be removed at the contractor's expense. "Manufactured" imported soils such as sand, gravel, base course, filter mix, etc., are also subject to the above, as well as the provision of manufacturer QA/QC testing certificates for review – preliminarily and for each load brought to the site.

Plans and specifications should indicate that the grading contractor shall notify the Project Geotechnical Engineer not less than 72 hours in advance of the location of any soils proposed for import. Each proposed import source shall be sampled, tested, and approved by the Geotechnical Consultant prior to delivery of soils for use on the site. Final acceptance of the proposed import will be based upon evaluation of the actual materials delivered to the site –

import not meeting the approval of the Geotechnical Consultant is subject to rejection and removal / replacement.

Testing and Observations

The site preparation, over-excavation, and earthwork should be performed under full-time observation and testing by a representative of the Project Geotechnical Engineer or Project Engineering Geologist. As a minimum, earthwork testing should include the following:

- Fill materials should be compacted to the minimum 90 percent relative compaction based on the laboratory maximum density determined in accordance with ASTM: D 1557 and fill should be tested at the time of placement to verify that the required compaction is achieved.
- The fill compaction should be determined in the field by the Sand Cone Method (ASTM: D 1556) or Nuclear Gauge Method (ASTM: D 2216), or other test method approved by the Project Geotechnical Engineer.
- During grading, an adequate number of field density tests should be performed using approved test procedures in order to determine compliance of earthwork to the project requirements. The frequency of field density testing should be in accordance with the recommendations of the Project Geotechnical Engineer and must comply with the minimum requirements of the County of Orange, City of Newport Beach, and/or other governing jurisdictions.
- Quality control testing performed to determine the acceptability of the fill compaction should be based on the laboratory maximum dry density and optimum moisture content determined in accordance with ASTM: D 1557 test procedure.

Any surface or subsurface obstructions, or questionable materials encountered during grading should be brought immediately to the attention of the Project Geotechnical Engineer / Engineering Geologist.

If it is determined during grading that site soils require over-excavation to greater depths for proper structural support, this additional work should be performed in accordance with the recommendations of the Project Geotechnical Engineer.

Temporary Shoring

Shoring design and installation will need to address several factors, including but not limited to: height of excavation, feasibility of restrained (internally braced or tied back) vs. cantilevered (free to rotate), soldier pile spacing and lagging, location of off-site improvements / supporting foundations / existing utilities, construction and vehicle traffic loading, and tolerance to lateral deflection.

Considering the excavation depth necessary for proposed subterranean level construction, and property line and existing structure constraints, temporary shoring should consist of a soldier pile/ lagging system, consisting of H-piles encased in concrete within drilled holes extending below the excavation level. Soldier piles will need to incorporate temporary tiebacks to provide lateral restraint and limit lateral deflection of the shoring pile. Presented below are preliminary criteria for use in temporary shoring design. The structural design requirements should be evaluated by a qualified and experienced shoring engineer.

Soldier Pile Criteria

Hole Diameter	:	24 inches (minimum);
Pile Embedment	:	15 feet (minimum, below excavation);
Lagging	:	Treated Lumber, pre-cast concrete panels or steel plate - properly sized and adequate for the intended purpose as evaluated by the project geotechnical and structural engineers;
Structural Concrete	:	Drilled hole filled with f'c = 4,500 psi (minimum) concrete below the excavation level, 1-1/2 sack cement-sand slurry above the excavation level.

For temporary shoring evaluation, the downward capacity for soldier piles may be determined based on a skin friction of 500 psf for that portion of pile embedded below the planned excavation level.

Drilling Sequence

Soldier piles should not be drilled sequentially such that adjacent pile excavations are left open. This means that pile excavations should be “staggered” such that no two adjacent pile excavations are open at the same time, nor are piles drilled before the concrete of adjacent piles has been given sufficient time to set. The acceptable timeframe for drilling piles adjacent to concreted piles should be determined by the project Structural Engineer.

Lagging

It is recommended that the lagging be installed as the excavation progresses providing positive contact with the retained earth to transfer earth pressures from lagging to soldier piles and to minimize adverse lateral movement of earth between soldier piles and subsequent subsidence. This may require grouting of any voids behind the lagging or back-packing with very low expansion / granular soils behind the lagging. It may be necessary to locally stabilize site sands, should a caving condition develop during installation of lagging between soldier piles.

Protection of Shoring Assembly and Backfill

Top of shoring assembly should be protected from inadvertant water infiltration – this may be done by grade sloping away from assembly, berming / sandbagging and sheeting. Voids behind lagging should be filled with slurry or similar material subject to review and acceptance by the geotechnical consultant. Cut and backfill operations should be performed incrementally at no more than 5 feet vertical intervals to limit voiding behind lagging. Complete infill of potential voids behind lagging should be evaluated and verified throughout construction.

Lateral Earth Pressures

The lateral earth pressures for shoring design would be dependent upon the wall conditions (restrained or unrestrained) and a function of the soil materials behind walls. In order to limit lateral deflection of shoring adjacent to existing adjacent structures located off property, at-rest (restrained) earth pressures should be considered in design. Presented below are the recommended lateral earth pressures, presented as equivalent fluid pressures for temporary shoring, based on at-rest earth pressure considerations, to limit lateral deflection of shoring piles.

Piles:	60 pcf – efp
	30H (trapezoidal distribution)
Lagging (*):	40 pcf, to a maximum value of 400 psf

(*) The earth pressure on lagging is less due to arching effects

The surcharge load effects from existing and anticipated loads (e.g., structures, construction loads, and vehicle traffic/ parking should be included in design when such loading is within a distance from the shoring equal to the depth of excavation.

Lateral Resistance

Passive Earth Pressure (*): 300 pcf, not to exceed 3000 psf

(*) An effective width of twice the pile diameter may be used in computing lateral resistance for piles, when considering minimum soldier pile spacing of 3 diameters center to center.

Point of Fixity

For lateral resistance determination, a point of fixity of 5 feet into competent and undisturbed soils should be utilized in design.

Pile Deflection

Maximum lateral deflection at top of piles should not exceed ½ inch. A baseline survey of top of soldier piles should be performed following soldier pile installation and subsequent survey readings performed during and following excavation operations to achieve the planned subterranean level of construction.

Soldier Pile Installation

Presented below are general guidelines for soldier pile installation:

- The pile locations should be accurately surveyed and staked in the field by the Project Civil Engineer.
- Pile excavations should be drilled at the design locations within the tolerances for lateral deviation and plumb condition specified by the project structural engineer.

- The pile excavations should be accomplished using proper drilling equipment with sufficient power to advance the holes to the required design tip elevation.
- H-piles should be installed in holes drilled to minimum specified diameter. Concrete should be placed and vibrated to the excavation level, with H-pile extending above to receive the lagging. The balance of the annulus should be backfilled with sand-cement slurry (1-1/2 sack cement minimum). H-pile excavations should be staggered and backfilled with concrete / slurry so that no two adjacent soldier pile excavations are open at the same time.
- Due to the predominantly granular nature of the existing site soils, potential caving/ sloughing may be experienced when excavating in this material. Casing should therefore be made available on site in the event that caving is experienced during the drilling of pile shafts. If temporary casing is required to facilitate drilling, a minimum 5 feet of casing (as feasibility possible) should be maintained below the concrete surface concrete placement.
- In lieu of temporary casing, polymer slurry to stabilize the drill holes against caving and instability may be considered. Use of polymer slurry application procedures are subject to prior approval by the Project Geotechnical Engineer.
- Pile excavations should be thoroughly cleaned of loose soils and cuttings and verified as clean and suitable just prior to placing steel beams and concrete;
- Soldier pile excavations should be logged and installation observed and documented by a representative of our office.
- Pile excavations should be filled with concrete on the same day that they are drilled. Concrete should be placed using the Tremie Method to prevent aggregated segregation.
- Pile concreting operations should be inspected by a qualified Concrete Deputy Inspector. The Concrete Deputy should maintain field records including, but not limited to, estimated concrete quantity used for each pile and details of concrete mix design.

Proper measures to control surface and subsurface drainage of rainwater, groundwater seepage, etc. should be made available during soldier pile drilling/ installation and excavations. It is imperative that these surface/ subsurface drainage provisions (e.g., sump/ pump, erosion control, etc.) at the shoring and bottom provide clear positive drainage away from the shoring system.

Tiebacks

The installation of temporary tiebacks will be needed to provide lateral shoring system restraint and limit lateral deflection of the shoring pile to accommodate the proposed excavations for the subterranean levels of construction.

Preliminary Design Criteria

Presented below are preliminary geotechnical criteria for the design and construction of tieback anchors:

Preliminary Tieback Anchor Design Criteria

Minimum Shaft Diameter	:	6 inches (minimum)	
Angle of Anchor (from horizontal) (*)	:	10-20 degrees	
Bond Strength (**):	:	500 psf	:
Anchor Spacing	:	5 feet (minimum)	

(*) Final design locations/ inclination of tieback anchors may be dependent upon the existing off-site utilities and foundations.

(**) For pressure grouted anchors, bond strengths are expected to be greater, and a bond strength of 2,000 psf may be assumed for preliminary design purposes, Actual bond strengths would be dependent on the means / methods employed by the specialty contractor and should be confirmed / substantiated by anchor load testing.

The bond strength is applicable for that length of tieback extending beyond the active wedge adjacent to shoring, defined as a projected plane at a 35 degree angle from the vertical, extending upwards and back from the bottom of excavation. The un-bonded length of tieback (within the active wedge) should be sleeved and backfilled with sand.

Tieback installation should be performed by a qualified and experienced contractor knowledgeable in shoring / tieback construction with full-time observation and documentation by a representative of this office.

Verification and Proof Tests

Anchor load testing should, as a minimum, conform to the following requirements or as dictated by controlling jurisdictional agency, whichever is more restrictive, Tieback testing procedures should be provided by our office for acceptability and comments, as appropriate.

- For anchor systems, the estimated bond strength should be verified by a pre-production test program on the initial anchor, with anchor incrementally loaded to 200 percent of the design capacity. Should unsatisfactory results be obtained in these preproduction anchors, the anchor diameter and/or length should be increased to achieve satisfactory results.
- Approximately 10 percent of the production anchors should be quick tested to 200%, with a portion of these anchors selected for 24-hour 200% tests.
- Remaining production anchor should be satisfactorily tested to a minimum of 150 percent of the design load. The total deflection of anchor rod including rod stretch should not exceed 12 inches during the application of the test load from 0 % to 150 %. At 150% test load, the anchor movement should not exceed 0.1 inch during a 15-minute test

period for the anchor to be accepted. Where satisfactory are not achieved on the initial anchors, additional post-grouting and/ or increase in anchor diameter/ length should be performed until satisfactory tests are obtained.

- No anchor test shall be performed until the concrete has attained 100% of the design 28-day compressive strength.
- Anchors determined to be acceptable by testing should be locked off at the design load
- Certification from an approved testing laboratory is required for the calibration of the anchor loading devices at the start of the job.
- A complete record of anchor testing (anchor loads, anchor movements, etc.) should be maintained. The Geotechnical Engineer should be retained to monitor, inspect, and assess adequacy of testing of anchors from a geotechnical standpoint.

Detailed anchor testing procedures can be provided upon request and authorization at a later date, following review of tieback anchor layout and schedule.

Permanent Shoring

As the proposed building is located in close proximity to property lines, the feasibility of integrating the shoring system with the structural elements of the subterranean levels of construction may be evaluated by the Project Structural Engineer. The combined shoring/ retaining wall system should be adequately designed and constructed for the long-term performance and service loading conditions, including the potential seismic earth pressures presented in the Retaining Walls section of his report as well as designed to resist corrosion and degradation. Permanent shoring walls should also include installation of appropriate waterproofing and blindside drainage.

Foundations

Presented in the following sections are the recommended geotechnical criteria for use in foundation design and construction for the proposed residence.

Structure Foundations

Presented below are geotechnical criteria for use in design and construction of conventional foundations to be located at the lowest subterranean level of construction, and when supported on approved native soils:

Allowable Bearing Pressure (1)	:	4,000 psf
Width	:	48 inches (minimum)
Embedment (2)	:	24 inches (minimum)
Passive Soil Pressure (2)	:	300 psf/ft, not to exceed 3,000 psf.
Friction Coefficient	:	0.35 (ultimate)
Reinforcement	:	Per Project Structural Engineer

(1) The allowable bearing pressure may be increased by 500 psf for each additional foot of embedment and by 300 psf for each additional foot of foundation width, to a maximum bearing pressure not exceeding 6,000 psf. A one-third increase in the allowable bearing pressure may be used to resist transient loads due to wind or seismic forces.

(2) Below lowest adjacent finish soil grade.

Mat Foundation Considerations

Depending upon the structural demand requirements, a mat foundation system at the subterranean level of construction may be more appropriate for structure support. A mat foundation distributes structural load across the structure footprint, resulting in more uniform applied pressures to the bearing stratum. The contact pressure distribution beneath the mat is a function of foundation rigidity and the type of bearing material. The contact pressure distribution beneath the mat is a function of foundation rigidity and the type of bearing material.

The mat can be designed as a flat, reinforced concrete slab of uniform thickness, if the center of gravity of the column loads and the centroid of the mat coincide approximately. For this, the column loads should be distributed in such a way that the contact pressure below mat will be nearly uniform. For columns supporting heavy loads, it may be necessary to thicken / stiffen the mat below the column to provide sufficient strength for shear and negative bending moment or to incorporate a pedestal at the column base.

If the column layout and loads are non-uniform, a thicker mat with heavy reinforcements may be necessary. As an alternative, a relatively thinner mat stiffened with concrete beams along two orthogonal directions (inverted T-beam construction) may be considered.

For mat foundation design, the following criteria may be considered:

Allowable Bearing Pressure (*):	4,000 psf
Mat Thickness:	Per Structural Engineer
Passive Earth Pressure:	300 psf
Coefficient of Friction:	0.35 (ultimate)
Modulus of Subgrade Reaction (“k”-value):	50 pci

(*) The above allowable bearing value may be increased by one-third to resist transient loads such as wind or seismic.

The structural details of the mat foundation such as thickness, reinforcements, concrete strength, etc. should be established by the Project Structural Engineer, considering the loading and service conditions. The mat should be adequately reinforced based on structural design considerations and/or Code requirements.

Foundation excavations should be observed and approved by the Project Geologist and Geotechnical Engineer prior to the placement of reinforcement or concrete.

The mat foundation should be underlain by a subdrainage system and moisture vapor retarder, as recommended hereinafter, to mitigate moisture / water vapor intrusion.

At-Grade Construction

The following footing criteria may be used for the design and construction of at-grade footings supporting ancillary construction, when supported on an improved horizon of engineered compacted fill:

Allowable Bearing Pressure (1)	:	2,000 psf
Width	:	Per Code requirements
Embedment (2)	:	18 inches (minimum)
Passive Soil Pressure (2)	:	250 psf/ft, not to exceed 2,500 psf.
Friction Coefficient	:	0.3 (ultimate)
Reinforcement	:	Per Project Structural Engineer

(1) Allowable bearing pressures may be increased by one-third for short term loading due to wind or seismic forces.

(2) Below lowest adjacent finished soil grade.

General Foundation Construction Comments / Guidelines

General remarks regarding the construction of conventional footings are presented below:

- Footing embedment depths should be maintained throughout the life of the structure, and not compromised via erosion softening, digging, landscaping, etc.
- Where foundations encroach closer than five (5) feet horizontally from the flow line of drainage swales, the footings edges should be deepened sufficiently to maintain the required embedment depth below the adjacent flow line.
- Foundation details such as concrete strength, reinforcements, thickness, etc. should be established by the Project Structural Engineer, considering the loading conditions. The recommended foundation embedment, thickness and reinforcements are minimum requirements and should be established by the Project Structural Engineer. More restrictive criteria based on structural design considerations or Code requirements shall govern.
- Foundation excavations should be observed and approved by the Project Geotechnical Engineer prior to the placement of reinforcement or concrete. Forming of footing excavations may be required. Excavations should be free of slough and debris and thoroughly moisture conditioned prior to placing concrete.
- Excavated material from footing and utility trenches should not be placed in slab-on-grade areas unless properly compacted and tested.

- Footings should be doweled to the floor slab, with dowels tied into the reinforcement per structural engineer requirements.
- Floor slabs should be underlain by a moisture vapor retarder, as recommended hereinafter, to mitigate moisture / water vapor intrusion into the structure.

<u>Settlements / Adjustments / Deformation of Excavations</u>

In general, the bottom of deep excavations would be expected to experience heave due to stress release. As the site soils consists predominantly of sandy soils, the foundation heave due to stress release is expected to be negligible and will be mostly elastic and develop rapidly during and following excavation. As any stress relief would be in the direction of excavation, the foundation settlement due to the structural loading is expected to be small. Based on the above considerations, long-term foundation deformation / settlement should not adversely impact proposed development.

Some structure movement should be expected both during and following construction, even when supported in competent bedrock, due to various factors including, but not limited to:

- Sequence of foundation and slab loading during construction;
- Variation in structural loads along foundation elements;
- Variation in underlying material types with different compressibility indices and subsurface profile, and associated primary and long-term secondary consolidation settlements;
- Reorganizations / establishment of new equilibrium with regard to soil moisture and stress distributions.

It should also be recognized that given general construction tolerances, concrete floor slabs will not be cast / built perfectly level, and it has been our experience that floors slab elevations across a structure may, as built, commonly vary by as much as an inch or more. It is recommended that slab areas have a floor level survey performed following construction to establish an as-built baseline.

Presented below are preliminary settlement estimates for at-grade ancillary foundations and structure foundations at the subterranean level (assumed 40 feet below existing site grades). These preliminary settlement estimates should be confirmed based on the actual loading and foundation conditions.

Ancillary At-Grade Construction

Total static settlements for shallow at-grade conventional footings supporting ancillary construction and bearing on approved engineered compacted fill, when designed and constructed as recommended above and supporting loads not exceeding about 2 kips per lineal foot and 20 kips for wall and column loads, respectively, are not anticipated to exceed about 1 inch. Differential settlements, when founded on uniform fill prism, are expected to be less than 3/4-inch between similarly loaded adjacent footings and along a distance of about 30 feet.

Excavation Bottom Heave

In general, the bottom of deep excavations would be expected to experience heave due to stress release. As the site soils consists predominantly of sandy soils, the foundation heave due to stress release is expected to be less than about 1-1/2 inches and will be mostly elastic and develop rapidly during and following excavation. As any stress relief would be in the direction of excavation, the foundation settlement due to the structural loading is expected to be small. Based on the above considerations, long-term foundation deformation / settlement should not adversely impact proposed development.

Subterranean Level Foundations

The potential settlement of foundations constructed at the subterranean level of construction would be dependent on the foundation configuration and load conditions, location of footings, and recompression of potential excavation bottom heave in response to new structure loads.

For preliminary design purposes and based on an assumed maximum column loads of 3,000 kips and wall loads of 30 kips/lineal foot, total settlements are expected to be less than 2 inches. Differential settlements are not expected to exceed 3/4 inch between similarly loaded adjacent footings and along a distance of about 30 feet.

For mat foundation construction, total and differential settlements are anticipated to be less than 1-1/2 inches and 1-inch over a distance of about 30 feet, respectively.

Approximately two-thirds of the above estimated settlements are expected to be immediate and occur during construction, with remaining settlement occurring long term.

Utility Line Tie-Ins

For utility connections to the street mains made during the latter part of construction, a majority of structure settlement would be expected to have occurred by that time. As such, post-construction settlement along these connections, following their emplacement, is expected to be minor. If utility lines are to be installed and connected during the initial phases of construction, this office should be notified for additional recommendations, as appropriate.

Seismic Design Considerations

The site and southern California, as a whole, are within a zone of active seismicity. Strong ground motion from an earthquake generated along active faults should therefore be anticipated at this site. The proposed residence should be designed and constructed to the prevailing standards regarding seismic design. Seismic design should be based on current and applicable CBC requirements, as appropriate. Seismic design parameters based on ASCE 7-16 are included in Appendix E.

Water Vapor Mitigation Beneath Concrete Floor Slabs / Mat Foundations and Basement Walls

It should be recognized that, in particular for the portions of development which extend below ground, even with prudent site surface and sub-drainage measures, there is potential for saturation of the subgrade materials beneath the slab system due to water infiltration as well as a potential for the intrusion of moisture-vapor as a result of unsaturated soil-moisture transmission. Because of this, there is a potential for upward/ inward migration of moisture in both capillarity and vapor phase from the subgrade through the mat / slab under normal living conditions as they exist within a closed environment (e.g., building interior). In order to reduce the potential for moisture / water vapor migration up through the foundation / floor slab and possibly affecting floor covering, a moisture vapor retarder is recommended under concrete floor slab-on-grade. Presented below are the recommendations based on the tentative guidelines by the American Concrete Institute (ACI, April 2001) to reduce the potential moisture / water vapor intrusion in concrete slab-on-grade:

- The moisture / water vapor retarder should consist of high strength polyethylene membrane and should meet or exceed the ASTM: E-1745-97 Class A material requirements for water vapor permeance, tensile strength, and puncture resistance. The vapor retarder should be underlain by a capillary break comprised of minimum 4 inches 3/4-inch gravel. The gravel layer should be placed and compacted on approved soil sub-grade.
- The membrane should be placed on approved gravel layer and properly lapped and sealed. Membranes intersecting utility pipes, sewer lines, ducts, or drains must be properly wrapped around the penetrations and sealed. All punctures and rips in the membrane should be repaired prior to placement of concrete, following manufacturer's recommendations. The vapor retarder should be installed in general accordance with the procedures outlined in ASTM: E-1643, and in conformance with the installation procedures recommended by the manufacturer.
- For basement / inset levels of construction it is recommended that the above moisture water vapor retarder system be underlain by a floor slab subdrainage system as recommended hereinafter.
- The use of a hydrophobic admixture in the concrete placed for subterranean level construction (as recommended in the Concrete section presented hereinafter) would also provide added measure to mitigate potential water / moisture vapor transmission into subterranean level construction and is highly recommended.

It is imperative that the Contractor properly install the recommended site drainage measures as discussed in the Site Drainage / Subdrainage / Wall Backdrainage Section of this report and the moisture / water vapor retarder system in accordance with the project design requirements and specifications, to collect and discharge potential subsurface water and mitigate potential moisture / water vapor transmission into the structures.

It should be emphasized that, even with proper moisture / water vapor installation, proper control of irrigation and landscape water adjacent to the structure is very important to minimize problems caused by moisture and water vapor intrusion and is the responsibility of the ultimate owners / operators. In addition, the ultimate owners / operators are responsible for maintaining proper site drainage as recommended hereinafter.

Concrete Slab and Exterior Flatwork

The concrete slab design and construction details should be established by the Project Design Engineer. As recommended in the Site Drainage / Subdrainage / Wall Backdrainage section presented hereinafter, surface and subsurface drainage should be isolated from each other, and wall/ floor subdrainage systems should be integrated to provide positive/ aggressive free water removal.

From a geotechnical standpoint, the minimum criteria for slab-on grade are shown below:

Floor Slabs

Concrete floor slabs should be 5 inches thick (minimum) and should be reinforced with No. 4 bars at 18 inches on center, each way at mid height, and should be structurally tied to the footings. In order to minimize migration of moisture up the concrete slab from soil sub-grade and damage to floor coverings, a moisture barrier / water vapor retarder system should be placed beneath floor slabs as recommended hereinafter.

Exterior Flatwork

Sidewalks, walkways and patio slabs should be 4 inches thick (minimum) and should be reinforced with No. 4 bars at 18 inches on center, each way at mid height.

Concrete Driveways

Concrete driveways should be at least 6 inches in thickness and reinforced with No. 4 bars at 18 inches on center, each way. The concrete slab should be underlain by a 4- inch layer of crushed miscellaneous base compacted to at least 95 percent relative compaction. Joint spacing should not exceed about 8 feet on centers, each way. The driveway entrance and garage floors should include positive sheet flow to suitable drainage devices. These should be carefully designed and constructed to collect and direct the water to suitable discharge facilities / locations.

Slab Sub-Grade Pre-saturation

Prior to concrete placement, the prepared soil sub-grade should be moisture conditioned to and maintained at about 2 to 3 percentage points wet of optimum moisture contents to a depth of 12 inches and exhibit at least 90 percent relative compaction as determined by ASTM: D1557 or as otherwise specifically recommended by the Project Geotechnical Consultant based on actual conditions.

General Concrete Remarks

To minimize slab curling and other related adverse effects, a low shrinkage / low slump concrete (concrete mix with a minimum 4,500 psi compressive strength and maximum water cement ratio of 0.45) should be used for the slab construction, as determined by the Project Structural Engineer. The mix design should be verified by the project Civil / Structural Engineer, and placement of concrete should be observed and certified by the Concrete Deputy Inspector.

- Interior floor slabs and exterior concrete flatwork should be properly designed for the construction and service loading conditions, and potential differential movements. The structural details, such as slab thickness, concrete strength, reinforcing criteria, joint spacing, etc. should be established by the Project Civil / Structural Engineer. The recommended minimum reinforcements for concrete slabs provided above are intended for preliminary design only. More restrictive criteria as dictated by structural design or regulatory requirements shall govern.

- All reinforcement must be appropriately spaced and supported / maintained during the pouring / finishing work such that it remains in proper condition.
- Unless specifically allowed for and approved as such by the project Civil / Structural Engineer, no water is to be added to the concrete mix after the truck leaves the plant. It should be cautioned that addition of water to the concrete mix will change the water-cement ratio of the plant design mix and can lead to undesirable shrinkage cracking, curling, etc. of concrete slabs during curing. Plant should be made aware of this and appropriate fluidizers / admixtures should be added to improve workability without increasing shrinkage potential
- All concrete to be properly finished per American Concrete Institution / Portland Cement Association standards and moist cured (for preferably at least 7 days). If moist curing is not feasible, an appropriate curing compound / sealant should be applied in accordance with the timing and methodology specified by the curing compound manufacturer.
- Truck tickets to include mix design, time leaving plant, time of site arrival, and time onsite / location of pour to be documented and copies sent to the project Civil / Structural Engineer.
- All poured concrete should be protected from loading and traffic for at least 7 days without written approval of the project Structural engineer.
- Concrete should not be allowed to "free-fall" into excavations and similar and appropriate tremmie methods should be used. Special mix considerations should be utilized, as appropriate, where concrete may be placed under water. Concrete placed under water requires special tremmie procedures.

Basement / Retaining Walls

The earth pressure acting on retaining walls depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. For basement walls, wall movement necessary to develop active earth pressure conditions will not occur and therefore basement walls should be considered restrained.

Retaining walls should be adequately waterproofed and incorporate a wall backdrainage system during construction to relieve hydrostatic pressure buildup behind walls. Recommendations for retaining wall backdrainage are presented in the Wall Backdrainage / Floor Subdrainage section of this report update.

The following lateral earth pressures are recommended for the design of cantilevered and basement / restrained walls:

Lateral Earth Pressures

The following lateral earth pressures are applicable vertical walls, level soil conditions behind wall, no hydrostatic pressure, and backfill materials comprised of predominantly granular non-expansive soils.

Active Earth Pressure (Static):	40 pcf
At-Rest Earth Pressure (Static):	60 pcf
Seismic Earth Pressure Increment:	35 pcf

Surcharge load effects from existing and anticipated loads (e.g., structures, construction loading and vehicle traffic / parking) should be included in design.

For both active and at-rest case, the above seismic earth pressure increment (triangular distribution) should be combined with the active earth pressure.

Wall Backfill

Backfill materials behind walls, if needed, should consist of predominantly granular, free draining material, and compacted to at least 90 percent relative compaction and near optimum moisture contents.

Site Drainage / Subdrainage / Wall Backdrainage

It is extremely important to design and construct suitable site drainage collection and discharge systems for the planned development. This should consist of both the surface and above grade drainage elements (i.e. surface sheet flow, roof, deck, and perimeter drainage and collection with associated discharge pipe systems to suitable discharge locations) as well as all subsurface drainage elements (i.e. wall backdrainage, first floor slab subdrainage, and associated discharge pipe systems to suitable discharge locations). Surface and subsurface water drainage systems should be kept separate. Drainage devices and waterproofing should be designed by the Project Civil Engineer. Drainage and, particularly, waterproofing need to account for all forms of water that may be present - including free liquid (hydrostatic), capillarity, moisture, and vapor.

The surface and subsurface drainage systems should be included in the project plans and specifications. All surface and subsurface drainage systems should be designed and constructed to remain hydraulically isolated from one another – at no point should surface water and subsurface water be allowed to comingle.

Roof, Deck, and Perimeter Site Drainage

The roof, deck and perimeter surfaces should include positive sheet flow to suitable drainage devices. These should be carefully designed and constructed to collect and direct the water to suitable discharge facilities / locations.

Subterranean Walls

Subterranean walls should include waterproofing for moisture / capillarity / vapor mitigation and a drainage blanket behind the walls for relieving possible hydrostatic pressures and free-water. As a minimum, this drainage blanket should consist of Miradrain 6200 drainboard and attendant drain collector and related components, and conveyance plumbing to suitable outlet. Miradrain 6200 is specifically called for given its mylar backing that protects waterproofing components from eing damaged by die-cutting effects. It is imperative that all wall subdrains/ floor underdrains / subdrains be isolated from communication with surface drainage. The waterproofing should be of a bonded impervious membrane or similar system specifically intended for the proposed application by the manufacturer. The system should address moisture, vapor, and capillarity water.

The wall drainage system should be directed into a perimeter drainage system which should consist of minimum 4-inch diameter perforated PVC drain pipe (Schedule 40), surrounded by minimum 3 cubic foot per lineal foot of ¾-inch gravel wrapped with geofabric (e.g., Mirafi 140N). The drains should be directed and connected to suitable drainage / outlet facilities.

Basement wall drains are to be hydraulically isolated, along with their outlet lines from surface and other sub / wall drains, and in particular, from the floor slab subdrainage.

Dedicated waterstops should be provided at all joints and seams and at interconnection to the basement / inset level floor slab. Details should be included on the project plans and provided to our office for review and comment.

Floor Subdrainage

This system will require careful consideration regarding positive flow, intersection of utility lines, and other construction interference. In addition, this system will require careful design and implementation to limit cross-communication between drains, provide positive drainage to suitable collection points, and ensure drainage is directed in concert with the wall drainage systems.

Basement level floor slabs should be equipped with a combination vapor retarder and underdrainage system. The vapor retarder should be in accordance the above "Moisture Vapor Retarder System" section which includes an underlainment of 4-inch layer of 3/4-inch gravel . The gravel layer should be laid on a grade that slopes preferably at least ½-percent to a collector drain. The floor slab subgrade should be graded with minimum 1 percent fall to collector drain trenches comprised of 4-inch perforated pipe (Schedule 40 PVC, ½-inch holes, installed holes down) and encased in at least 2-cubic feet per foot ¾-inch gravel. The gravel underdrain and collector should join seamlessly, and the entire assembly separated from underlying grade by Mirafi 140N or approved equivalent filter fabric. All filter fabric should be shingle lapped (where feasible / applicable) and overlapped at least 2 feet and appropriately sealed.

Subterranean plumbing elements that lay below floor underdrains should be slurried-in-place using a minimum 2-sack sand-cement slurry, preferably with a bentonite or similar additive to limit permeability and shrinkage. The slurry should not be allowed interfere with the underdrain or other drainage elements and should not be carried above the underdrain base line.

This system will require careful consideration regarding positive flow, intersection of utility lines, and other construction interference. In addition, this system will require careful design and implementation to limit cross-communication between drains, provide positive drainage to suitable collection points, and ensure drainage is directed in concert with the wall drainage systems.

Additional details of this system can be presented under separate cover and incorporated into the project plans and construction.

Good Words about Waterproofing Walls and Slabs

Introduction

Water permeation through floors and walls, particularly where they are below grade, has been an ongoing problem for builders since the dawn of civilization. This is due largely to the insidious nature of water, its ubiquitous occurrence around and under structures, and the different forms water takes – often simultaneously. Combined with gradients that draw water and induce flows that are not necessarily gravitational, and not necessarily intuitive – as well as factoring in entropy and tiny defects present from construction – water often finds ways around engineering solutions to prevent it.

Notes on the Nature of Water and Flow

In addressing waterproofing and damp proofing issues – effectiveness in design and construction is critically dependent on first having a firm grasp of the intrinsic behavior of water and how it interacts with soils and construction materials. The first stage of this understanding is related to the nature of water itself. Key factors include:

- Water is a polar material that has high surface tension. Water molecules have strong affinity for each other and also for many substances like silica, clay and related minerals, etc. which is what makes it "wet." It also is what makes it have high capillarity – the ability to wick into and through substances it finds attractive.
- Water exists in the subsurface in multiple forms – as free liquid water present in void spaces, which is mobile and will flow via gravity through suitably porous materials; as moisture that is adsorbed onto and held relatively tightly by soil grains and material surfaces which does not flow as a liquid; as capillary water existing in very thin threads and seams – i.e. "wicking water" as would be present in a paper towel with an edge dipped in a bowl of water, that also does not flow as a free liquid; and lastly in the form of vapor, which is not a liquid but rather a gas – i.e. humidity.
- Liquid water, in the form of free-flowing groundwater and seepage typically flows under hydraulic gravitational gradients and pressure heads. In other words, it tends to flow downhill and accumulate in low spots. Where it builds up, it exerts a force on its

surroundings of about 62 to 63 pounds per square foot per foot of depth. This represents water most people are familiar with.

- Moisture and capillary water are much more controlled by surface tension and "suction" potential of the materials they are in contact with. These water forms literally chain-gang and climb over themselves following "suction" gradients which can literally wick them uphill. Items with relatively high potential suction gradients include clays dry of the plastic limit, cured concrete, dry wood, paper, and cotton. Relatively low suction potential materials include open graded gravel, plastic, and metal. Thus, water in these forms can flow uphill and tend to flow from wet areas into drier ones. The free energy gradient is a function of the moisture differential – dry clay can have gradients that can lift water by capillarity tens of feet. Dry concrete and cement board may have capillarity lift gradients measured in feet. Conversely, low suction materials such as open graded coarse gravel in comparison be only a fraction of an inch, and in plastic or similar – virtually nil.
- Vapor is not a liquid, and thus under normal atmospheric physical conditions on Earth, is not appreciably controlled by gravity for the purposes of discussion here. For discussion here, its development is a function of the atmosphere of its surroundings (pressure, temperature, and composition) and the presence of local sources of liquid water – combined with the relative humidity of the atmosphere. Vapor develops in a space with liquid water present by changing phase through evaporation from liquid to a gas. The tendency for this phase change is related to the partial pressure of water vapor and the "saturation" of the immediate atmosphere. Evaporation in a closed container will only continue to the point that 100% saturation (or 100% humidity) is developed. The amount of water vapor that can be held in the air is both a function of air density and more importantly, temperature. Warmer air can hold more water vapor than colder air – readily observable by the formation of condensation on a bath mirror. The warm air at near 100% saturation becomes super saturated when cooled by contact with the cool mirror surface, forcing the vapor out to condense as liquid water. Thus, water vapor is controlled by temperature, humidity, and air pressure differences and can flow following such free energy gradients in any direction they exist.
- These three different forms commonly co-exist in the subsurface. These different forms can interact and interchange with one-another in response to the thermodynamics of their environment – and changes in the environment.
- These forms must all be taken into account, and addressed both separately and combined – if effective water / moisture proofing is to be achieved.

<u>Key Takeaway Points About the Intrinsics of Water</u>

- Water is present in the subsurface in multiple forms.
- Water flow is driven by multiple gradients – including gravity, suction potential, capillarity, humidity, pressure, and temperature.

- Water in liquid form flows under gravity and develops a pressure controlled by its head – the shape of the accumulation does not play a role – a thin seam a fraction of an inch wide exerts the same fluid force per unit of depth as a broad reservoir (this has also been referred to as the hydraulic paradox).
- Water in capillary / suction form is driven by surface tension and held by suction. Although a liquid, it can move laterally and uphill a considerable degree. Another important factor is that it is NOT available to flow into open drains (ie. French drains, perforated pipes, etc.). An analogy to this is one would not try to dry a wet towel by setting it over a drain grate. The water less than saturation would simply sit in the towel until it evaporated. This water tends to flow from wetter to drier.
- Water vapor is not a liquid, but rather a gas. It is a product of evaporation of liquid or solid water (ice). Its movements are not governed by surface tension or other factors governing liquid forms, but rather thermodynamics of gasses. Water vapor tends to flow from high humidity to low humidity. Water vapor may readily condense back into liquid due to temperature and pressure changes.

<u>Application of Understanding of the Above Intrinsics to Effective Waterproofing</u>

It is paramount – given the above discussions – that it be realized there is:

- Free Liquid Water
- Capillary Water
- Water Vapor

All are likely to be present in the subsurface. All are likely to interact with the proposed construction of which they are in contact with. All three forms must be addressed separately if effective mitigation is to be achieved. Water is INSIDIOUS – like rust, it never sleeps – unlike rust, it can change forms and means through which it can move – it WILL find a way across barriers where even tiny defects exist, and it WILL focus on such defects.

Know This:

1. Free liquid water WILL tend to accumulate around structures, particularly ones landscaped, irrigated, and in contact with or below grade.
2. Free liquid water exerts pressure against things it accumulates in and WILL find the smallest of defects through which to flow.
3. Free liquid water WILL change to capillary water or vapor.
4. Methods to drain and control free water WILL LIKELY NOT be effective at controlling capillary or vapor water forms, and often these methods allow water a relatively easy way to access areas it did not previously have access to.
5. Methods to control capillary water WILL LIKELY NOT be effective at controlling free liquid and require careful consideration in controlling vapor.

6. Fine grained soils and concrete / mortar / cementitious materials have high suction and capillarity potentials – they can readily wick capillarity water even though they seem impervious to liquid flow.
7. The transition between the subgrade and interior of a building is the wall / slab. Gradients exist across these that tend to flow INTO the building.
8. The three different water forms can readily interchange in response to the thermodynamics of their surroundings.
9. The ground and subgrade tend to be of high humidity – near 100%. The interior of a building area, especially where air conditioned – is typically much less. This creates a thermodynamic free energy gradient.
10. The often cooler interior of a building relative to the outside soil is a thermodynamic gradient that not only induces flow from warmer to cooler, but condensation of vapor into liquid – especially where it is forced to build up (say under wood or tile flooring and impervious wall coverings).
11. Salts present in liquid water and leached from materials by passing / condensing vapor will be left behind on the surface where evaporation occurs.

Controlling this requires careful combinations of methods:

- Free liquid water requires a drainage device and physical impervious barrier that can intercept such water and drain it off and cut off a means of transmission allowing contact with the substance to be protected.
- Capillary water control requires a device that can “break the chain” of water molecules – such a plastic, metal, or bituminous / hydrophobic layer or sheeting that water will not conduct through by wicking.
- Vapor control requires means similar to capillarity control, but must be vapor-tight.
- Transmission of all three forms can be retarded further by doing everything possible to limit the transmissivity of the wall / slab itself and by eliminating any defects, joints, gaps, cracks or similar that pass-through can occur by. Water can only accumulate and cause damage where its influx is more rapid than its outflux. If it can dissipate / evaporate safely faster than it comes in, its impacts are minimized.
- With regard to concrete and similar – permittivity is partly a function of the density and effective porosity of the concrete itself. Making the concrete as watertight as possible will go a long way to limit permeation issues. This means making the concrete with as low a water cement ratio as possible – ideally no more than about 0.45, and using fluidizers and hydrophobic additives that limit the capillary affinity. Proper finishing and curing is hand-in-hand with mix design. Proper finishing and hard troweling followed by wet / moist curing will go a long way.
- In theory, the most effective water-stopping involves:

- Control influx of water into the subgrade around the structure.
- Limit joints and separations in the work. Any seams or joints must be properly designed and constructed to account for soil, movement, and degradation effects.
- Intercept free liquid water and drain it as efficiently as possible.
- Cut off inevitable accumulations of free liquid water with an impervious barrier.
- Cut off contact of that barrier with a capillary break.
- Make sure the break has intimate connection to the back of wall / bottom of slab such that water cannot develop and accumulate.
- Make the wall / slab as impervious and watertight as possible by proper mixture design and placement / finishing / curing.
- Provide drain redundancies to limit failures of systems by clogging or interruptions.
- Take the time and care to ensure the works to be as defect free as possible.
- Provide for common sense prudent long-term maintenance to identify and repair small defects or problems before they become larger.
- Use common sense in assigning floor and wall coverings. Use floor and wall coverings that have higher transmissivities to moisture / vapor where water may be anticipated.

<u>Inevitabilities</u>

- Water is insidious – it will find a way in through the smallest of defects.
- Water can change forms and flow in ways which are not intuitive.
- Some moisture / vapor transmission is INEVITABLE regardless of methods used.
- Some defects are INEVITABLE in construction.
- Degradation and declines in effectiveness are INEVITABLE with most construction products.
- All basements become damp.

<u>Key Aspects of Subdrainage and Waterproofing Installation / Construction</u>

Several engineering solutions / construction methods are recommended in order to mitigate moisture / vapor intrusion of subsurface water into the interiors of structures, including:

- Surface drainage;
- Subdrainage;
- Waterproofing membranes / systems;

- Moisture-vapor retarder systems;
- High strength / low permeability concrete.

All four of these waterproofing elements will be of critical importance in the success of the project in terms of preventing the damaging effects of moisture / vapor intrusion.

The application of drainage and high-strength / low permeability concrete will provide the greatest level of protection with emphasis on the latter. The primary control, aside from physical drainage control, of water / moisture / vapor is the concrete wall / slab foundation system itself with secondary controls afforded by waterproofing coatings. Design and construction of these systems are ripe with pitfalls that can limit the effectiveness of any one system. It is CRITICAL that these systems provide some redundancy such that any localized failure stemming from construction and/or material defects of any one component (e.g., floor subdrain, wall subdrain, moisture / vapor retarder system, concrete, area drainage) can be compensated for by the other components. The following sections will describe the key details of construction regarding these individual components.

<u>Surface and Subsurface Drainage</u>

Surface Drainage is the first line of defense against subsurface water infiltration. Area drains and gutters are important components in any waterproofing design that capture and discharge excess runoff before it has the opportunity to infiltrate into the subsurface. Subdrainage is the second line of defense against free liquid water that has either bypassed surface drainage, as in the case of nuisance water, and/or exists as part of the native groundwater regime which is currently not affecting the project site. In the absence of adequate surface and subsurface drainage, groundwater and nuisance water alike can build up and flow under gravitational pressure and other gradients that are less intuitive through soil-pore spaces, imperfections in waterproofing membranes, and voids in concrete, eventually penetrating into the lower levels of the project. Therefore, the following rules for surface / subsurface drainage construction must be applied:

- Surface and subsurface drainage systems must always be isolated from each other. Surface water can pollute subdrains with sediment and provide oxygenated water that can promote the growth of bacteria and related caking / clogging with bacterial slime. Regarding the latter, hand-in-hand with sedimentation, iron-reducing bacteria are highly effective at clogging subdrains with biological and chemical encrusting material and should not be underestimated.
- All drains have limits controlled by:
    - The physical size of the drain / area open to receive discharge affecting permeability (ability of the pipe to discharge water);
    - Length, slope, and outlet characteristics;
    - Medium through which water is moving;
    - Maintenance / damage / durability.

If any of these components are out of specification, drainage will be impaired.

- Excessive flow can cause backup reemphasizing the need for appropriate design criteria; criteria that must be followed during construction.
- No matter how well designed, drainage efficiency degrades with time due to:
    - siltation;
    - filter caking;
    - biological clogging;
    - lack of redundancy;
    - gaps in maintenance.
- Cleanouts and inspection provisions are needed to inspect and maintain subdrainage systems;
- Means of reducing / limiting influx of sediment help to reduce sediment clogging;
- Tree roots and roots from other vegetation are an enemy and will always find ways into drains in search of moisture;
- Any trees / vegetation near drains must be carefully isolated such that roots stay out of drains;
- Because the risks of root clogging, siltation, biological clogging, and other nuisances are high, and drains are critical and inaccessible following construction, inspection / cleanout ports and risers are necessary to provide long-term serviceability and maintenance.
- Following the established Unified Plumbing Code guidelines, all drainage piping should use appropriate Drain-Waste-Vent (DWV) sanitary sweep inspection cleanouts at all T-connectors, right-bends, and water traps.
- Drain outlets where low flow gradients exist, must be carefully designed to prevent backflow into drains during high flow events in discharge pipes (i.e. storm drains).
- All connections to discharge outlets, such as storm drains, are recommended to be connected at or above the spring line such that high flows within the outlet pipe will not enter or surcharge subdrains and debris entering the storm drain will not damage the inlet.
- In the case of basements / subterranean structures, wall drains must be kept separate / isolated from floor drains and other drains of lower elevation to prevent hydrostatic and/or hydrodynamic overloading of lower drains.
- All completed drain systems must be double-checked and tested prior to and following backfilling.

- All completed drain systems must be protected during backfilling and subsequent construction activities and verified functional prior to putting into service.
- All drainage devices should have a long-term maintenance component assigned to them including:
    - Periodic inspection for functionality;
    - Periodic flushing, cleanout, and root removal.

<u>Application of High Strength / Low Permeability Foundation Concrete</u>

From a geotechnical standpoint, foundation concrete serves two purposes at the project site:

1. Supporting the overlying structure to be built;
2. Protecting the interior sublevels against retaining wall and mat slab moisture / vapor intrusion.

High strength / low permeability foundation concrete will be the primary defense against moisture / vapor intrusion. However, this component is subject to more pitfalls than any of the other waterproofing elements combined. This is because concrete design and construction follows the old adage, “measure with a micrometer, mark it with chalk, and cut with an axe”. In other words, concrete as a building material is inherently variable in terms of its composition and application due to a number of factors including:

- mix design;
- workability;
- field conditions;
- dry vs. moist curing;
- cure time.

The following “good words” are provided to emphasize these factors and provide justification regarding the recommendations put forth by our firm and the structural engineer.

Mix design refers primarily to the relative proportions of materials contained with a given mix of concrete. These consist of:

- Aggregrate;
- Portland Cement;
- Water;
- Admixtures.

Aggregate refers to rock added to the concrete mix to provide interlocking structure, strength, and resistance to abrasion. Portland cement alone is relatively weak and easily abraded which is why proper finishing is necessary in order for the aggregate to consolidate properly and maintain an even distribution throughout the mix. Overworking of the concrete during finishing and folding of bleed water can cause the aggregate to settle forming a layer of Portland cement "fat" at the surface that will abrade easily. Therefore, finishing must be performed according to the guidelines of the Portland Cement Association and American Concrete Institute.

Water-to-cement ratio (W/C) has a profound effect on and is the primary control of strength and permeability as water reacts chemically with Portland cement to form concrete. If the stoichiometry of this reaction contains excess water, that water will form blebs within the concrete that become void spaces capable of transmitting moisture / vapor and reducing the strength of the concrete. It is for this reason that low W/C ratios are preferred where low permeability concrete is needed to mitigate transmission of moisture / vapor.

Workability refers to the ease of which a given concrete pour is able to be molded, leveled, and surfaced before it has time to set. In order to adhere to recommendations put forward regarding W/C ratio, admixtures such as plasticizers can be applied to increase workability without the addition of water.

Field conditions include temperature, humidity, and subgrade texture. Temperature and humidity are key factors that control the rate of shrinkage as the concrete sets up and cures. For example, warm / dry periods such as during Santa Ana Wind events are poor times to pour concrete as the outer surface of the concrete will dry and cure faster than the interior. This condition leads to the formation of micro-cracks that can link up and create pathways for water and vapor transmission through the mat slab and retaining wall structures. If concrete must be poured on such days, care must be taken to protect the surface of the concrete. Additionally, subgrade texture can impact the rate of curing if bleed water is not able to escape from the inner surface of the concrete. In the case of the planned construction of the mat slab foundation at the project site, concrete is to be poured directly on an impermeable membrane / vapor barrier (e.g. W.R. Grace Product) that will prevent the transmission of concrete bleed-water into the subgrade. This can negatively impact the curing of the concrete and can only be minimized by adhering to the W/C ratio recommendations.

Moist curing also has a profound effect on the strength and permeability of concrete. This practice involves maintaining moisture at the surface of the concrete to prevent water available for reaction with Portland cement from escaping. Moist curing allows concrete strength to continue developing long after concrete cured in dry air resulting in strength increases on the order of 100% after 28 days of curing. This practice is also ideal for mitigating unfavorable field conditions (e.g. Santa Ana Wind events).

The W/C ratio recommendations are critical in terms of maximizing strength and minimizing shrinkage, cracking, and permeability. For example, concrete poured with a W/C of 0.4 has higher two-day strength than the ultimate strength of the same concrete poured with a W/C of 0.7 after 27 days. Similar trends also exist in terms of permeability with permeability increasing dramatically as a result of higher W/C because excess trapped water not used in the chemical reactions that give concrete its character result in higher porosities. Moist curing and sufficient curing time also needs to be carried out to completion in order to achieve the desired results as a failure to do so will result in additional shrinkage and cracking that can provide pathways for water migration through the mat slab and reduce its overall strength. Therefore, we provide the following recommendations to achieve the maximum strength and minimum permeability of the mat slab foundation and retaining wall structures. These recommendations are GEOTECHNICAL / WATERPROOFING requirements are to be integrated with structural engineering requirements:

- W/C and all other design parameters and ratios are to remain unchanged from the recommended design mix unless specifically allowed for in writing by the engineer of record. This means that NO water is to be added to the mix once it is certified by the concrete plant. If workability proves to be an issue, we recommend that an appropriate fluidizer plasticizer be used along with appropriate equipment / worker skills to handle this material.
- Concrete trucks and pumps are NOT to be washed out into any structural / foundation area in between pours. This is to prevent uncontrolled addition of water to the concrete mix and limit any variability in the concrete as-placed.
- All washout activities are to be performed AWAY from structural / foundation areas into approved holding bins specific to this purpose.
- All cold joints will need to be sealed with appropriate water-stops, subject to review by our firm. Water-stops deemed inappropriate will be rejected.
- Old concrete does not bond readily with new concrete. Therefore, an appropriate bonding agent and, possibly, dowelling will be necessary in order to seal cold joints.
- All concrete must be finished according to the accepted standards of practice put forth by the Portland Cement Association and American Concrete Institute.
- All concrete poured as part of the mat slab foundations and retaining wall systems is subject to geotechnical review by this office.
- If concrete poured does not meet the geotechnical criteria provided in this report, it will be rejected.
- All concrete pours are subject to full-time observation by a representative of our firm.

Soil Expansion

The near surface site soils exhibit potential medium soil expansion potential and this soil expansion should be included in the design and construction of at-grade construction in contact with site soils. The deeper-seated native soils consist predominantly of silt sands and sandy silts, locally containing some clay fraction. For preliminary design purposes, structure elements in contact with these deeper-seated soils should be conservatively designed considering potential low soil expansion potential. The soil expansion potential should be confirmed by laboratory testing based on the soil types (on-site and/or import materials) placed at the time of construction.

Soil Corrosivity

Concrete

The results of preliminary soil soluble sulfate tests indicate that the site soils exhibit negligible

(Not Applicable, S0) sulfate attack exposure to concrete. However, it has been our experience that post-construction factors such as the use of fertilizers in lawn / landscape areas, near surface soil wetting and drying cycles, and other changes with time can increase the concentrations of soluble sulfate and other derogatory salts and these conditions predispose them to being highly corrosive to both concrete and buried metals. Higher strength concrete with lower water / cement ratio will improve overall slab performance, durability, and water and corrosivity resistance.

It is therefore recommended that concrete in contact with soils be designed to resist potentially high sulfate exposure (i.e., Type V Portland Cement, minimum compressive strength of 4,500 psi, and maximum w:c = 0.45).

As an added measure to mitigate potential water / moisture vapor transmission into subterranean (inset) level construction, we recommend incorporating a hydrophobic admixture (Hycrete W1000, Xypex, or architect / structural engineer approved equivalent) in the concrete. This hydrophobic mixture should be considered for both the basement floor slab and subterranean structure walls.

Metallic Installations

Laboratory tests to evaluate the potential soil corrosivity to metallic installations were not performed. In the absence of such testing, the soils along with any transient waters flowing through them should be considered to be highly corrosive to metals in contact with them. Attention to minimizing galvanic / chemical corrosivity (i.e., protective coatings, dielectric couplings, eliminating mixing metal types in contact or in near vicinity to each other) where in contact with soil and soil moisture can minimize these effects. Soils in contact with concrete or metallic structural elements should be tested at the time of construction to verify corrosion potential. An experienced corrosion consultant should be retained and their recommendations incorporated into the design if special / critical corrosive issues exist or further corrosion potential study is warranted.

Utility Trench Backfill

Utility lines will be located within the project building envelope as well as outside and within the street right of way for tie-ins. For utility trenches located outside the structure footprint, the on-site soils are considered suitable for trench backfill, provided they are free of organic material and rocks over 4 inches in maximum dimension. Bedding material should consist of on-site sandy or imported materials exhibiting a Sand Equivalent (S.E.) value of 30 or greater. Trench backfill should be placed in thin lifts, ideally not exceeding 4 inches and mechanically compacted to achieve a relative compaction of at least 90% of the maximum dry density as determined by ASTM: D1557. Care should be taken not to damage utility lines.

Regardless of utility line location, adequate drainage within the utility trench backfill needs to be considered.

Trenches in fill soils and terrace deposits greater than 4 feet in depth should be shored or sloped back as required by the local regulatory agency, the State of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements. Trenches within rock are expected to have higher stability than those in soil, however, these excavations should be evaluated by the Project Geotechnical Engineer on a case by case basis with regard to stability, preferably as the excavation work is performed, should localized unfavorable conditions be exposed.

<u>Site Drainage</u>

It should be noted that potential problems may develop when drainage is altered through construction of retaining walls, paved walkways, and patios. Conditions which will lead to ground saturation must be avoided:

- All roof and surface drainage should be directed away from structures and their appurtenances to approved drainage facilities. Ponding of water should be avoided. For graded soil areas, a minimum gradient of 2 percent away from structures should be maintained.
- The recommended drainage patterns should be established at the time of fine grading and maintained throughout the life of the structure or, if altered, should be replaced with a properly designed area drain system.
- Irrigation activities at the site should be monitored and controlled to prevent over-watering. Planter areas adjacent to structures should be avoided. If utilized, such planters should include measures to contain irrigation water and prevent moisture migration into walls and under foundations and slabs-on-grade.

Site drainage should also be designed, constructed, and maintained in accordance with appropriate City, County, State, and other jurisdictional requirements.

Landscape, Irrigation, and Maintenance

General guidelines for landscape, irrigation and maintenance are shown below:

1) Landscape and slope planting should consist of appropriate native, drought resistant vegetation as recommended by the Landscape Architect. Landscaping of slopes should be completed as soon as possible and properly maintained.
2) Proper irrigation, maintenance, and repair of installed irrigation systems are essential. Leaks should be repaired immediately. Sprinklers should be adjusted to provide maximum coverage with a minimum of water usage and overlap. Over-watering with consequent excessive runoff and ground saturation must be avoided.
3) If automatic sprinkler systems are installed, their use must be adjusted to account for natural rainfall conditions.
4) Irrigation activities at the site should be monitored and controlled to prevent over-watering. Planter areas adjacent to structures should be avoided. If utilized, such

planters should include measures to drain or contain irrigation water and prevent moisture migration into walls and under foundations.

5) All interceptor ditches, drainage terraces, down-drains, and any other drainage devices that have been installed must be maintained and cleaned.
6) The property owner must undertake a program for the elimination of burrowing animals. This must be an ongoing program in order to promote slope stability.
7) Water must not be allowed to flow over the natural slope. All drainage should be directed away from the slope to appropriate non-erosive drainage devices.

Plan Review, Observations, and Testing

There are numerous geotechnical and engineering geologic conditions, phenomena, and issues present that will have considerable influence on the design, construction, and long-term performance of the proposed development. Therefore, it is considered of high importance and prudence that this firm be retained throughout the design and construction process to provide appropriate geotechnical and geologic support, input, review, and documentation services to assist the design and construction team with accounting for these issues appropriately. It is critical that the geotechnical and engineering geologic recommendations be properly taken into account and understood by the parties involved, and the intent of the recommendations properly incorporated into the final design, construction, and long-term maintenance of the project.

G3SoilWorks, Inc. should be provided with final grading, foundation and shoring plans, site and subdrainage plans, and design loads, when available, for review to evaluate the acceptability of the preliminary recommendations presented herein and to develop additional / revised recommendations, as appropriate.

All excavation and grading operations must be performed under the continuous observation and testing by a representative of this firm to verify conformance of the earthwork to the requirements of the regulating agencies, the project specifications, and the recommendations presented in this report. Any earthwork performed in connection with the subject project without the full knowledge of, and not under the direct observation of this firm, will render the recommendations of this report invalid.

A representative of this office should be present on-site to observe and document the following:

- Removals of existing foundations and utilities;
- Site excavations and remedial grading;
- Conventional and deep foundation excavations;
- Stabilization fill slope construction;
- Installation of wall backdrains, subdrains, and underdrains;
- Fill placement and compaction, including utility trench backfills; and
- Slab and sub-slab (i.e. vapor retarder and underdrainage) preparation.

LIMITATIONS

This report has been prepared for the exclusive use of the Greens Group, Inc. and their design consultants relative to the design and construction of the proposed residence. This report is not intended for other parties, and it may not contain sufficient information for other purposes. This report and the recommendations confirmed herein are made with the understanding that G3SoilWorks will be appropriately retained to assist with the design and construction team in proper interpretation, incorporation, and implementation of the intent of our report recommendations. Should a different firm be retained to perform the subsequent phases of design and construction, this report will be considered null and void.

The Owner or their representative should make sure that the information and preliminary recommendations presented in this report are brought to the attention of the Project Design Team and incorporated into the project plans.

This office should be provided with final grading and foundation plans for review to enable us to confirm the preliminary recommendations and update the report as necessary.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from limited borings and the results of the laboratory testing and engineering analysis. The opinions and recommendations provided were based on the assumption of the geotechnical conditions, which exist across the site, are similar to those observed in the test excavations. The conditions and characteristics of the sub-surface materials at locations and depth other than those excavated and observed may be different and no representations are made as to their quality and engineering properties. Based on our experience with similar sites, some variability and unanticipated conditions may be present, and some degree of "as-grading" is anticipated to be warranted to appropriately address these conditions and to meet the intent of the recommendations presented herein. As such, many of the overexcavation, embedment, and replacement issues - based on actual exposed conditions - may be at odds with the generalized considerations made herein. These issues and conditions should be appropriately evaluated / addressed by this firm on a case-by-case basis at the time the work is performed - and the resulting recommendations and refinements reported in a final as-graded report documenting the geotechnical aspects of the project work.

Should any conditions encountered during construction differ from those described herein, this office should be contacted immediately for evaluation of the actual conditions and for appropriate recommendations prior to continuation of work.

CLOSURE

The findings and recommendations presented herein were developed in accordance with generally accepted professional engineering principles and local practice in the field of geologic and geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

Respectfully submitted,

G3SoilWorks, Inc.

By: ____________________________
Daniel J. Morikawa, P.E., G.E.
Director of Engineering

By: ____________________________
Erik C. Haaker, P.G., C.E.G.
Project Engineering Geologist

Reviewed By: ____________________________
Steven E. Strickler, P.E., G.E
C.E.O.

LIST OF SELECTED REFERENCES

1) AXIS / GFA Architecture + Design, AC Marriott Residence Inn Dual Brand, 3466 Mission Inn Avenue, Riverside, California 92501, Sheets G0.00-G0.02, A1.01, A2.B1-A2.B4, A2.01-A2.04, A4.01-A4.06, and A5.01-A5.04, Entitlement Set for Greens Development, dated July 22, 2019.

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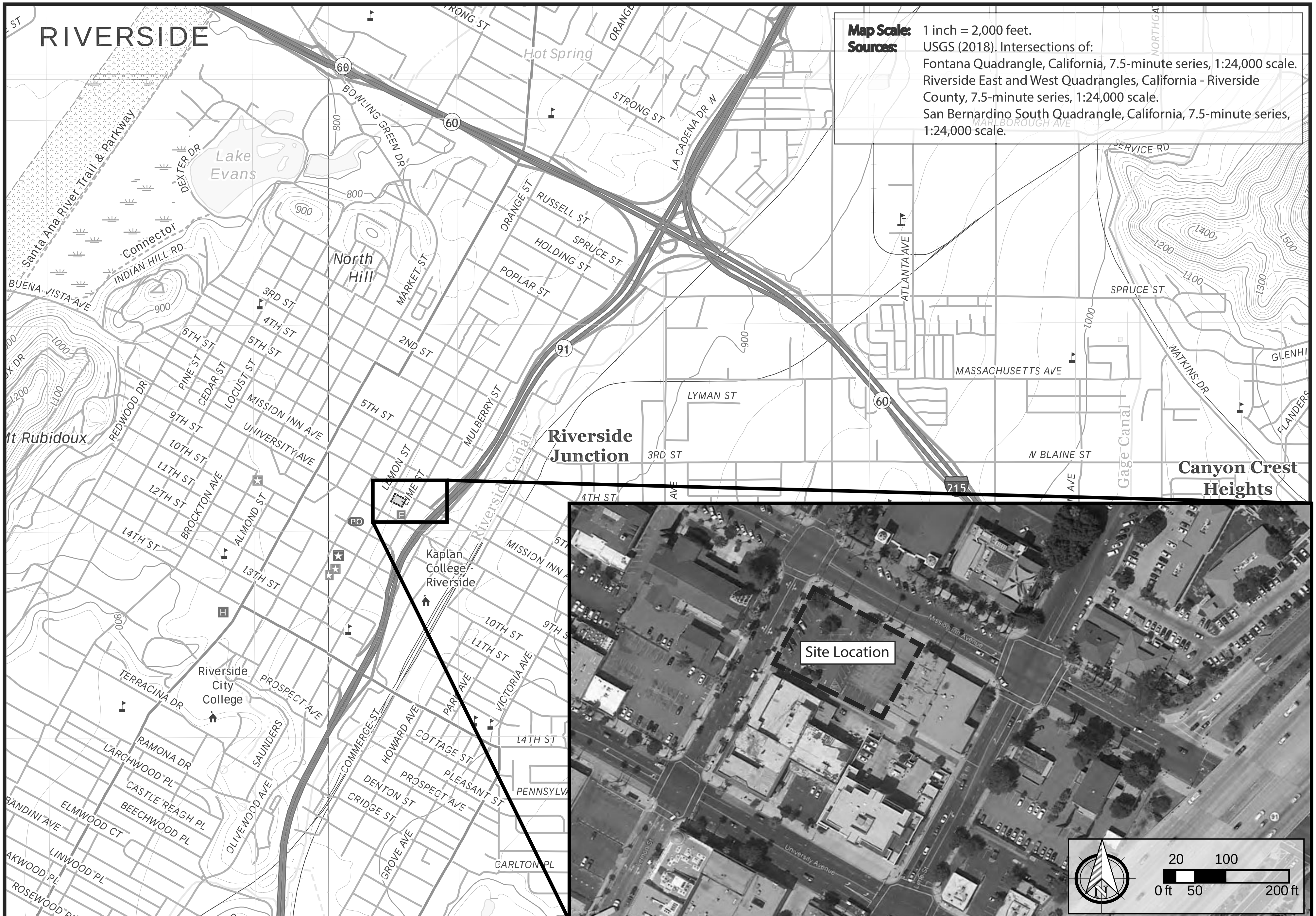
3) Dibblee, T.W., and Minch, J.A., 2003, Geologic map of the Riverside East/south 1/2 of San Bernardino South quadrangles, San Bernardino and Riverside County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-109, scale 1:24,000.

4) Dibblee, T.W., and Minch, J.A., 2004, Geologic map of the Riverside West/south 1/2 of Fontana quadrangles, San Bernardino and Riverside County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-128, scale 1:24,000.

5) Rockwell, T.K., A.J. Meltzner, and E.C. Haaker, (2018), Dates of the Two Most Recent Surface Ruptures on the Southernmost San Andreas Fault Recalculated by Precise Dating of Lake Cahuilla Dry Periods, Bulletin of the Seismological Society of America, Vol. 108, No. 5A, pp. 2634-2649, dated October 1, 2018.



Overview Map	Proj. No. 1-1167-A	Mar. 2020
3644 Mission Inn Ave.	Riverside, CA	Figure 1

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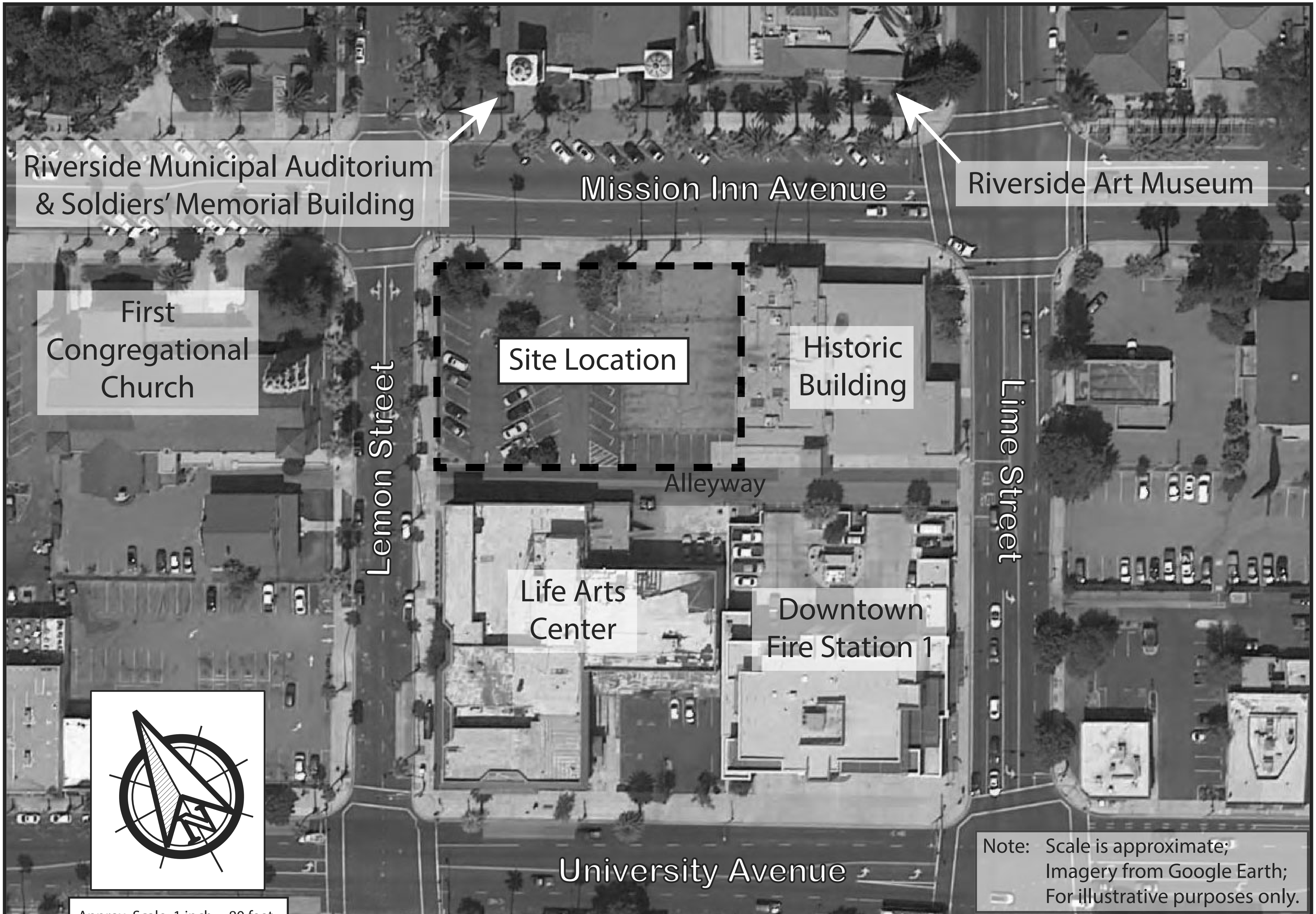

Site Vicinity Map	Proj. No. 1-1167-A	Mar. 2020	G3SoilWorks
3466 Mission Inn Ave.	Riverside, CA	Figure 2	

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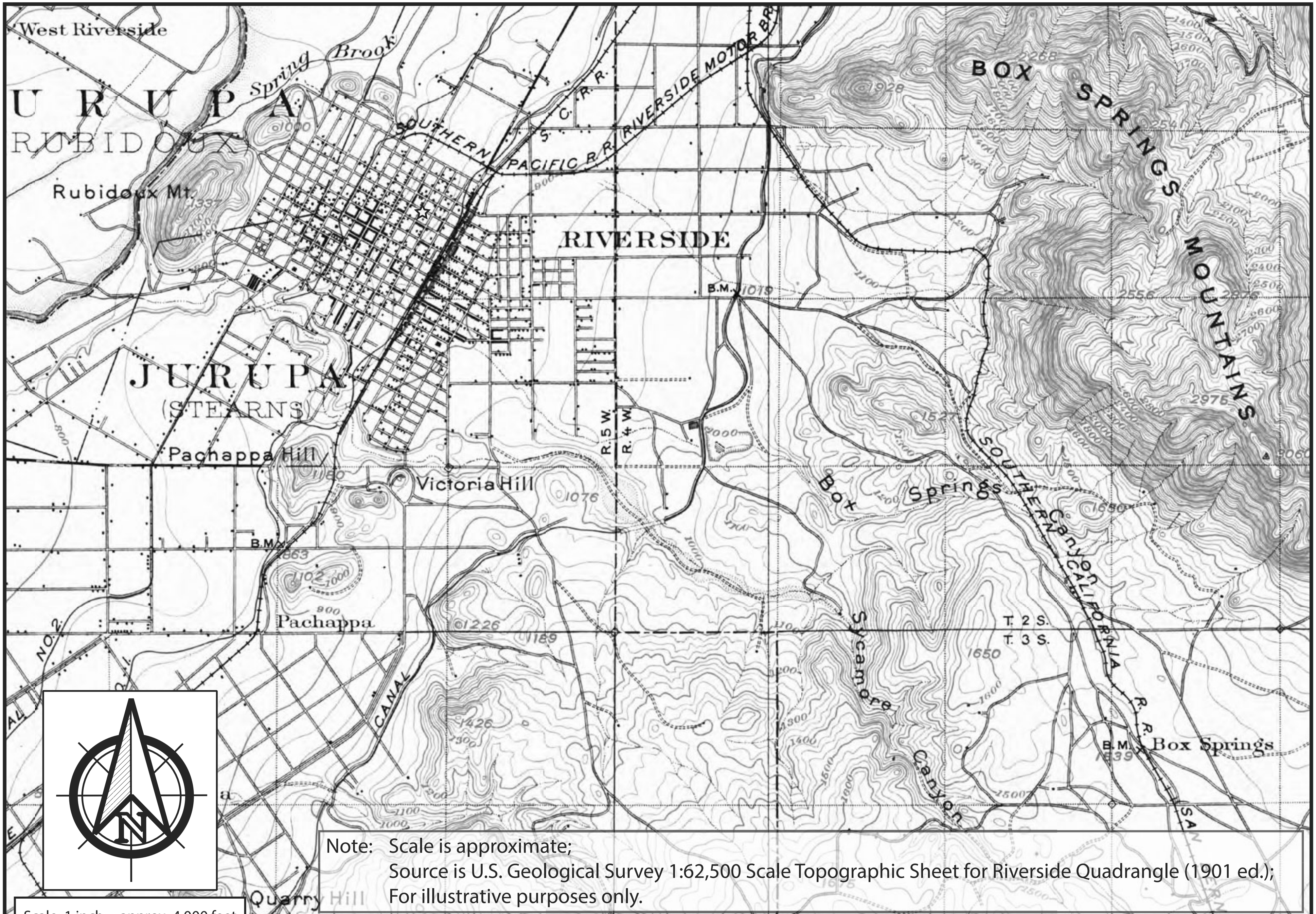

Note: Scale is approximate;
Source is U.S. Geological Survey 1:62,500 Scale Topographic Sheet for Riverside Quadrangle (1901 ed.);
For illustrative purposes only.

Historic Topographic Map	Proj. No. 1-1167	Mar. 2020
3466 Mission Inn Ave.	Riverside, CA	Figure 5

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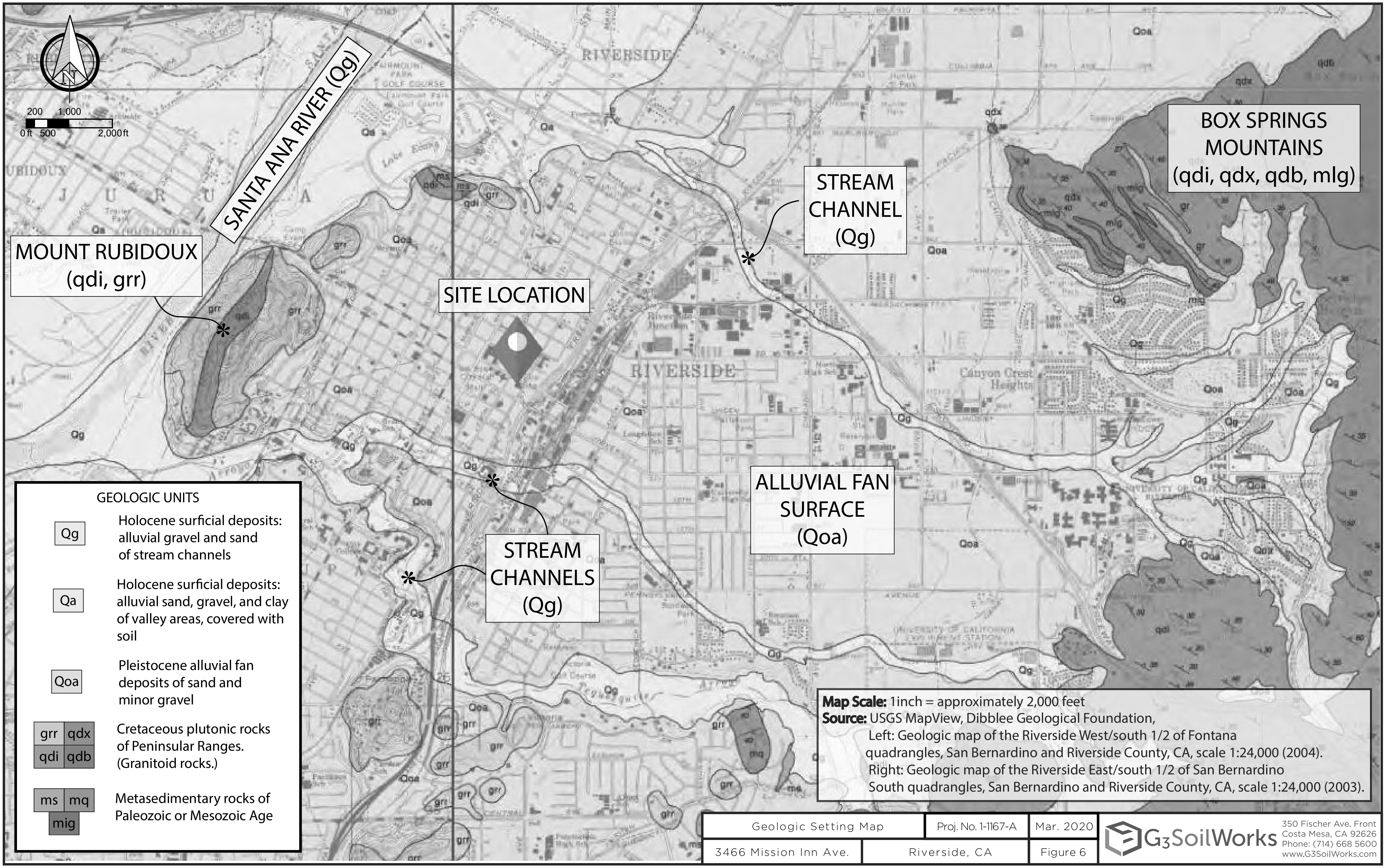
SANTA ANA RIVER (Qg)
MOUNT RUBIDOUX (qdi, grr)
SITE LOCATION
STREAM CHANNEL (Qg)
STREAM CHANNELS (Qg)
ALLUVIAL FAN SURFACE (Qoa)
BOX SPRINGS MOUNTAINS (qdi, qdx, qdb, mlg)
RIVERSIDE
Canyon Crest Heights
UNIVERSITY OF CALIFORNIA
200 1,000
0 ft 500 2,000 ft
GEOLOGIC UNITS
Qg Holocene surficial deposits: alluvial gravel and sand of stream channels
Qa Holocene surficial deposits: alluvial sand, gravel, and clay of valley areas, covered with soil
Qoa Pleistocene alluvial fan deposits of sand and minor gravel
grr qdx qdi qdb Cretaceous plutonic rocks of Peninsular Ranges. (Granitoid rocks.)
ms mq mig Metasedimentary rocks of Paleozoic or Mesozoic Age
Map Scale: 1inch = approximately 2,000 feet
Source: USGS MapView, Dibblee Geological Foundation,
Left: Geologic map of the Riverside West/south 1/2 of Fontana quadrangles, San Bernardino and Riverside County, CA, scale 1:24,000 (2004).
Right: Geologic map of the Riverside East/south 1/2 of San Bernardino South quadrangles, San Bernardino and Riverside County, CA, scale 1:24,000 (2003).
Geologic Setting Map
Proj. No. 1-1167-A
Mar. 2020
3466 Mission Inn Ave.
Riverside, CA
Figure 6
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Boring Location Map		Proj. No. 1-1167-A	Mar. 2020
3466 Mission Inn Ave.	Riverside, CA		Figure 7

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APPENDIX A

Geotechnical Boring Logs

GEOTECHNICAL BORING LOG

PROJECT NO.	1-1167-A	PROJECT NAME	Marriott Riverside		
DATE STARTED	2/18/20	GROUND ELEV.(FT)	861	BORING DESIG.	B-1
DATE FINISHED	2/18/20	GW DEPTH (FT)		LOGGED BY	ECH
DRILLER	Oneway Drilling	DRIVE WT.	140 lb.	NOTE	
TYPE OF DRILL RIG	Hollow Stem	DROP	30 in.		

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
				ML	**Artificial fill undocumented (afu).** @ 0-5': Sandy SILT with Clay, dark yellowish-brown (10 YR 4/4), moist, low apparent plasticity, fine micaceous sand.			
5	856	R	25	ML	**Quaternary Old Alluvium (Qoa).** @ 5': Sandy SILT, dark yellowish-brown (10 YR 4/4), slightly moist to moist, stiff to very stiff, low to very low apparent plasticity, fine micaceous sand.	14.0	111.8	
10	851	SPT	21	ML	@ 10': Sandy SILT, dark yellowish-brown (10 YR 4/4), slightly moist to moist, very stiff, low to very low apparent plasticity, fine micaceous sand.			
15	846	R	52	SM/SC	@ 15': Fine to medium grained Silty SAND / Clayey SAND, brown (10YR 4/3), moist, dense to very dense.	13.0	119.3	
20	841	SPT	24	SM/ML	@20': Silty SAND / Sandy SILT, brown to dark yellowish-brown (10 YR 4/3-4/4), slightly moist, medium dense / very stiff, fine micaceous sand.			
25	836	R	44	SM/ML	@ 25': Silty SAND / Sandy SILT, brown (10 YR 4/3), moist, medium dense/stiff to very stiff, low to very low apparent plasticity, fine micaceous sand.	19.3	108.1	
30	831	SPT	18	SM/ML	@ 30': Silty SAND / Sandy SILT, brown (10YR 4/3), moist, medium dense / very stiff, fine micaceous sand.			
35	826	R	72	SP	@ 35': Poorly graded SAND, fine- to medium-grained with some coarse sand, slightly moist, dense, subrounded to subangular, quartz-feldspar rich.	2.4	128.2	

SAMPLE TYPES:
- R RING (DRIVE) SAMPLE
- S SPT (SPLIT SPOON) SAMPLE
- B BULK SAMPLE

Water Seepage
Groundwater
DS - Direct Shear
GS - Grain Size Analysis
EI - Expansion Index
CONS - Consolidation

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PN: 1-1167-A REPORT DATE: 03/31/20

GEOTECHNICAL BORING LOG

PROJECT NO.	1-1167-A	PROJECT NAME	Marriott Riverside		
DATE STARTED	2/18/20	GROUND ELEV.(FT)	861	BORING DESIG.	B-1
DATE FINISHED	2/18/20	GW DEPTH (FT)		LOGGED BY	ECH
DRILLER	Oneway Drilling	DRIVE WT.	140 lb.	NOTE	
TYPE OF DRILL RIG	Hollow Stem	DROP	30 in.		

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
		SPT	38	SM SP	@ 40': Upper 2/3 Silty SAND, brown (7.5 YR 4/3), moist, dense, fine micaceous sand; Lower 1/3 Poorly-graded SAND, pale brown, slightly moist, dense, some mica, quartz and feldspar rich.			
45	816	R	43	SP CL	@ 45': Upper half poorly-graded SAND, slightly moist, medium dense, fine micaceous sand, fine- to medium-grained. Lower half Silty CLAY, brown (7.5 YR 4/3-4/4), moist to very moist, very stiff, low to medium apparent plasticity, fine micaceous sand.	10.6	120.7	
50	811	SPT	24	ML	@ 50': Sandy SILT, dark yellowish-brown (10 YR 4/4), moist to very moist, very stiff, low apparent plasticity, fine micaceous sand. Total Depth: 51.5 feet. Depth of artificial fill is approximate. No groundwater encountered. Hole backfilled with cuttings.			

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE

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Groundwater
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PN: 1-1167-A REPORT DATE: 03/31/20

GEOTECHNICAL BORING LOG

PROJECT NO.	1-1167-A	PROJECT NAME	Marriott Riverside		
DATE STARTED	2/18/20	GROUND ELEV.(FT)	861	BORING DESIG.	B-2
DATE FINISHED	2/18/20	GW DEPTH (FT)		LOGGED BY	ECH
DRILLER	Oneway Drilling	DRIVE WT.	140 lb.	NOTE	
TYPE OF DRILL RIG	Hollow Stem	DROP	30 in.		

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
		BULK			**Artificial fill undocumented (afu)** @ 0-2.5': Sandy SILT with Clay, dark yellowish-brown (10 YR 4/4), moist, low apparent plasticity, fine micaceous sand.			
		R	17	ML	@ 2.5': Sandy SILT, dark yellowish-brown (10 YR 4/4), moist, stiff, low apparent plasticity, micaceous.	10.5	105.4	
5	856	SPT	11	ML	**Quaternary Old Alluvium (Qoa).** @ 5': Sandy SILT, brown to yellowish-brown (10 YR 4/3-4/4), moist, stiff, low to very low apparent plasticity, with micaceous fine sand.			
		R	72	SM/ML	@ 7.5': Silty SAND / Sandy SILT with trace clay, dark yellowish-brown (10 YR 4/3-4/4), moist, dense, very fine micaceous swand	9.8	113.5	
10	851	SPT	30	ML	@ 10': Sandy SILT with Clay, dark yellowish-brown (10 YR 5/6), moist, very stiff, very low apparent plasticity.			
		R	38	SM	@ 12.5': Silty SAND, brown to dark yellowish-brown (7.5YR 4/4 - 10YR 5/6) slightly moist to moist, dense, fine-grained, micaceous, calcareous.	5.9	112.9	
15	846	SPT	29	SM/ML	@ 15': Silty SAND / Sandy SILT with Clay, brown (7.5 YR 4/3-4/4), slightly moist, medium dense / very stiff, very low apparent plasticity, fine moderate rootlets less than 1 mm.			
		R	34	SM	@ 17.5': Silty SAND, brown to strong brown (7.5 YR 4/4-4/6), slightly moist to moist, medium dense, fine-grained.	6.7	108.2	
20	841	SPT	14	ML	@ 20': Sandy SILT, brown (7.5 YR 4/3-4/4), moist, stiff, micaceous fine sand.			
		R	58	SM	@ 22.5': Silty SAND, brown (7.5 YR 4/3-4/4), moist, dense, micaceous, fine-grained.	21.1	104.8	
25	836	SPT	16	ML	@ 25': Clayey SILT, dark yellowish-brown (10 YR 4/4-4/6), moist, stiff to very stiff, low apparent plasticity, fine micaceous sand.			
		R	47	SW/SM	@ 27.5': Well-graded SAND / Silty SAND, brown (7.5 YR 4/3), moist to very moist, dense, fine- to coarse-grained, subrounded, micaceous.	8.5	115.9	
30	831	SPT	21	SP	@ 30': Poorly-graded SAND with Silt, slightly dark yellowish-brown, slightly moist to moist, medium dense, fine- to medium-grained, subangular, quartz and feldspar rich with abundant biotite.			
		R	46	SP	@ 32.5': Poorly-graded SAND with Silt, yellowish-brown to light yellowish-brown (10 YR 5/4-6/4), slightly moist, dense, subangular to subrounded, sparse mica, arkosic / feldspar rich.	3.0	114.0	
35	826	SPT	20	SM/ML	@ 35': Silty SAND / Sandy SILT with Poorly-Graded SAND, brown (7.5 YR 4/4-4/4), moist to very moist, medium dense, fine- to medium-grained sand. Upper 6" = Sandy SILT, mid 6" = Poorly-graded SAND, lower 6" = Silty SAND.			
		R	35	SM	@ 37.5': Silty SAND with Clay, brown (7.5 YR 4/3-4/4), very moist, dense, fine- to medium-grained, micaceous.	0.6	129.4	

SAMPLE TYPES:
- R RING (DRIVE) SAMPLE
- S SPT (SPLIT SPOON) SAMPLE
- B BULK SAMPLE

▽ Water Seepage
▼ Groundwater
DS - Direct Shear
GS - Grain Size Analysis
EI - Expansion Index
CONS - Consolidation

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PN: 1-1167-A REPORT DATE: 03/31/20

GEOTECHNICAL BORING LOG

PROJECT NO.	1-1167-A	PROJECT NAME	Marriott Riverside		
DATE STARTED	2/18/20	GROUND ELEV.(FT)	861	BORING DESIG.	B-2
DATE FINISHED	2/18/20	GW DEPTH (FT)		LOGGED BY	ECH
DRILLER	Oneway Drilling	DRIVE WT.	140 lb.	NOTE	
TYPE OF DRILL RIG	Hollow Stem	DROP	30 in.		

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
		SPT	31	SM/ML	@ 40': Silty SAND / Sandy SILT, brown (7.5 YR 4/3-4/4), moist, dense / very stiff, fine-grained, micaceous.			
		R	78	SP	@ 42.5': Poorly-graded SAND, pale brown (10 YR 7/3), slightly moist, very dense, medium- to coarse-grained, subrounded to subangular, quartz and feldspar rich.			
45	816	SPT	26	SM/SC	@ 45': Silty SAND / Clayey SAND interbeds, moist to very moist, medium dense, fine- to medium-grained, micaceous, slightly sticky / apparently plastic where clayey.			
		R	72	SC	@ 47.5': Clayey SAND, brown (7.5 YR 4/3-4/4), moist to very moist, dense, fine-grained, apparent low plasticity in fines, micaceous.	15.6	113.7	
50	811	SPT	15	ML/CL	@ 50': Sandy SILT with Clay, brown (7.5 YR 4/3-4/4), moist to very moist, stiff, low apparent plasticity, micaceous fine sand.			
		R	58	ML/CL	@ 55': Sandy SILT / Sandy CLAY, brown to yellowish-brown (10 YR 5/3-5/4), hard / indurated, slightly mottled, micaceous fine sand. Note: drilling became hard at 50 feet - added water.	13.8	115.0	
		SPT	39	ML/SM	@ 60': Interbedded Sandy SILT / Silty SAND, brown (7.5 YR 4/3-4/4), moist to very moist, hard / dense, low apparent plasticity, fine micaceous sand, predominantly silt.			
		R	76	SM/CL	@ 65': Silty SAND, brown to strong brown (7.5 YR 4/3-4/6), very moist, very dense, fine-grained, highly micaceous. Tip: Silty CLAY, brown to strong brown (7.5 YR 4/3-4/6), very moist, low to medium apparent plasticity, very fine mica, thin lamination of fine sand / silt.	19.8	107.3	
		SPT	20	CL	@ 70': Grades between Silty CLAY and Sandy CLAY, brown (7.5 YR 4/4), very moist, very stiff, low to medium apparent plasticity, fine- to medium-grained sand, horizon of silty sand.			
		R	79	SM	@ 75': Silty SAND with Clay, brown to strong brown (7.5 YR 4/4-4/6), moist, very dense, well-graded, fine- to medium-grained with some coarse sand, rich with quartz, feldspar, and biotite. Total Depth: 76.5 feet. Depth of artificial fill is approximate. No groundwater encountered. Hole backfilled with cuttings.	6.6	117.8	

SAMPLE TYPES:
- R RING (DRIVE) SAMPLE
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GEOTECHNICAL BORING LOG

PROJECT NO. 1-1167-A
DATE STARTED 2/18/20
DATE FINISHED 2/18/20
DRILLER Oneway Drilling
TYPE OF DRILL RIG Hollow Stem

PROJECT NAME Marriott Riverside
GROUND ELEV.(FT) 862
GW DEPTH (FT)
DRIVE WT. 140 lb.
DROP 30 in.

BORING DESIG. B-3
LOGGED BY ECH
NOTE

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
		BAG		CL	**Artificial Fill Undocumented (afu)** @ 0-3': Silty CLAY, very dark gray, moist to very moist, low to medium plasticity, brick inclusions.			
				ML	@ 3-5': Sandy SILT with Clay, dark yellowish-brown (10 YR 4/4), moist, low apparent plasticity, fine micaceous sand.			
5	857	R	31	ML	**Quaternary Old Alluvium (Qoa)** @ 5': Sandy SILT, olive brown to brown (2.5 Y - 10 YR 4/3), moist, medium dense/very stiff, very low apparent plasticity to non-plastic silt w/ fine micaceous sand.	13.4	109.8	
10	852	SPT	19	ML	@ 10': Sandy SILT, brown to yellowish-brown (10 YR 5/4), slightly moist, stiff, fine micaceous sand, apparent non plastic silt fines.			
15	847	R	56	ML	@ 15': Sandy SILT interbedded w/ Silty SAND, dark grayish-brown to olive brown (2.5 Y 4/2-4/3), moist, hard, low plasticity, fine- to medium- grained, micaceous.	14.0	113.3	
				SM	@ 16.5: Tip is Silty SAND, dark grayish-brown to olive brown (2.5 Y 4/2-4/3), moist, hard / dense, fine- to medium- grained, micaceous.			
20	842	SPT	11	SM	@ 20': Silty SAND, brown (10 YR 4/3), slightly moist, medium dense, fine-grained.			
25	837	R	49	SP	@ 25': Poorly-graded SAND with Silt, brown (10 YR 4/3), slightly moist, dense, fine-grained, micaceous, quartz rich.	4.4	108.6	
30	832	SPT	32	SM	@ 30': Silty SAND, brown to dark yellowish-brown (10 YR 4/3-4/4), moist, dense, fine-grained, micaceous.			
				SP	@ 31.5': Tip is poorly-graded SAND, brown to dark yellowish-brown (10 YR 4/3-4/4), slightly moist, dense, fine- to medium-grained, subrounded to subangular, quartz and feldspar present.			
35	827	R	35	SP	@ 35': Poorly-graded SAND, very slightly moist, medium dense, fine- to medium-grained, subrounded to subangular, quartz and feldspar present. Note: soil fell apart in rings.	5.7	109.8	

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
S SPT (SPLIT SPOON) SAMPLE
B BULK SAMPLE

Water Seepage
Groundwater
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PN: 1-1167-A REPORT DATE: 03/31/20

GEOTECHNICAL BORING LOG

PROJECT NO.	1-1167-A	PROJECT NAME	Marriott Riverside		
DATE STARTED	2/18/20	GROUND ELEV.(FT)	862	BORING DESIG.	B-3
DATE FINISHED	2/18/20	GW DEPTH (FT)		LOGGED BY	ECH
DRILLER	Oneway Drilling	DRIVE WT.	140 lb.	NOTE	
TYPE OF DRILL RIG	Hollow Stem	DROP	30 in.		

DEPTH (feet)	ELEV.	SAMPLE TYPE	BLOWS/FT	GROUP SYMBOL	GEOTECHNICAL DESCRIPTION	MOISTURE CONT. (%)	DRY (pcf) DENSITY	OTHER TESTS
		SPT	22	SM/ML	@ 40': Grades between Silty SAND and Sandy SILT, brown to dark yellowish-brown (10 YR 4/3-4/4), slightly moist to moist, medium dense / stiff, fine micaceous sand, low apparent plasticity silt.			
45	817	R	58	SM	@ 45': Silty SAND, moist, dense, fine-grained, micaceous.	8.8	118.4	
50	812	SPT	28	ML	@ 50': Sandy SILT, brown (10 YR 4/3), moist, very stiff, low apparent plasticity, fine micaceous sand.			
					Total Depth: 51.5 feet. Depth of artificial fill is approximate. No groundwater encountered. Hole backfilled with cuttings.			
55	807							
60	802							
65	797							
70	792							
75	787							

SAMPLE TYPES:
R RING (DRIVE) SAMPLE
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PN: 1-1167-A REPORT DATE: 03/31/20

APPENDIX B

Laboratory Testing Results

AB A E P CED E AND E

The sa ples obtained during the field investigation ere transported to the laboratory for testing and analysis. The results of tests perfor ed on selected sa ples and the test procedures are su ari ed belo .

Dry Den ity an oi ture Content

ield dry density and oisture contents of undisturbed soils sa ples retained in 2 3 inch inside dia eter by one-inch height rings ere deter ined, and oisture test results ere obtained for the s all bul sa ples. Dry density and oisture content testing ere perfor ed in accordance ith ASTM D2 37 and ASTM D2216, respectively. The test results are posted on the Geotechnical Boring Logs in Appendi A.

a imum Dry Den ity an timum oi ture Content

Ma i u dry density and opti u oisture content tests ere perfor ed on representative bul soil sa ples in accordance ith ASTM D 1 7. The results are presented belo

am le ocation	a imum Dry Den ity c	timum oi ture Content
B-2 0-7.	12 .0	.

E an ion In e

Representative soil sa ples ere tested for e pansion potential follo ing the BC 2 -2 Test Procedure. Test results are presented belo .

am le ocation	E an ion In e	E an ion Potential BC
B-2 7.	70	Mediu

ul ate Content

Selected soil sa ples ere tested for soluble sulfate content in accordance ith the ach ethod. The test results are sho n belo .

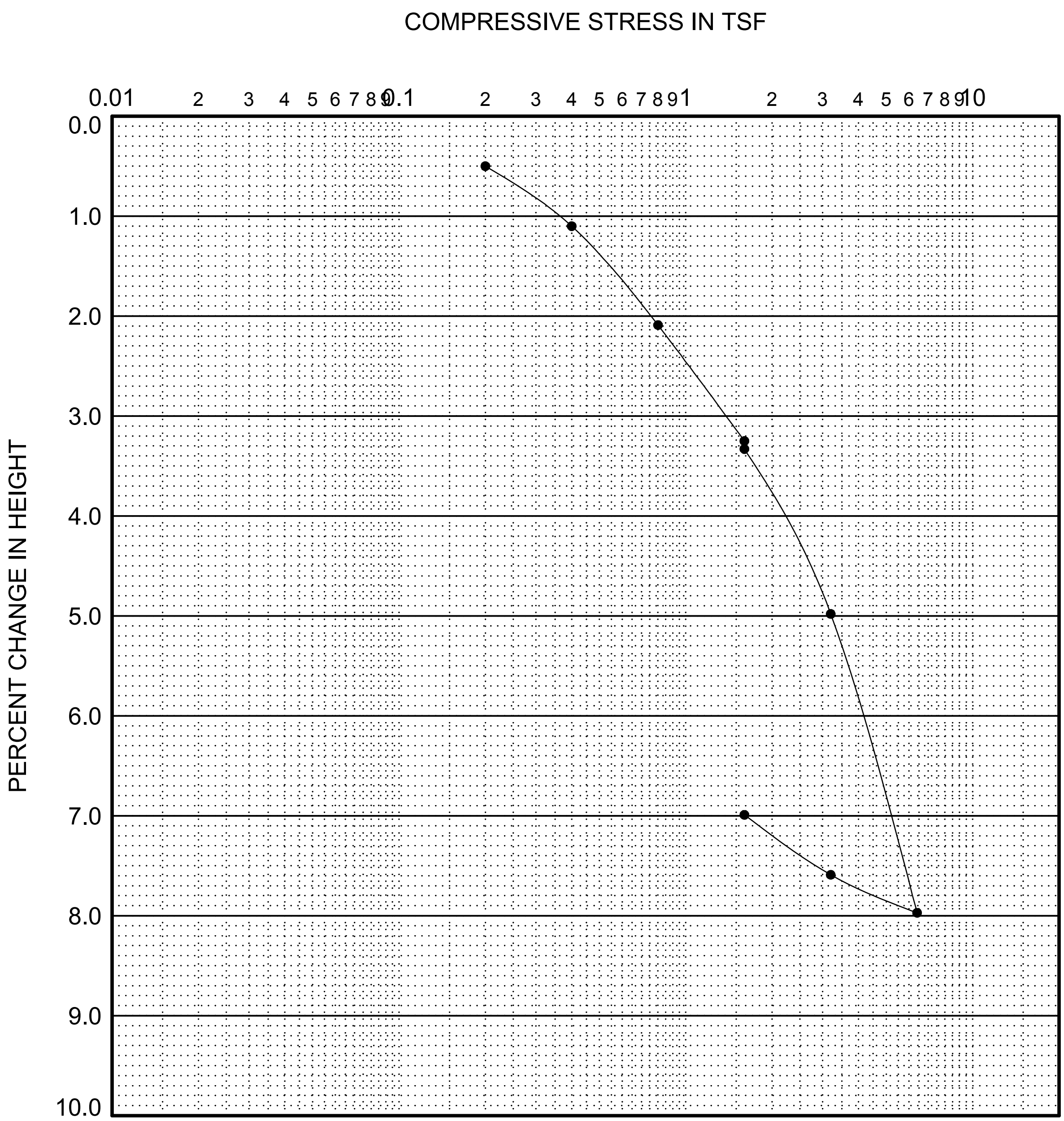
am le ocation	ater olu le ul ate in oil m	ul ate E o ure ACI a le
B-2 0-7.	7	S0
B-2 1	12	S0
B-3 40	6	S0

Con oli ation

To deter ine the co pressibility characteristics, consolidation tests ere perfor ed on representative, relatively undisturbed soil sa ples. The test speci ens ere initially loaded to 0.2 tons per s uare foot and soa ed during the tests to si ulate possible adverse field conditions. Progressive loading as then applied to a a i u of 12. tons per s uare foot to si ulate e pected additional loading due to the proposed i prove ents. Loading as then reduced to deter ine rebound characteristics. Consolidation test results are presented on igures C-1 through C-7.

Direct hear

Direct shear tests ere perfor ed on representative, relatively undisturbed soil sa ples ith a direct shear achine of the strain-controlled type in hich the rate of strain is 0.01 inches per inute. The soil speci ens ere soa ed in a confined state prior to shearing and ere sheared under varied nor al loads ranging fro 1.0 sf to .0 sf. The test results are plotted on igures S-1 through S-7.



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-1	25.0	108.1	19.3		SM/ML	3466 Mission Inn Avenue, Riverside

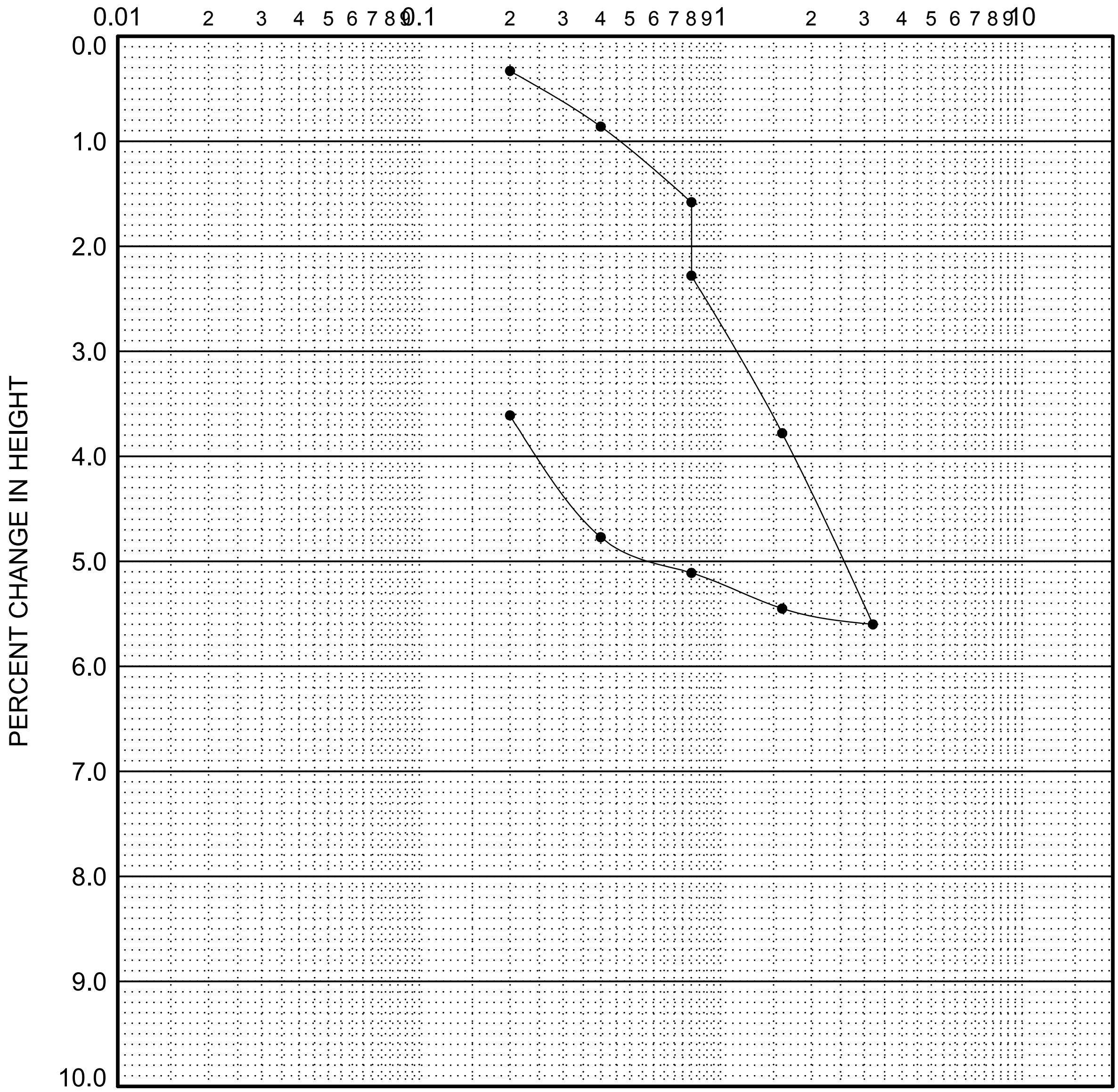
WATER ADDED AT 1.6 TSF.

FIGURE C-1
CONSOLIDATION CURVE

PN:1-1167-A REPORT DATE: 03/31/20

G3SoilWorks
350 Fischer Ave. Front
Costa Mesa, CA 92626
Phone: (714) 668 5600
www.G3SoilWorks.com

FIG. C-1



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-2	17.5	108.2	6.7		SM	3466 Mission Inn Avenue, Riverside

WATER ADDED AT 0.8 TSF.

FIGURE C-2
CONSOLIDATION CURVE

PN:1-1167-A REPORT DATE: 03/31/20

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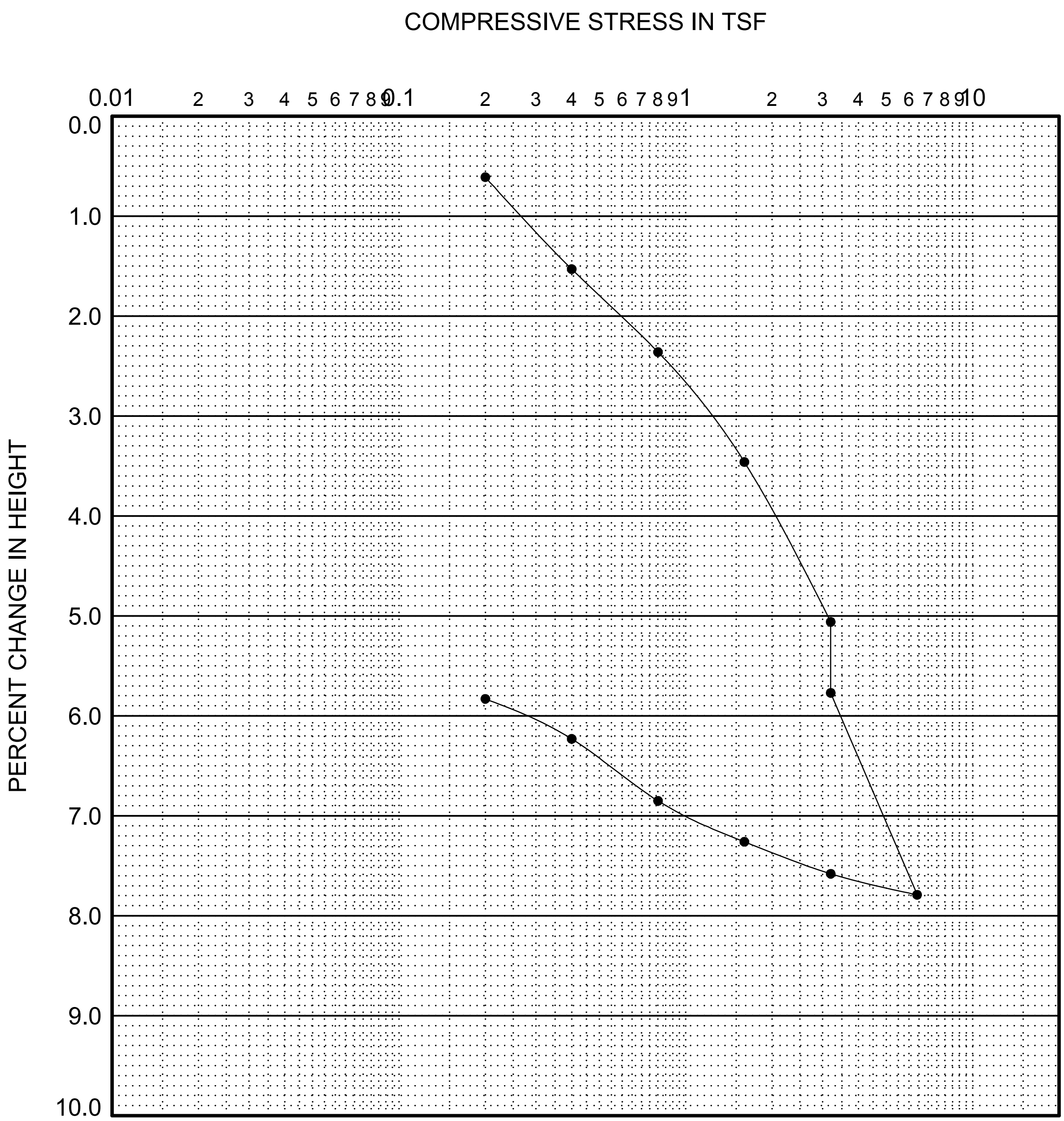
Costa Mesa, CA 92626
Phone: (714) 668 5600
www.G3SoilWorks.com

FIG. C-2



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-2	65.0	107.3	19.8		SM	3466 Mission Inn Avenue, Riverside

WATER ADDED AT 3.2 TSF.

FIGURE C-3
CONSOLIDATION CURVE

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Costa Mesa, CA 92626
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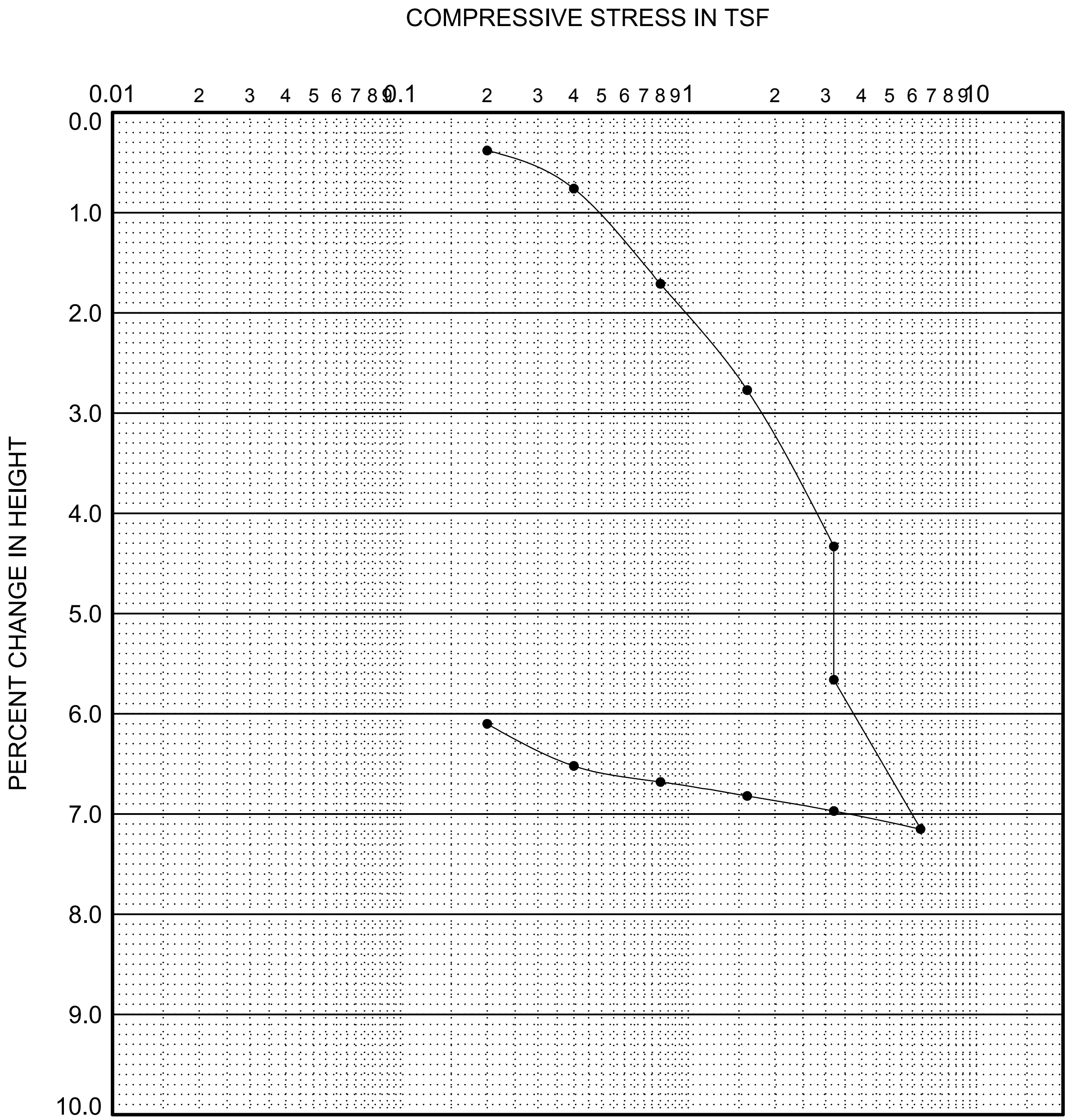
FIG. C-3



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-2	75.0	117.8	6.6		SM	3466 Mission Inn Avenue, Riverside

WATER ADDED AT 3.2 TSF.

FIGURE C-4
CONSOLIDATION CURVE

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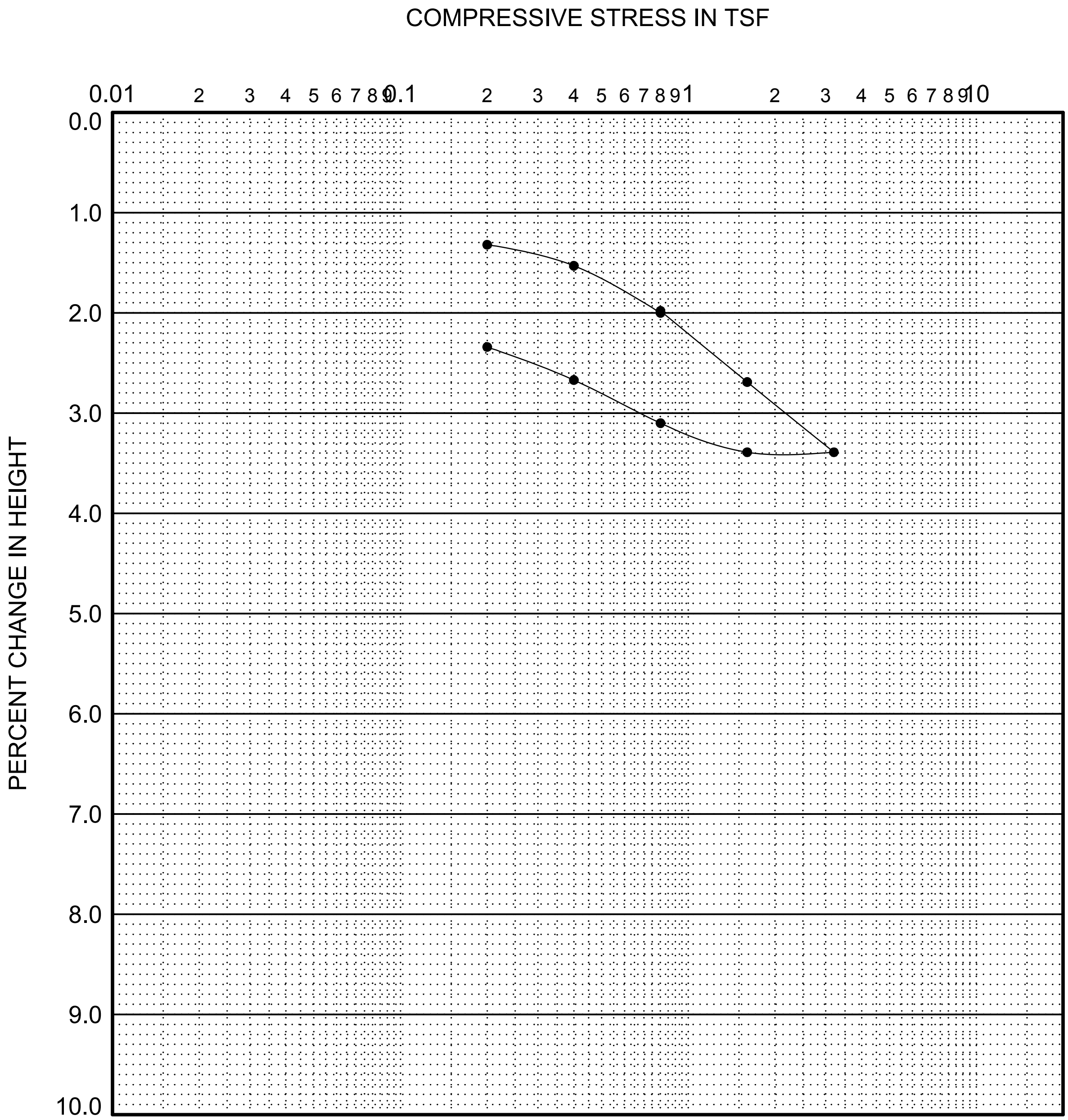
FIG. C-4



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-3	5.0	109.8	13.4		ML	3466 Mission Inn Avenue, Riverside

WATER ADDED AT 0.8 TSF.

FIGURE C-5
CONSOLIDATION CURVE

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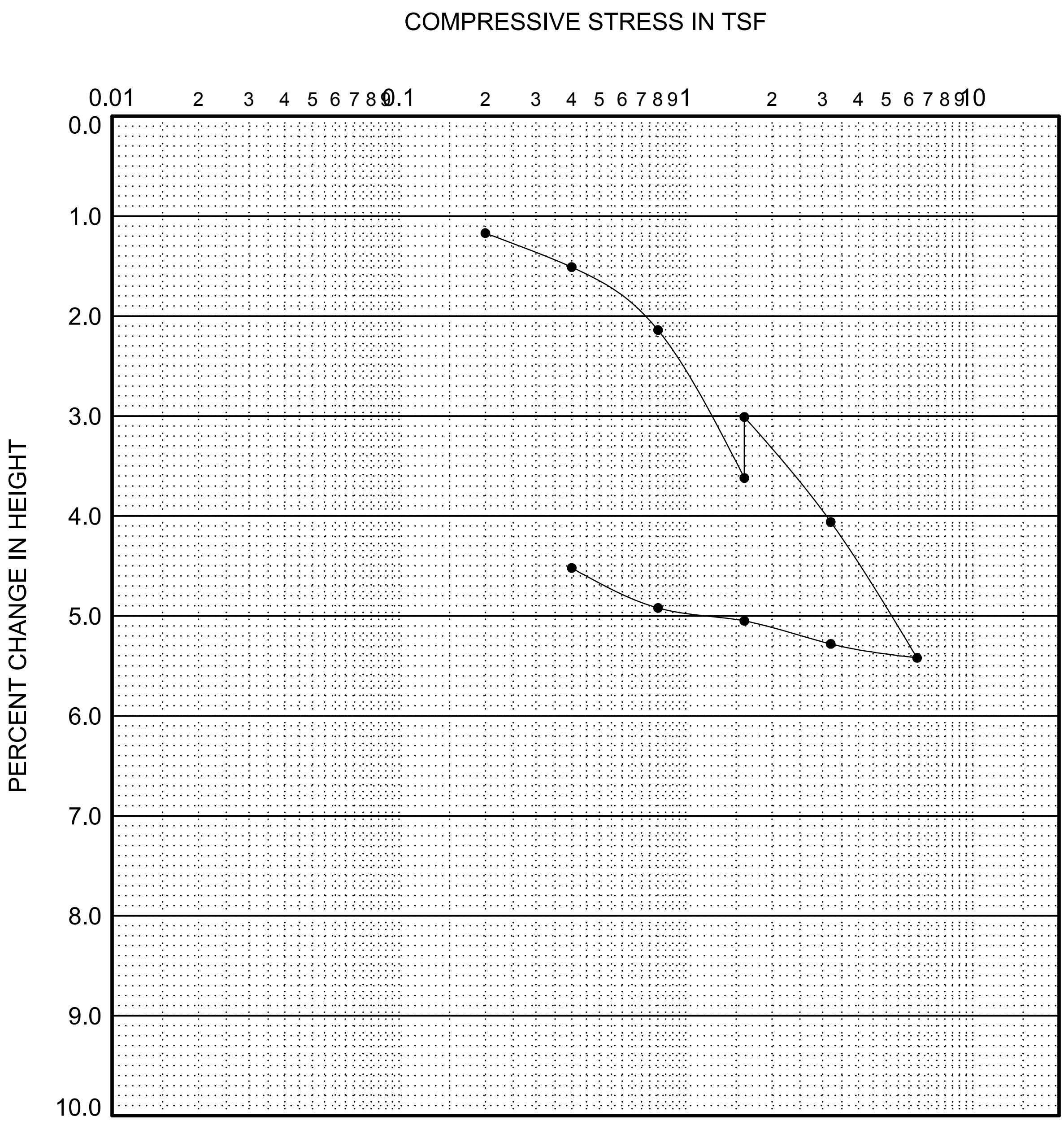
FIG. C-5



Boring	Depth(ft.)	Dry Density	in situ Moist.	-200 sieve	Group Symbol	Soil Description
B-3	45.0	118.4	8.8		SM	3466 Mission Inn Avenue, Riverside

WATER ADDED AT 1.6 TSF.

FIGURE C-6
CONSOLIDATION CURVE

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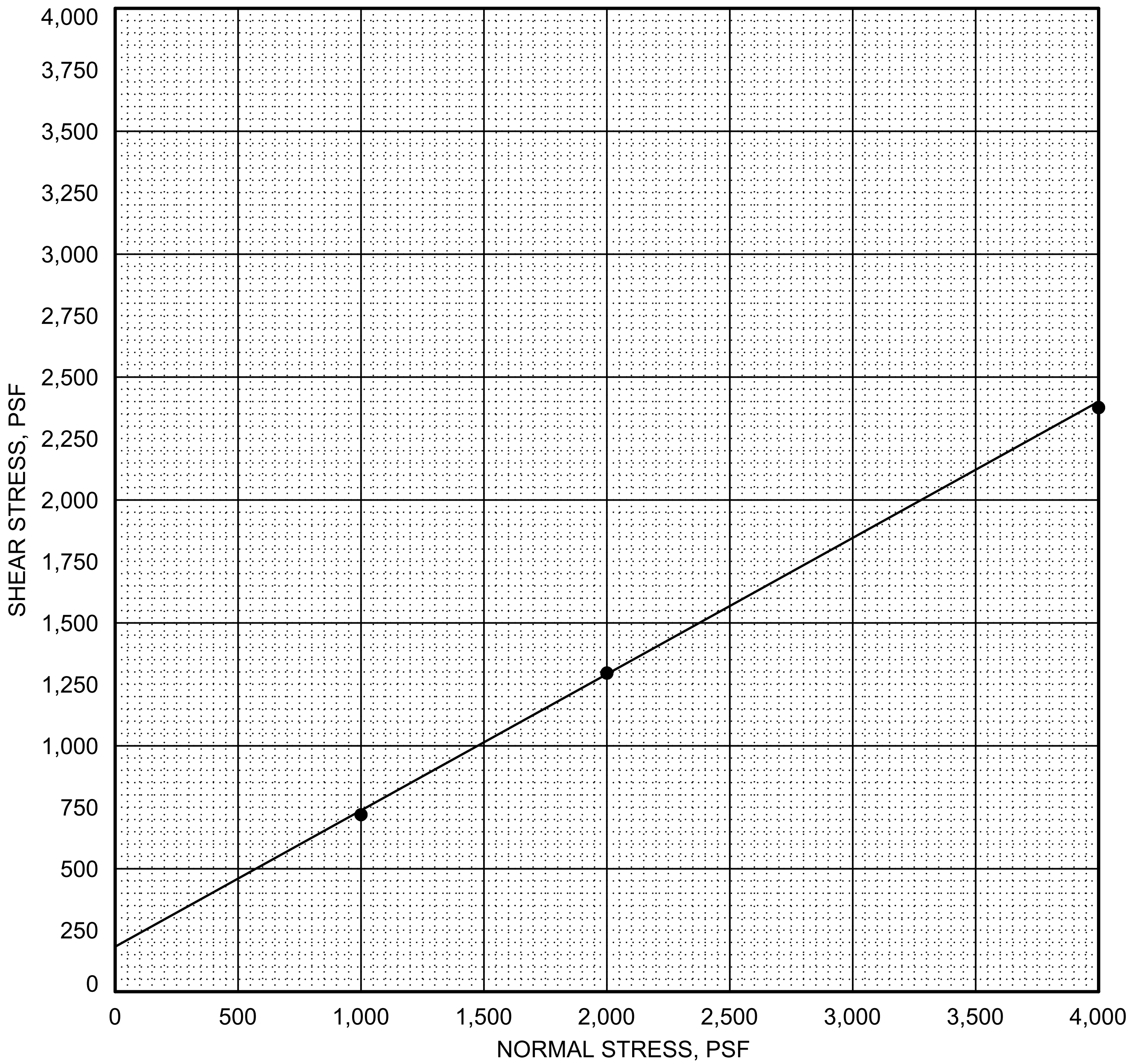
FIG. C-6

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION 180 psf.
	FRICTION ANGLE 29.0 degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-1	15.0			

FIGURE S-1
DIRECT SHEAR TEST

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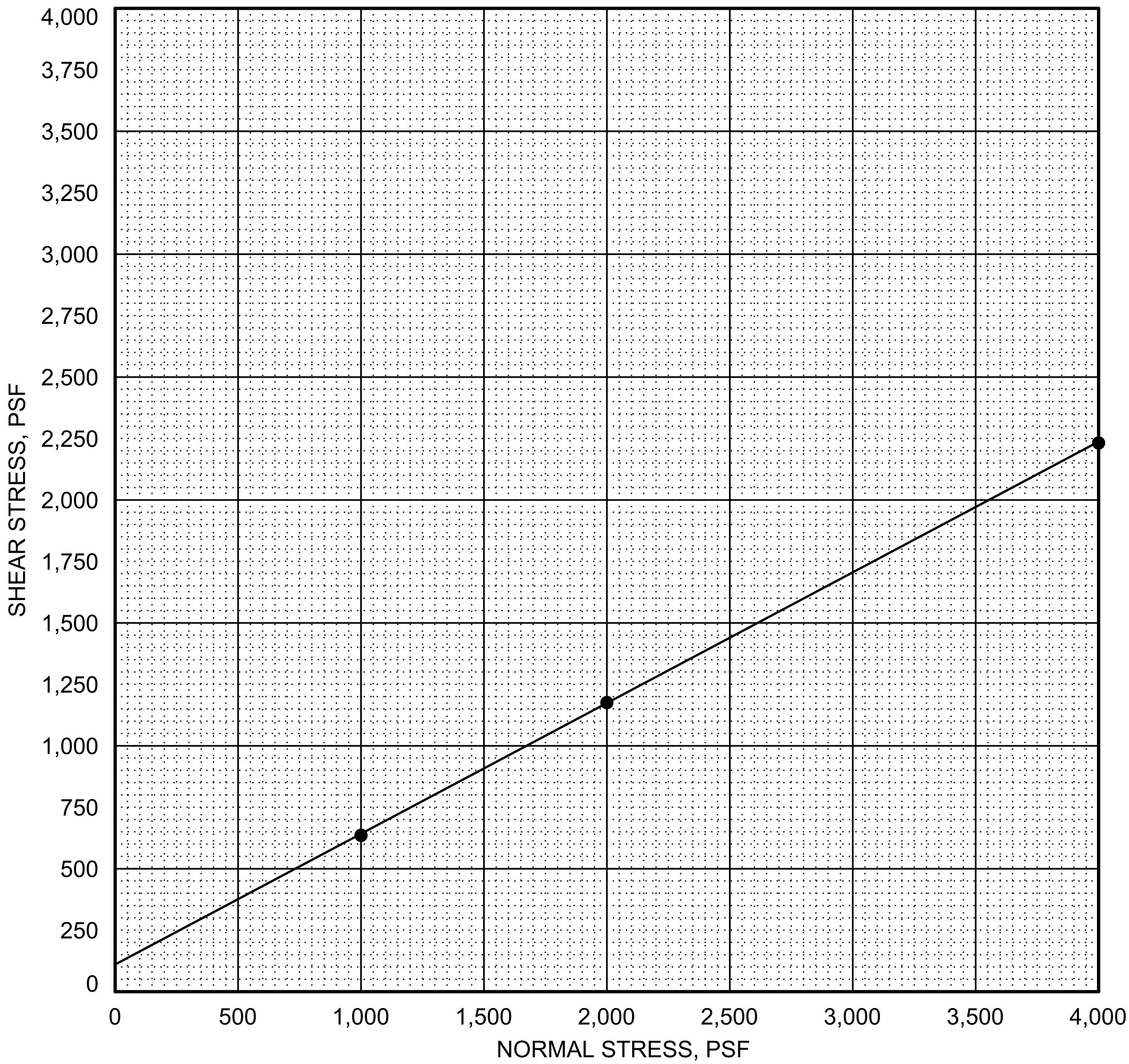
FIG. S-1

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION	108	psf.
	FRICTION ANGLE	28.0	degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-1	25.0			

FIGURE S-2
DIRECT SHEAR TEST

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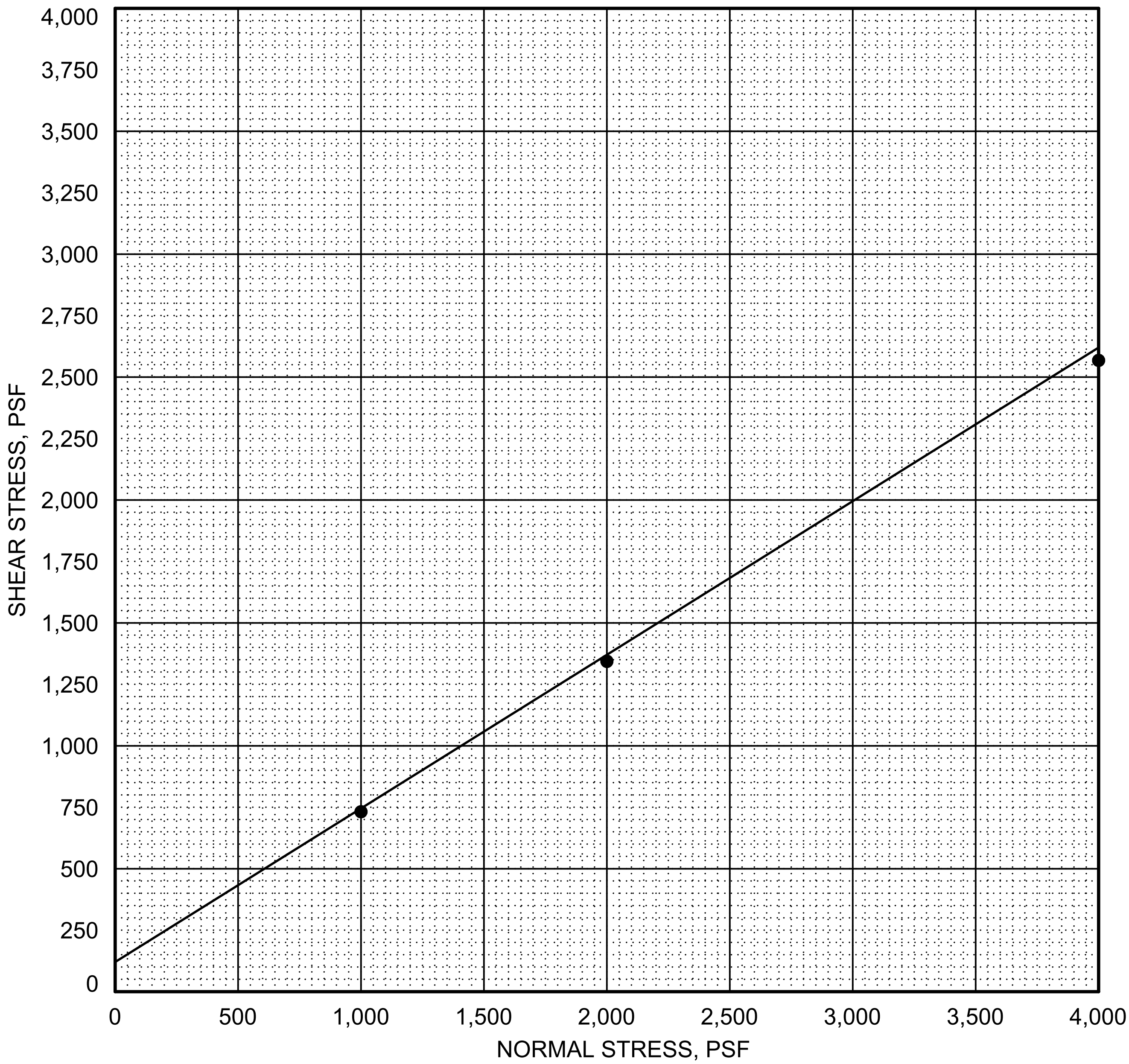
FIG. S-2

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION 120 psf.	
	FRICTION ANGLE 32.0 degrees	

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-2	17.5			

FIGURE S-3
DIRECT SHEAR TEST

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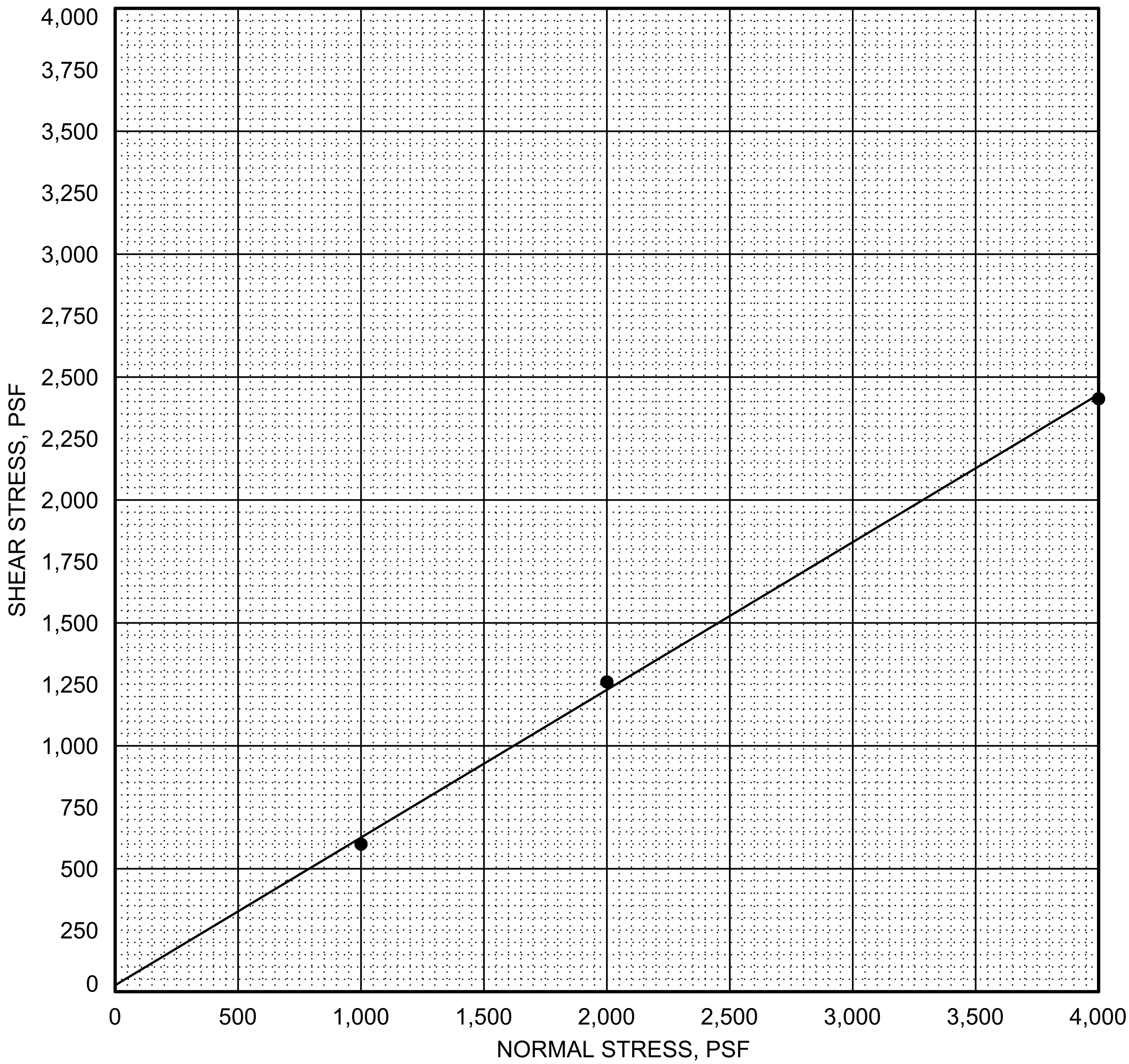
FIG. S-3

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION	24	psf.
	FRICTION ANGLE	31.0	degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-2	65.0			

FIGURE S-4
DIRECT SHEAR TEST

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Costa Mesa, CA 92626
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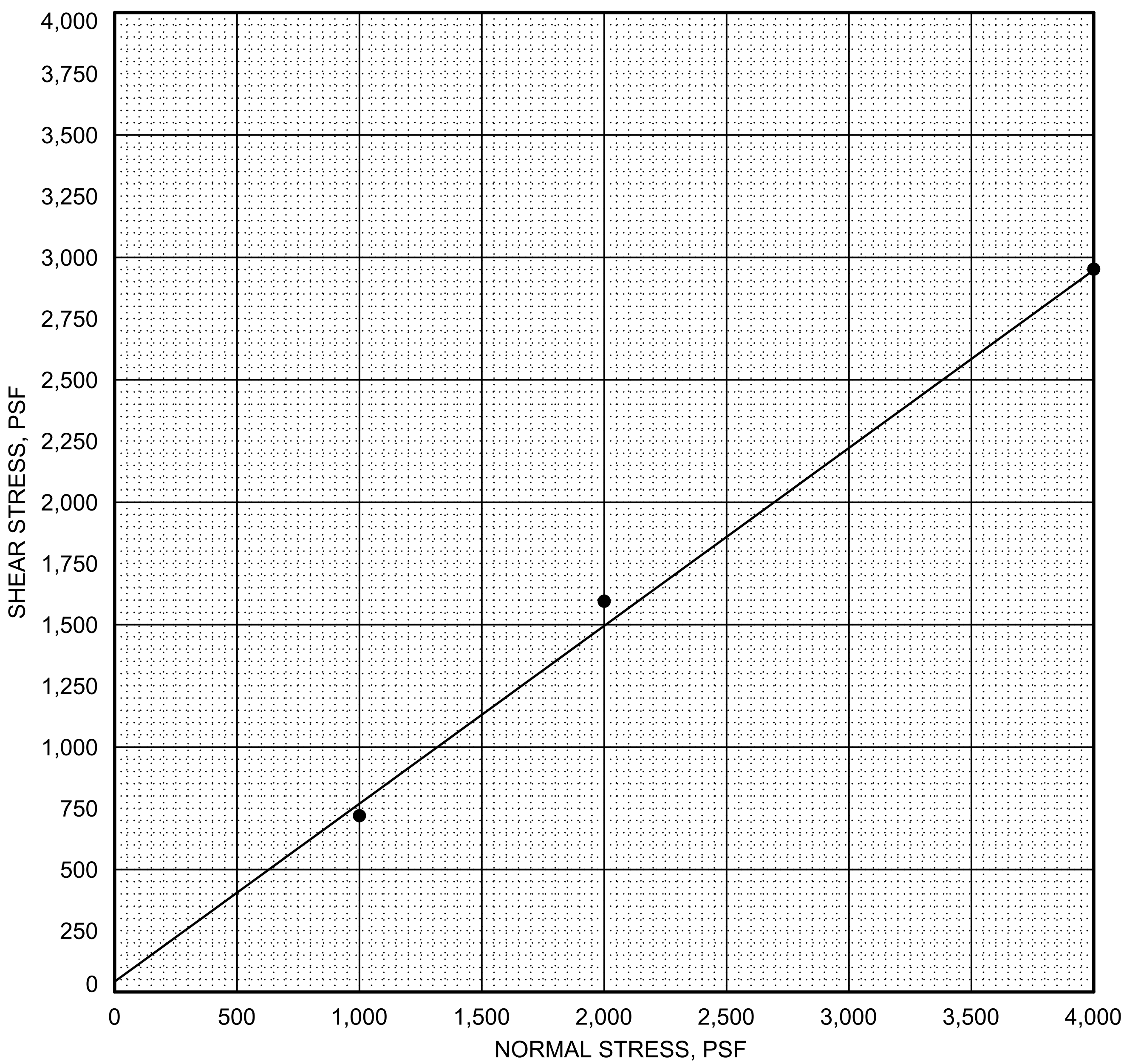
FIG. S-4

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION 42 psf.
	FRICTION ANGLE 36.0 degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-2	75.0			

FIGURE S-5
DIRECT SHEAR TEST

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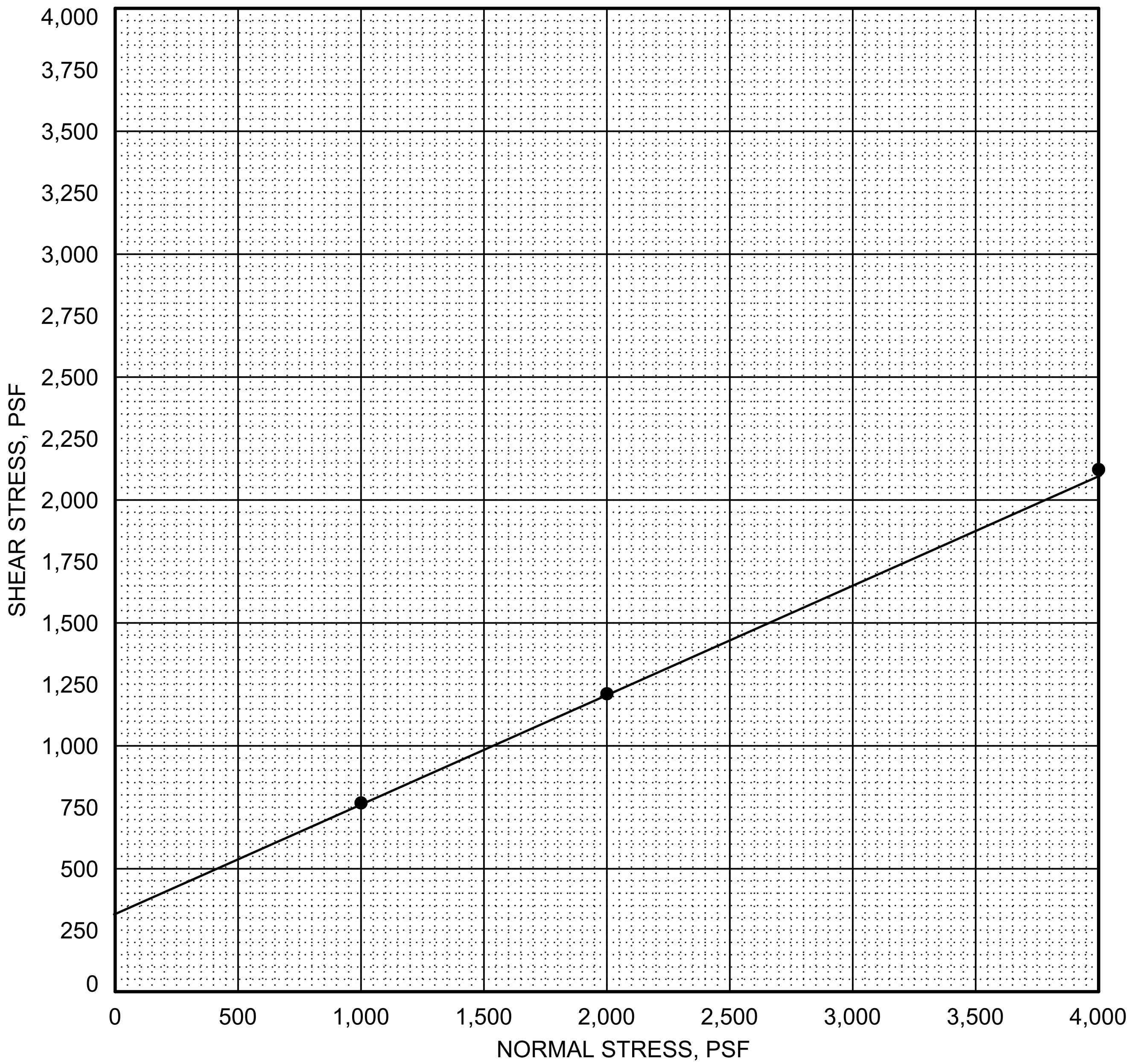
FIG. S-5

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION 312 psf.
	FRICTION ANGLE 24.0 degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-3	5.0			

FIGURE S-6
DIRECT SHEAR TEST

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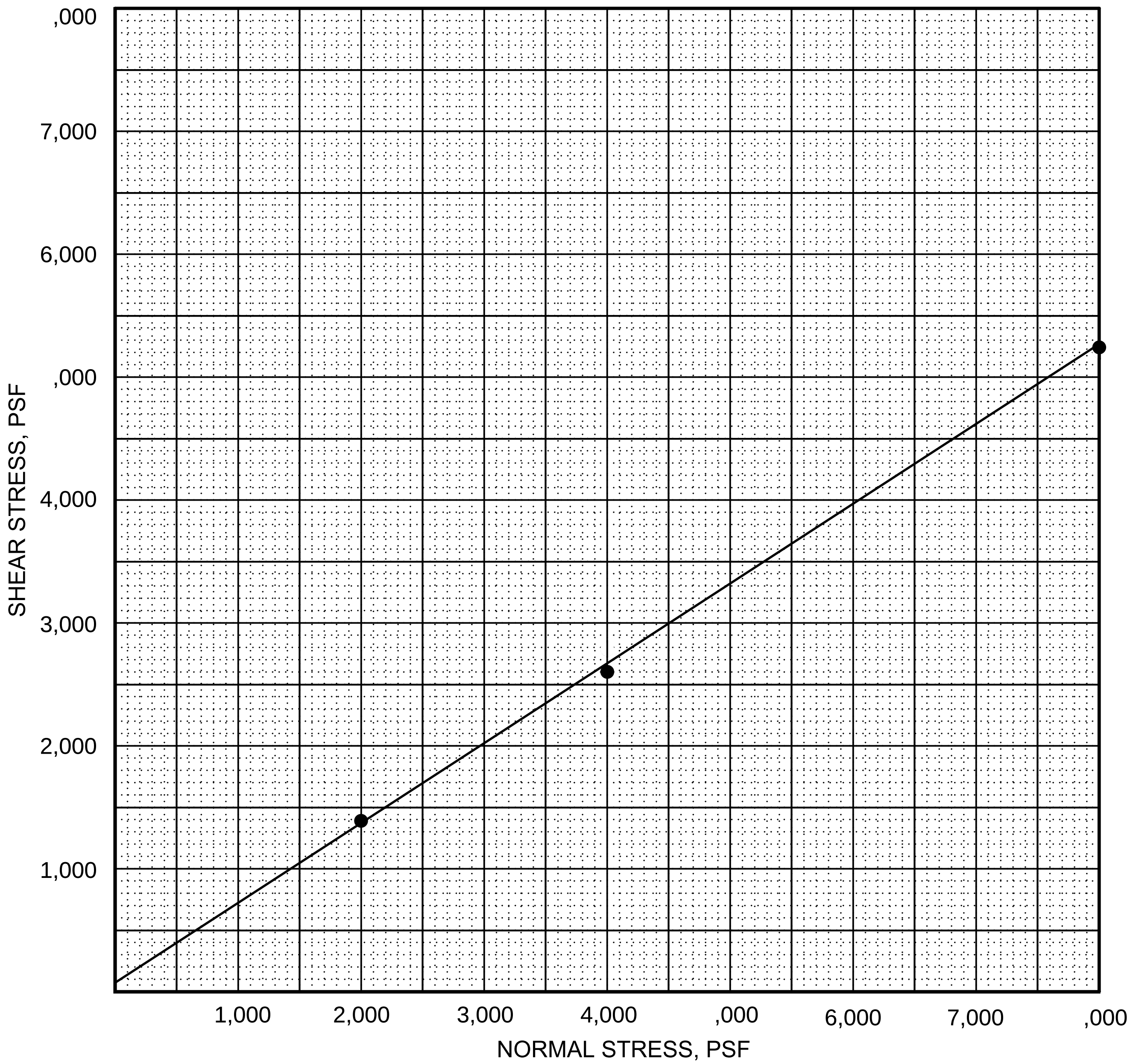
Costa Mesa, CA 92626
Phone: (714) 668 5600
www.G3SoilWorks.com

FIG. S-6

DIRECT SHEAR TEST
Undisturbed



3466 Mission Inn Avenue, Riverside	COHESION	72	psf.
	FRICTION ANGLE	33.0	degrees

symbol	boring	depth (ft.)	symbol	boring	depth (ft.)
●	B-3	45.0			

FIGURE S-7
DIRECT SHEAR TEST

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Costa Mesa, CA 92626
Phone: (714) 668 5600
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FIG. S-7

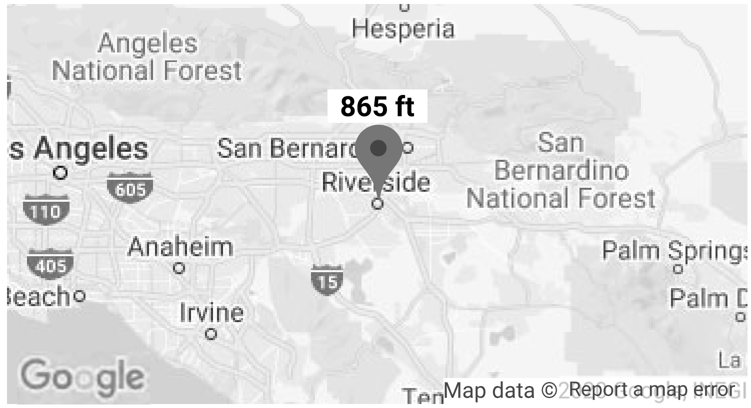
APPENDIX C

ASCE 7-16 SEISMIC DESIGN PARAMETERS

Hazards by Location

Search Information

Address:	3466 Mission Inn Avenue, Riverside, CA 92501, USA
Coordinates:	33.981265, -117.370897
Elevation:	865 ft
Timestamp:	2020-03-23T20:43:40.984Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	III
Site Class:	D



Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.6	MCE_R ground motion (period=1.0s)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.939	Coefficient of risk (0.2s)
CR_1	0.912	Coefficient of risk (1.0s)
PGA	0.541	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.595	Site modified peak ground acceleration

T_L	8	Long-period transition period (s)
SsRT	1.733	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.847	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.643	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.704	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.541	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.