REPORT OF PRELIMINARY GEOTECHNICAL / GEOLOGIC STUDY & INFILTROMETER TESTING PROPOSED RESTAURANT PADS AND BUILDING RENOVATION 216 BROOKSIDE AVENUE, NORTHWEST CORNER OF EUREKA STREET AND BROOKSIDE AVENUE CITY OF REDLANDS SAN BERNARDINO COUNTY, CALIFORNIA

PROJECT NO.: 1151-A17
REPORT NO.: 1

FEBRUARY 12, 2018

SUBMITTED TO:

VANTAGE ONE REAL ESTATE INVESTMENTS V, LLC
4 CORPORATE PLAZA DRIVE, SUITE 210
NEWPORT BEACH, CA 92660

PREPARED BY:

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February 12, 2018

Vantage One Real Estate Investments V, LLC
4 Corporate Plaza Drive, Suite 210
Newport Beach, CA 92660

Project No.: 1151-A17
Report No.: 1

Attention: Mr. Thomas N. Robinson

Subject: **Report of Preliminary Geotechnical / Geologic Study & Infiltrometer Testing, Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Northwest Corner of Eureka Street and Brookside Avenue, City of Redlands, San Bernardino County, California.**

References:

1. **GreenbergFarrow**, Undated, Unsigned, *Redlands, AC NWC Brookside Ave. & Eureka St.*, GFA Project No. 2016103.0, Sheet Nos. SP-5 & SP-6, Scales 1"=20' and 1"=40'.

2. **County of San Bernardino**, May 19, 2011, *Technical Guidance Document Appendices, Appendix VII., Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations.*

3. Technical References - See Appendix 'B.'

Mr. Robinson:

According to your request, we have completed a preliminary geotechnical / geologic study for the design and construction of the proposed restaurants and building renovation. We are presenting, herein, our findings and recommendations.

The recommendations presented in this report are considered preliminary since the proposed grading, the floor level elevations, the type of structures construction, the structural loads, etc. were not known at the time of this report. The findings of this study indicate that the project site is suitable for the proposed development and renovations provided the recommendations presented in the attached report are complied with and incorporated into the design and construction of the project.

Copies of this report should be forwarded to your other consultants for the project (i.e., Civil Engineer, Architect, Structural Engineer, etc.) as needed to implement the recommendations presented. The required number of the original, wet ink signed reports should be saved for submittal, and the other required documentation to the appropriate agency having jurisdiction over the project for review and permitting purposes.

If you have any questions after reviewing the findings and recommendations contained in the attached report, please do not hesitate to contact this office. This opportunity to be of professional service is sincerely appreciated.

Respectfully Submitted,
HILLTOP GEOTECHNICAL, INC.

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PROJECT NO.: 1151-A17
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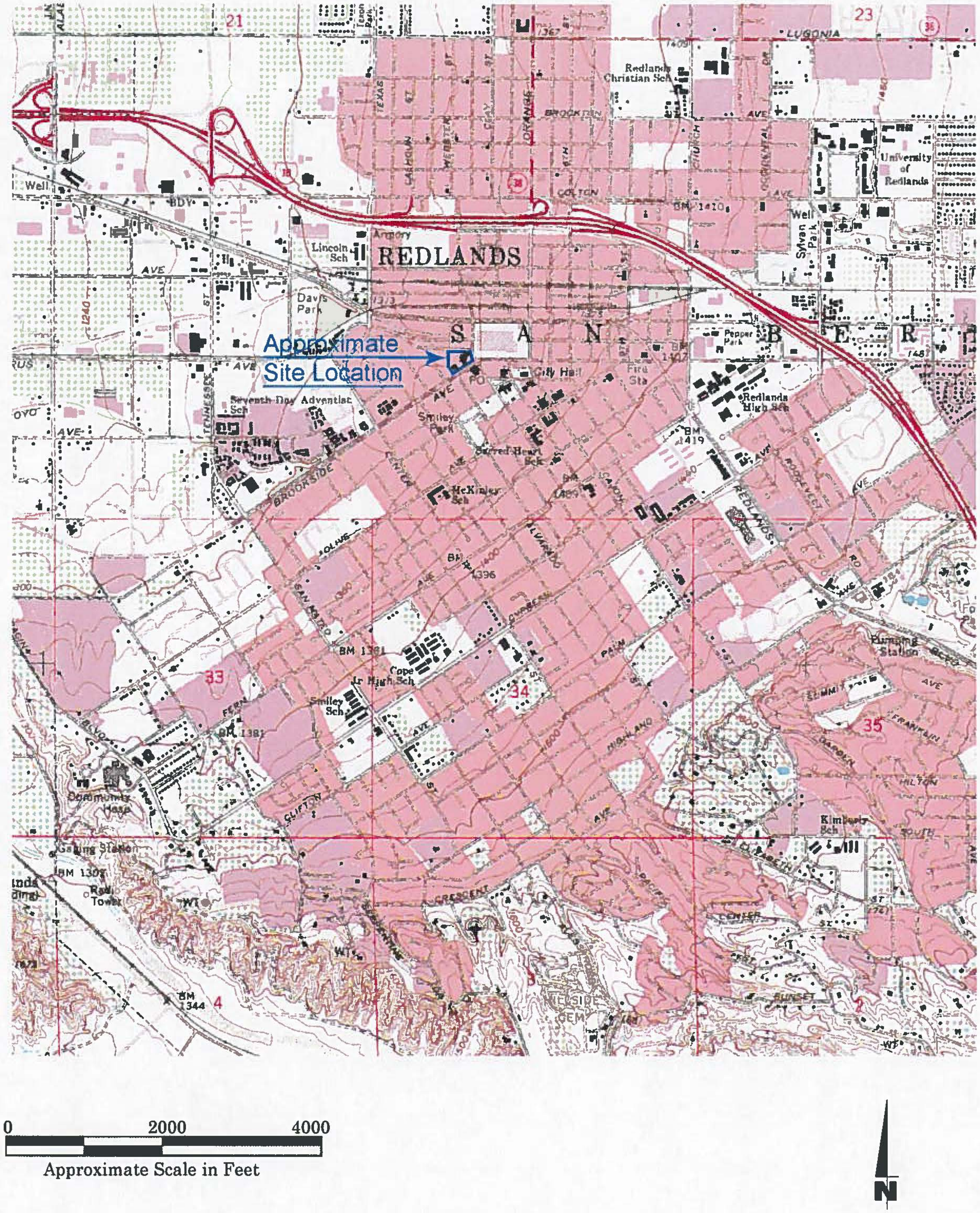
INTRODUCTION

AUTHORIZATION

This report presents results of the preliminary geotechnical / geologic study conducted on the subject site for the proposed restaurants and building renovation be located on the northwest corner of Eureka Street and Brookside Avenue at 216 Brookside Avenue in the City of Redlands, San Bernardino County, California. The general location of the subject site is indicated on the 'Site Location Map,' Figure No. 1.

Authorization to perform this study was in the form of a signed proposal from Mark Hulett of **Hilltop Geotechnical, Inc.** (Geotechnical / Geologic Consultant) to **Vantage One Real Estate Investments V, LLC** (Client), dated November 30, 2017, Proposal Number: P17196 and was received and signed by Mr. Thomas N. Robinson, the managing member, on December 14, 2017.



Reference: **United States Department of the Interior, Geologic Survey**, 1967, Photorevised 1988, *Redlands Quadrangle, California, San Bernardino Co.*, 7.5 Minute Topographic Series, Scale 1:24,000.

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SITE LOCATION MAP

By: AH	Date: 02/2018
Project No.: 1151-A17.1	Figure No.: 1

PURPOSE AND SCOPE OF STUDY

The scope of work performed for this study was designed to determine and evaluate the surface and subsurface conditions on the subject site with respect to geotechnical characteristics, including potential geologic hazards that may effect the development of the site, and to provide geotechnical recommendations and criteria for use in the design and construction of the proposed development. The scope of work included the following:

- Review of locally and easily available published and unpublished soil, geologic, and seismologic reports and data for the area (see References in Appendix 'B'), available historic photographs, flood hazard maps, well data, etc. to ascertain earth material, geologic, and hydrologic conditions of the area.
- Telephone conversations with the client and/or representatives of the client.
- Site reconnaissance.
- Subsurface exploration by means of borings to characterize the existing pavement section, earth materials, geologic, and groundwater conditions that could influence the proposed development.
- Sampling of on-site earth materials from the exploratory excavations.
- Laboratory testing of selected earth material samples considered representative of the subsurface conditions to determine the engineering properties and characteristics.
- Define the general geology of the subject site and evaluate potential geologic hazards which would have an effect on the proposed site development.
- Determine seismic classification of the site to meet the requirements of the 2016 California Building Code (CBC), effective on January 1, 2017.
- Engineering analysis of field and laboratory data to provide a basis for geotechnical conclusions and recommendations regarding site grading and foundation, floor slab, retaining wall, pavement, etc. design parameters.

- Preparation of this report to present the geotechnical and geologic conclusions and recommendations for the proposed site development.

This report presents our conclusions and/or recommendations regarding:

- The geologic setting of the site.
- Potential geologic hazards (including landslides, seismicity, faulting, liquefaction potential, etc.)
- General subsurface earth conditions.
- Presence and effect of expansive, collapsible, and compressible earth materials.
- Groundwater conditions within the depth of our subsurface study.
- Excavation characteristics of the on-site earth materials.
- Characteristics and compaction requirements of proposed fill and backfill materials.
- Recommendations and guide specifications for earthwork.
- Seismic design coefficients for structural design purposes.
- Types and depths of foundations.
- Allowable bearing pressure and lateral resistance for foundations.
- Estimated total and differential settlements.
- Preliminary corrosion potential evaluation for concrete in direct contact with the on-site earth materials.
- Temporary and permanent cut and fill slope recommendations.
- Utility trench excavation and backfill recommendations.

- Slope maintenance and protection recommendations.
- Preliminary pavement recommendations.
- Percolation parameters.

The scope of work performed for this report did not include any testing of earth materials or groundwater for environmental purposes, an environmental assessment of the property, or opinions relating to the possibility of surface or subsurface contamination by hazardous or toxic substances. In addition, evaluation of on-site private sewage disposal systems for the proposed development was not part of this study.

This study was prepared for the exclusive use of **Mr. Thomas N. Robinson** of **Vantage One Real Estate Investments V, LLC**, and his consultants for specific application to the proposed restaurant pads and building renovation in accordance with generally accepted standards of the geotechnical and geologic professions and generally accepted geotechnical (soil and foundation) engineering and geologic principles and practices at the time this report was prepared. Other warranties, implied or expressed, are not made. Although reasonable effort has been made to obtain information regarding geotechnical / geologic and subsurface conditions of the site, limitations exist with respect to knowledge of unknown regional or localized off-site conditions which may have an impact at the site. The conclusions and recommendations presented in this report are valid as of the date of this report. However, changes in conditions of a property can occur with passage of time, whether they are due to natural processes or to works of man on this and/or adjacent properties.

If conditions are observed or information becomes available during the design and construction process which are not reflected in this report, **HGI**, as Geotechnical / Geologic Consultant of record for the project, should be notified so that supplemental evaluations can be performed and conclusions and recommendations presented in this report can be verified or modified in writing, as necessary. Changes in applicable or appropriate standards of care in the geologic / geotechnical professions occur, whether they result from legislation or the broadening of knowledge and experience. Accordingly, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes outside the influence of the project Geotechnical / Geologic Consultant which occur in the future.

PREVIOUS SITE STUDIES

No previous geotechnical and/or geological studies for design and/or the construction of existing structures and associated improvements on the subject property were available for review at the time of this study. A records request was submitted to the City of Redlands on January 17, 2018 for all documents that refer the soils related projects such as grading reports, plans, preliminary geotechncial reports, and soils reports etc. On February 5, 2018 the City of Redlands, Jimmy Nguyen, responded to the request via email. The response concluded that the City does not have any records responsive to our request. It is our understanding the existing building and basement was once the City of Redlands Police Department, and is anticipated to be demolished for the new proposed development.

PROJECT DESCRIPTION / PROPOSED DEVELOPMENT

As part of our study, we have discussed the project with Mr. Thomas N. Robinson, the client, and Mr. Matt Hicks of Hicks and Hartwick, the civil engineer for the project. We have also been provided with the referenced plans for the project noted on the first page of the cover letter for this report.

Based on information presented to this firm, it is our understanding that the proposed project will consist of two new restaurant buildings with accompanying Hot Mix Asphaltic (HMA) concrete parking lot, Portland Cement concrete (PCC) driveways, curbs and gutters, a decorative concrete block perimeter wall, and a trash enclosure. It is our understanding that the building previously occupied by the police department will be demolished. The existing office building on the southwest portion of the site will be renovated. Additionally, we understand that the existing parking lot will be removed and replaced with new asphalt.

The proposed new restaurant buildings are expected to be a single-story structure consisting of wood trusses on wood beams and steel columns, wood studs, and veneered walls. It is assumed that light to moderate loads will be imposed on the foundations. The foundation loads are not anticipated to exceed 3,500 pounds per lineal foot (plf) for continuous footings and 60 kips for column footings. The proposed structure ground level floor will consist of a concrete slab cast on compacted subgrade. Finish floor elevation for the structure had not been furnished at the time of our study, but it is anticipated to be within 3.0 feet of existing site grades. Therefore, no cut or fill slopes are anticipated to be required for the development of the site. Subterranean construction is not anticipated for the proposed structure. It is anticipated that low height retaining walls may also be needed to develop the subject site. On-site stormwater drainage system for Water Quality Management Plan (WQMP) is anticipated to be constructed in the west portion of the subject site.

The above project description and assumptions were used as the basis for the field exploration, laboratory testing program, the engineering analysis, and the conclusions and recommendations presented in this report. **HGI** should be notified if structures, foundation loads, grading, and/or details other than those represented herein are proposed for final development of the site so a review can be performed, a supplemental evaluation made, and revised recommendations submitted, if required.

FIELD EXPLORATION AND LABORATORY TESTING

The field study performed for this report included a visual reconnaissance of existing surface conditions of the subject site and surrounding area. A study of the property's subsurface condition was performed to evaluate underlying earth strata and the presence of groundwater. Surface and subsurface conditions were explored on January 4, 2018.

The subsurface exploration consisted of excavating four (4) exploratory borings and two (2) shallow infiltration borings in the area of the proposed structures on the subject property. The approximate locations of the exploratory excavations are shown on the 'Exploratory Excavation Location Plan,' Plate No. 1, presented in Appendix 'A' of this report. The exploratory excavations were observed and logged by a representative of **HGI**. Earth materials encountered in the exploratory excavations were visually described in the field in general accordance with the current Unified Soils Classification System (USCS), ASTM D2488, visual-manual procedures, as illustrated on the attached, simplified 'Subsurface Exploration Legend,' Plate No. 2, presented in Appendix 'A' of this report. The results are presented on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in Appendix 'A' of this report.

A more detailed explanation of the field study which was performed for this report is presented in Appendix 'A'of this report.

Relatively undisturbed ring samples, and representative bulk samples of on-site fill and natural earth materials were collected during the field exploration and returned to the laboratory for testing. Laboratory tests were conducted to evaluate the index and engineering properties of on-site earth materials and included in-situ dry density and moisture content tests, an expansion index test, a soluble sulfate chemical tests, a sieve analysis test, a maximum dry density / optimum moisture content relationship test, and consolidation tests. A more detailed explanation of laboratory tests performed for this study and test results are presented in Appendix 'A' of this report, Plate Nos. 7 through 10.

FINDINGS

SITE DESCRIPTION

The subject property comprises approximately 3.03 acres and was irregular in shape as shown on the Reference No. 1 'Site Plan' noted on the first page of this report. The property to be demolished is located at 216 Brookside Avenue in the City of Redlands, San Bernardino County, California. The building to be renovated was located on Brookside Avenue and connected via parking lot to the adjacent property on the northwest corner of Eureka Street and Brookside Avenue. The properties are located in southeast one-quarter of T1S, R3W of the San Bernardino Principle Meridian at Latitude: 34.0556° North, Longitude: 117.1860° West.

Currently the site contains two buildings with basements, a parking lot, various planter boxes, a fountain, light vegetation, and sidewalks. The parking lot was generally open with many access points from the surrounding streets. Additionally, a flag pole stood on the southern side of the eastern building along with a radio tower. It is our understanding the eastern building, proposed to be demolished, was the past City of Redlands Police Department.

The subject property is bounded by Brookside Avenue to the south, Eureka Street to the east, existing residences to the west and north, and an additional access point on the northwest portion of the site from Citrus Avenue. The two proposed restaurant pads are to be located on the eastern portion of the site, where the existing building is to be demolished. The renovations are proposed to be furnished on the existing office building on the south southwest portion of the site, as shown on the 'Exploratory Excavation Location Plan,' Plate No. 1, presented in Appendix 'A' of this report. It is our understanding that the proposed renovations do not require any geotechnical consideration.

Overall the site had a shallow, downward inclination toward the northeast at an average gradient of approximately 3.5 percent. Total on-site relief was approximately 15 feet. On-site drainage was accomplished by sheetflow toward the northeast. It was noted on the eastern portion of the site the ground was sloping away from the building towards the associated streets and drive areas, and was likely a fill during previous construction.

At the time of the field study, utilities consisting of electric, telephone, gas, sewer, water, as well as other unknown underground and overhead lines, were observed to be present on and adjacent to the site. Underground service alert (USA) had marked the known utility lines on the site prior to the date of drilling. Due to the age of the structures and the fact the building was once utilized as a police station, unknown utilities likely traverse the site. The oldest photos available on Google Earth revealed the age of the buildings to be older than 1994.

At the time of the field study, vegetation was light and consisted of landscaped areas with seasonal native grasses, flowers and shrubs. It was noted the grass on the eastern portion of the site had not been watered for sometime. Small to medium sized, palms and oak trees were on site in addition to remnant trunks of trees that had been removed.

ENGINEERING GEOLOGIC ANALYSIS

Regional Geologic Setting

San Bernardino and its namesake valley lie very near the northern margin of the Peninsular Ranges Physiographic Province, one (1) of 11 provinces recognized in California. The physiographic provinces are topographic-geologic groupings of convenience based primarily on landforms, characteristic lithologies, and late Cenozoic structural and geomorphic history. The Peninsular Ranges encompass southwestern California west of the Imperial-Coachella Valley trough and south of the elevated terraces of the San Gabriel, San Bernardino, and Santa Monica Mountains. Most of the province lies outside of California, continuing south to include much of the Baja California Peninsula. The province is characterized by youthful, steeply sloped, northwest-trending, elongated ranges and intervening valleys. In gross aspect, average elevations across the province rise slowly to the east, usually culminating in abrupt escarpments near the eastern margin. Approaching the northern edge of the province, however, several anomalously flat and low basins stretch from the San Bernardino region to western Los Angeles as a result of fault junctures and tectonic interaction with the adjacent Transverse Ranges.

Structurally, the bulk of the Peninsular Ranges are composed of a number of relatively stable crustal blocks bounded by active strike-slip faults of the San Andreas transform system. Although some folding and minor faulting has occurred within the blocks, intense structural deformation and earthquake activity are mostly limited to block margins. The anomalous east-west trending San Bernardino Valley itself defines a small, irregularly-shaped block bounded by the San Andreas fault to the northeast, the San Jacinto fault to the southwest, and an arcuate set of sometimes obscure faults trending southwest through the Yucaipa, Redlands, and Loma Linda areas. The valley is not an erosional feature, but a deep structural basin that apparently continues to slowly subside in response to the transference of slip from the San Jacinto fault to the San Andreas fault.

The province contains a diverse array of metamorphic, sedimentary, volcanic, and intrusive igneous rocks. In general, the metamorphic rocks represent the highly altered host rocks for the emplacement of very large masses of granitic rock of varying composition. Closer to the coastline, younger rocks include thick sequences of marine and non-marine clastic sedimentary rocks of Mesozoic and Tertiary age, ranging from claystones to conglomerate. Inland, the province is dominated by crystalline basement rock, but the San Bernardino region also contains thick sequences of pre-Quaternary, continental, sedimentary rocks. These rocks are widely exposed in the hills bounding the south side of the San Bernardino Valley, and underlie the valley floor at depth.

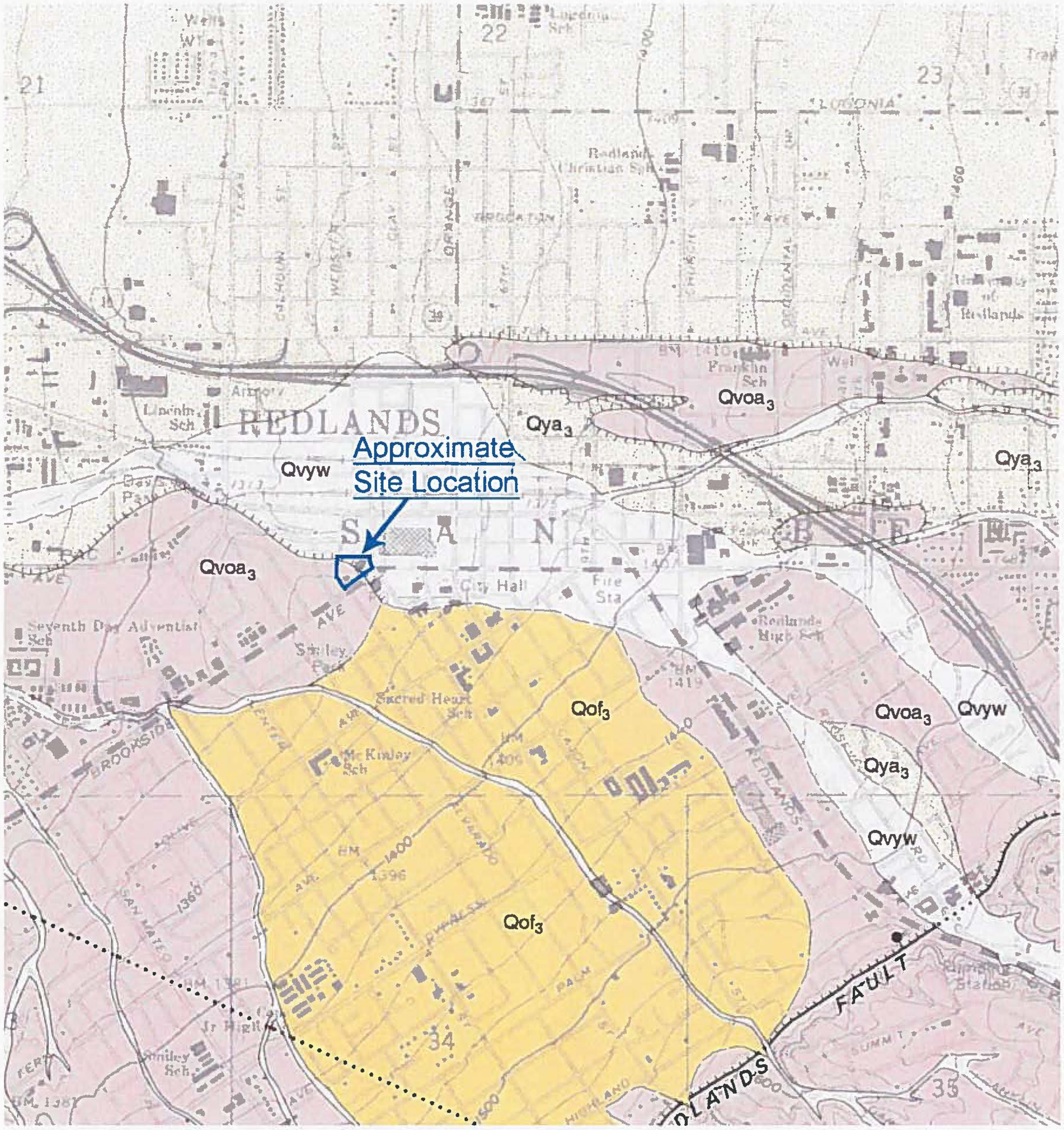
The site had been surficially mapped with two geologic units, a very young wash deposit (Qvwy) and an older axial valley deposit ($Qvoa_3$). The younger wash deposit was characterized as sand and gravel deposits in active washes, and are generally coarser in nature than the older axial valley deposits. The older axial valley deposits are generally alluvial in nature and were sandy clays to consolidated silt sand and gravel. Locally, the older axial valley deposits appear to interfinger with the young wash deposits. The general geology in the area of the subject site is shown on the 'Regional Geology Map,' Figure No. 2a, and the 'Regional Geology Map Legend,' Figure No. 2b.

Local Subsurface Conditions

Earth Materials Description: Presented as follows are brief descriptions of the earth materials encountered in the exploratory excavations. More detailed descriptions of encountered earth materials are presented on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in Appendix 'A' of this report. The earth material strata, as shown on the logs, represent conditions at the actual exploratory excavation locations. Other variations may occur beyond and/or between the excavations. Lines of demarcation between earth materials on the logs represented the approximate boundary between the material types; however, the transition may be gradual.

The earth materials encountered on the subject site during the field exploration were identified as Hot Mix Asphalt (HMA) concrete pavement over, man-made fill (af), over (Qvyw) very young wash deposits or ($Qvoa_3$) very old axial valley deposits.

Hot Mix Asphalt thickness was measured from Boring B-4 and two shallow percolation tests performed in the existing parking lot. The pavement was measured to be three (3) inches in thickness at the location of Boring B-4. The two shallow percolation borings excavated on the northern portion of the site had a measured asphalt thickness of 1.75 inches and 2.0 inches. No base materials were encountered within any of the borings excavated in the pavement.



Reference: California Department of Conservation, Division of Mines and Geology, 2003, Johnathon C. Matti, Douglas M. Morton, Brett F. Cox, Katherine J. Kendrick, and others, Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, Version 1.0, Open File Report 03-302, Scale: 1:24,000.

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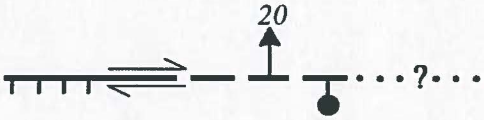
REGIONAL GEOLOGY MAP

By: AH	Date: 02/2018
Project No.: 1151-A17.1	Figure No.: 2a

Legend for Geologic Symbols and Units

Separates geologic-map units. Solid where meets map-accuracy standard; dashed where may not meet map-accuracy standard; dotted where concealed

Contact—Separates terraced alluvial units where younger alluvial unit is incised into older alluvial unit; hachures at base of slope, point toward topographically lower surface. Solid where meets map-accuracy standard; dashed where may not meet map-accuracy standard



Fault—Solid where meets map-accuracy standard; dashed where may not meet map accuracy standard. Dotted where concealed by mapped covering unit; queried where existence uncertain. Hachures indicate scarp, with hachures on downdropped block. Paired arrows indicate relative movement; single arrow indicates direction and amount of fault-plane dip. Bar and ball on down-thrown block.

Qvyw Qvyw - Very Young Wash Deposit, active (latest Holocene).

Qya3 Qya3 - Young Axial Valley Deposit, Unit 3 (middle Holocene).

Qvoa3 Qvoa3 - Very Old Axial Valley Deposit, Unit 3 (late to middle Pleistocene).

Qof3 Qof3 - Old Alluvial Fan Deposits, Unit 3 (late to middle Pleistocene).

Reference: **California Department of Conservation, Division of Mines and Geology**, 2003, Johnathon C. Matti, Douglas M. Morton, Brett F. Cox, Katherine J. Kendrick, and others, Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, Version 1.0, Open File Report 03-302, Scale: 1:24,000.

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REGIONAL GEOLOGY MAP LEGEND

By: AH

Date: 02/2018

Project No.: 1151-A17.1

Figure No.: 2b

Man made fill was encountered at all of the boring locations. The fill extended to a maximum depth of approximately 7.0 feet at the location of the exploratory excavations. The fill generally consisted of clayey fine to coarse sand which was red-brown in color, dry at the surface to moist with depth, and loose to medium dense in relative density. The in-place density tests indicated that the fill had an average relative compaction of approximately 85 percent. The fill is considered to be undocumented and unsuitable for support of structural fill and/or a building structure. The axial valley deposits and young wash deposits were encountered beneath the fill.

The very young wash deposits were encountered in the locations of the building demolition and proposed new construction within in borings B-1 through B-3. The young wash deposits generally consisted of silty fine to medium sands to silty fine to coarse sand with various amounts of gravel (SM), slightly silty fine to coarse sand (SP/SM) and gravelly fine to coarse sand with a variable amount of cobbles (SP). The materials ranged from red brown, pale brown, gray brown, and orange brown in color. The wash deposits extended to a depth of 42.0 feet at the location of B-1. Due to the contact between the wash deposits and the axial valley deposits some inter-fingering of clayey materials was also noted within boring B-4.

The old axial valley deposits were generally encountered in the existing parking lot on the western half of the site. The axial valley deposits generally consisted of silty fine to coarse sands with varying amounts of gravel and cobbles (SM), clayey fine to coarse sands with some gravel (SC). The axial valley deposit was generally reddish brown in color and dry near the surface to moist with depth. Locally, the axial valley deposits extended to depths in excess of 21.5 feet below the existing ground surface at the excavation location of B-4. A distinct but very gradual coarsening of the materials with depth was noted. Boring B-4 and the percolation test borings were terminated in the old axial valley deposit.

Existing Pavement Evaluation

The existing pavement on the eastern portion of the site differed from the pavement on the western portion of the site, and both areas showed signs of distress. The eastern half of the property had a thicker pavement section, and appeared to be older than the adjacent western parking lot. The eastern portion of the site contained speed bumps near the entrances and was highly alligatored. Subsurface materials were exposed in some areas and weeds had begun to grow from within the cracks. The western portion appeared to had been recently sealed, as shown in the Google Earth 10/21/2016 photograph. Large pavement cracks were evident on the surface and smaller cracks were beginning to appear around the larger cracks. No aggregate base materials were encountered or observed beneath the pavement in both locations.

Groundwater: Groundwater was not encountered in the exploratory excavations to the maximum depth explored of approximately 42.0 feet below existing ground surface at the boring locations at the time the field study was performed for this report.

A review of available groundwater contour maps indicated the historical minimum depth to groundwater in the general area has been approximately 75 feet (1985, Matti, J.C. and Carson, S.E.).

Depth to groundwater data for the site area was available through the **California Department of Water Resources** internet web site. The depth to groundwater in State Well No. 01S03W28J001S, located approximately 0.25 mile west of the site, was 193.1 feet on December 6, 2017. The surface elevation of this well is approximately 50 feet lower (topographically) than that of the site. Based on this information, the current depth to static groundwater beneath the site is estimated to be greater than 50 feet. Based on proposed lot grading and the inferred groundwater depths, groundwater should not be a factor for project design or long-term performance.

Surface Water: Surface water was not observed on the subject site at the time the field study was performed for this report.

Site Variations: Based on results of our subsurface exploration and experience, variations in the continuity and nature of surface and subsurface conditions should be anticipated. Due to uncertainty involved in the nature and depositional characteristics of earth materials at the site, care should be exercised in extrapolating or interpolating subsurface conditions between and beyond the exploratory excavation locations.

Groundwater observations were made in the exploratory excavations at times and under conditions stated on the boring logs. These data have been reviewed and interpretations made in the text in other sections of this report. However, it should be noted that fluctuations in levels of groundwater, springs, and/or perched water may occur due to variations in precipitation, temperature, and other factors.

Faulting and Regional Seismicity

The site is situated in an area of active and potentially active faults, as is most of metropolitan southern California. Active faults present a variety of potential risks to structures, the most common of which are strong ground shaking, dynamic densification, liquefaction, mass wasting, and surface rupture at the fault plane.

Generally speaking, the following four (4) factors are the principal determinants of seismic risk at a given location:

- Distance to seismogenically capable faults.
- The maximum or "characteristic" magnitude earthquake for a capable fault.
- Seismic recurrence interval, in turn related to tectonic slip rates.
- Nature of earth materials underlying the site.

Surface rupture represents the primary potential hazard to structures built in an active fault zone. A review of official maps delineating State of California earthquake fault zones (**California Department of Conservation, Division of Mines and Geology**, Effective January 1, 1977, *State of California Special Studies Zones, Redlands Quadrangle, Revised Official Map*, Scale 1:24,000) indicated the site is not located within a zone of mandatory study for active faulting. In addition, the site is not located within a zone of mandatory study for active faulting per the **San Bernardino County Planning Department**, *San Bernardino County Land Use Plan, GENERAL PLAN, Geologic Hazard Overlays*, Sheet FH31 C Redlands, Plot Date: 03/09/2010, Scale: 1:14,400 (http://www.co.san-bernardino.ca.us/landuseservices/general). Reviews of other geology maps of the Redlands region revealed no known faults that cross the subject site. Additionally, no known active faults trend toward the subject property.

The most recent, large earthquake that occurred in close proximity to the subject property was the June 28, 1992 Big Bear earthquake. The epicenter of this quake was located approximately 37.5 kilometers northeast of the subject property at Latitude: 34.2030° North, Longitude: 116.8270° West. The Big Bear quake had a measured magnitude of 6.7, had no surface rupture, and is believed to have occurred on a blind thrust fault, the exact location and geometry of which currently are unknown. Several aftershocks also were centered very near the epicenter, including a magnitude 5.6 aftershock.

Ground shaking is judged to be the primary hazard most likely to affect the site, based upon proximity to seven (7) regionally significant active faults as listed in the following table. Other significant fault zones, including the Pinto Mountain fault, the Chino-Central Avenue fault, and several zones in the high desert area are located at distances exceeding 40 kilometers from the site. Greater distances, lower slip rates, and lesser maximum magnitudes indicate much lower risk to the site from the latter fault zones than the seven (7) closest faults including the regionally significant San Andreas fault. Characteristics of the major active fault zones selected for inclusion in analysis of strong ground shaking are listed in the following table:

Fault Zone[1]	**Distance (km)**[2] **/ Direction from Site**	**Fault Length (km)**[1]	**Slip Rate (mm/yr)**[1]	**Reference Earthquake $M_{(Max)}$**[1]	**Fault Type**[1]
San Jacinto (San Jacinto Valley Segment) (rl-ss)	6.1 / Southwest	43±4	12.0±6.0	6.9	A
San Jacinto (San Bernardino Segment) (rl-ss)	6.4 / Southwest	36±4	12.0±6.0	6.7	A
San Andreas (San Bernardino Segment) (rl-ss)	9.4 / Northeast	103±10	24.0±6.0	7.5	A
North Frontal (Western Segment) (r, 45 S)	20.3 / North	51±5	1.0±0.5	7.2	B
Cleghorn (ll-ss)	25.1 / Northwest	25±3	3.0±2.0	6.5	B

Fault Zone[1]	Distance (km)[2] / Direction from Site	Fault Length (km)[1]	Slip Rate (mm/yr)[1]	Reference Earthquake $M_{(Max)}$[1]	Fault Type[1]
Cucamonga (r,45 N)	27.0 / Northwest	28±3	5.0±2.0	6.9	B
North Frontal (Eastern Segment) (r,45 S)	39.3 / North Northeast	27±3	0.5±0.3	6.7	B

1. **Tianqing, C.W., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J.**, June 2003, *The Revised 2002 California Probabilistic Seismic Hazards Maps (Appendix A - 2002 California Fault Parameters).*
   California Department of Conservation, Division of Mines and Geology, 1996, *Probabilistic Seismic Hazard Assessment for the State of California*, DMG Open-File Report 96-08.
2. **Blake, Thomas F.**, 2000, *Preliminary Fault-Data for EQFault, EQSearch and FriskSP* and **Blake, Thomas, F.**, *Computer Services and Software, Users Manuals, FriskSP v. 4.00, EQSearch v. 3.00, and EQFault v. 3.00.*
3. Fault Geometry: (ss) strike slip; (r) reverse; (n) normal; (rl) right lateral; (ll) left lateral; (O) oblique; (45 N) direction.

Probabilistic seismic hazard maps and data files prepared by the **California Geological Survey (CGS)** determine ground motions with a 10-percent probability of being exceeded in the next 50 years (475 years mean return time) as a fraction of the acceleration due to gravity for peak ground acceleration (PGA) and spectral accelerations (Sa) for short and moderately long periods, 0.2 seconds and 1.0 second, respectively. This data was available at the **CGS** 'PSHA Ground Motion Interpolator (2008)' web site (http://www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html). The values are presented in the following table for reference:

GROUND MOTION*	SITE ACCELERATION Site Class D**
PGA	0.633g
Sa @ 0.2 Sec.	1.348g

GROUND MOTION*	SITE ACCELERATION Site Class D**
Sa @ 1.0 Sec.	0.928g

* 10-percent probability of being exceeded in the next 50 years (475 years mean return time).
** Shear Wave Velocity of 274 m/sec was assumed for the on-site materials.

California Geological Survey (CGS) assign a 2-percent likelihood that a Peak Horizontal Ground Acceleration (PGA) of approximately 0.995g will occur at this site within the next 50 years (2,475 years mean return time). This data was available at the **CGS** 'PSHA Ground Motion Interpolator (2008)' web site (http://www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html).

Actual shaking intensities at the site from any seismic source may be substantially higher or lower than estimated for a given earthquake magnitude, due to complex and unpredictable effects from variables such as:

- Near-source directivity effects.
- Direction, length, and mechanism of fault rupture (strike-slip, normal, reverse).
- Depth and consistency of unconsolidated sediments.
- Topography.
- Geologic structure underlying the site.
- Seismic wave reflection, refraction, and interference.

Secondary Seismic Hazards

Secondary hazards include induced landsliding or mass wasting, liquefaction, flooding (from ruptured tanks and reservoirs, surface oscillations in larger lakes, or seismic sea waves), and subsidence as a result of soil densification. Landsliding and liquefaction susceptibility maps have been prepared for much of coastal Los Angeles and Orange County, California by the **CGS**. However, this area of San Bernardino County, California is not presently scheduled for mapping by the State.

Landslide: The subject site is not located within a designated area as having a landslide susceptibility per **San Bernardino County Planning Department**, *San Bernardino County Land Use Plan, GENERAL PLAN, Geologic Hazard Overlays*, Sheet FH31 C Redlands Plot Date: 03/09/2010, Scale: 1:14,400 (http://www.co.san-bernardino.ca.us/landuseservices/general).

Due to the flat-lying nature of the site, on-site landsliding or debris flows sourced from higher elevations should not be considered to be a geologic constraint at this site.

Liquefaction: Liquefaction is a phenomenon in which cohesionless, saturated, fine-grained sand and sandy silt soils lose shear strength due to ground shaking. The subject site is not located within a designated area as having a liquefaction potential per **San Bernardino County Planning Department**, *San Bernardino County Land Use Plan, GENERAL PLAN, Geologic Hazard Overlays*, Sheet FH31 C Redlands, Plot Date: 03/09/2010, Scale: 1:14,400 (http://www.co.san-bernardino.ca.us/landuseservices/general).

It is our opinion that liquefaction potential at the subject site is very low due to an estimated depth of groundwater of 50 feet or greater beneath the existing ground surface on the site.

Seismically Induced Subsidence: Loose sandy soils subjected to moderate to strong ground shaking can experience settlement. Experience from the Northridge Earthquake indicates that structural distress can result from such seismic settlement. Based upon the results of this study, the subject site is underlain at depth by dense to very dense or hard, consolidated deposits that should not be prone to a significant degree of seismic settlement. Where applicable, loose, near-surface, young wash deposits, alluvial soils and undocumented fills should be removed and recompacted to uniform high densities to mitigate both settlement and consolidation potentials.

Lateral Spreading: Lateral spread is the most pervasive type of liquefaction-induced ground failure. Lateral spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shear stress biases on essentially horizontal ground. During lateral spread, blocks of mostly intact, surficial earth material displace downslope or towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures or a graben at the head of the failure, shear deformations along the side margins, and compression or buckling of the earth material at the toe. The amount of lateral displacement typically ranges from a few centimeters to several meters and can cause considerable damage to engineered structures and lifelines.

A formal lateral spread analysis was not performed as part of this study. The lateral spread potential of the subject site is not considered to be a geologic hazard for the proposed structure on the subject property.

Seiching: Seiching involves an enclosed body of water oscillating due to ground shaking, usually following an earthquake. Lakes and water towers are typical bodies of water affected by seiching. However, the site does not appear to be within the influence of large bodies of water and, as such, seiching should not be considered a geologic hazard for the development of the subject site.

Tsunamis: Because of the inland geographic location of the site, tsunamis are not considered a geologic hazard for the development of the subject site.

Lurching: Lurching is a phenomena in which ground cracking and/or secondary faulting occurs as a result of ground shaking. Generally, lurching primarily occurs in the immediate vicinity of faulting or within typical building setback zones or "No Human Occupancy" zones. No evidence of faulting was encountered on the site and although the potential for lurching cannot be entirely ruled out, the likelihood for lurching to impact the site is considered to be low.

OTHER GEOLOGIC HAZARDS

Flooding

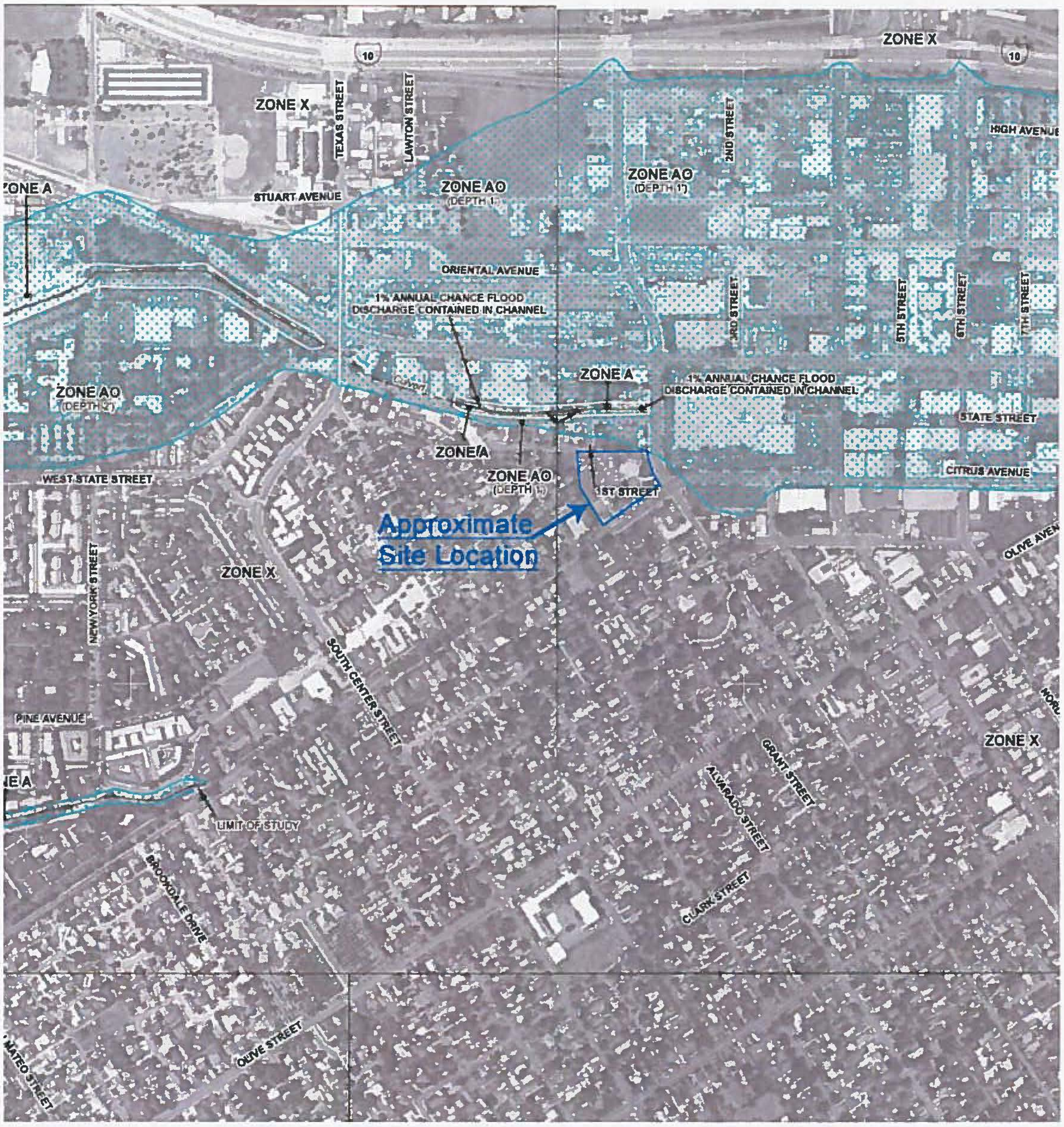
The subject site is not located within a designated area as having a flooding potential per **San Bernardino County Planning Department**, *San Bernardino County Land Use Plan, GENERAL PLAN, Hazard Overlays*, Sheet FH31 B Redlands, Plot Date: 03/09/2010, Scale: 1:14,400 (http://www.co.san-bernardino.ca.us/landuseservices/general).

Flood Insurance Rate Maps (FIRM) were compiled by the **Federal Emergency Management Agency (FEMA)** for the Flood Insurance Program and are available for most areas within the United States at the **FEMA** web site (http://msc.fema.gov/). The attached 'FEMA Flood Hazard Map' and 'FEMA Flood Hazard Map Legend,' Figure Nos. 3a and 3b, respectively, are based on FIRMs provided by **FEMA** and are specific to the area around the subject site. The 'FEMA Flood Hazard Map,' Figure 3a, indicates that the site is located within 'Zone X' (an area of 0.2-percent annual chance flood; an area of 1-percent annual chance flood (100 year flood) with average depths of less than 1.0 foot or with drainage areas less than 1.0 square mile; and an area protected by levees from the 1-percent annual chance flood).

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The conclusions and recommendations presented in this report are preliminary since a grading plan, the type of structure construction, structural loads, finish floor elevations, etc. were not available and are, in part, based on information provided to this firm, the results of the field and laboratory data obtained from four (4) exploratory excavations located on the subject property, experience gained from work conducted by this firm on projects within the general vicinity of the subject site, the project description and assumptions presented in the 'Project Description / Proposed Development' section of this report, engineering analyses, and professional judgement. Based on a review of the field and laboratory data and the engineering analysis, the proposed development is feasible from a geotechnical / geologic standpoint. The subject property can be developed without adverse



0 800 1,600

Scale Approximate in Feet

N

Reference: **U.S. Federal Emergency Management Administration (FEMA)**, Maps Revised August 28, 2008, *Flood Insurance Rate Map*, Map Nos. 06071C 8712 H, 06071C 8716 H. Site specific information obtained through FEMA website, Map Service Center (http://msc.fema.gov/).

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FEMA FLOOD HAZARD MAP

By: AH

Date: 02/2018

Project No.: 1151-A17.1

Figure No.: 3a

LEGEND

▒ SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

▨ FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

▒ OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

☐ OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

▧ COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

▧ OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

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FEMA FLOOD HAZARD MAP LEGEND

By: AH
Date: 02/2018
Project No.: 1151-A17.1
Figure No.: 3b

impact onto or from adjoining properties providing the recommendations contained within this report are adhered to during project design and construction.

The average in-situ moisture contents and in-situ dry densities of the upper 5.0 feet of the near-surface alluvial materials on the subject site suggests that the soils have an average relative compaction of less than 85 percent.

The field observations indicate that up to 7.0 feet of material present on the subject site was an undocumented fill material. The artificial fills on the site are also considered loose and compressible. The man-made fills are not considered suitable for the support of structural fills, foundations, slab-on-grade floor slabs, hardscape, and/or pavement.

The laboratory tests suggest that the alluvial materials on the site are subject have a 0.7 to 8.1 collapse potential if they should become saturated while under a load (hydroconsolidation). This was emphasized by the laboratory consolidation tests that collapsed from 0.7 to 8.1 percent under a load of 1,600 psf when water was added during the testing procedure. An additional consolidation test was run on a sample at the same depth to verify the amount of hydro-collapse. Theoretically, a 0.7 to 8.1 percent collapse of 3.0 feet of soil beneath a footing and/or structural fills would result in an additional settlement of approximately 0.25 to 2.92 inches, respectively, beyond what would normally be anticipated. Greater collapse potentials and/or a deeper zone of saturation would result in a larger settlement. Depending on the uniformity of the depth and the area of saturation, the settlement may not be uniform throughout the structure and/or fill area.

Some remedial grading consisting of removals and replacement will have to be performed within loose, compressible, artificial fill, and moisture sensitive loose, near-surface alluvium in the area of proposed structural fills, structures, exterior hardscapes, and/or pavement.

The actual conditions of the near-surface supporting material across the site may vary. The nature and extent of variations of the surface and subsurface conditions between the exploratory excavations may not become evident until construction. If variations of the material become evident during construction of the proposed development, **HGI** should be notified so that the project Geotechnical / Geologic Consultant can reevaluate the characteristics of the material and the conclusions and recommendations of this report, and, if needed, make revisions to the conclusions and recommendations presented herein.

Specific recommendations for site grading, foundations, slab support, pavement design, are presented in the subsequent paragraphs.

SITE PREPARATION RECOMMENDATIONS

General

The grading recommendations presented in this report are intended for: 1) the rework of unsuitable, near-surface, fill and alluvial earth materials to create a uniformly thick engineered building pads and satisfactory support for exterior hardscape (i.e., sidewalks, patios, etc.) and pavement; and 2) the use of shallow foundation system and concrete slabs cast on-grade for the proposed structures.

If hardscape and pavement subgrade earth materials are prepared at the time of grading of the building sites, and the improvements are not constructed immediately, additional observations and testing of the subgrade earth material will have to be performed to locate areas which may have been damaged by construction traffic, construction activities, and/or seasonal wetting and drying.

The additional observations and testing should be performed before placing aggregate base material, Hot Mix Asphalt (HMA) concrete, and/or Portland Cement concrete (PCC) in those areas.

The grading should be performed in accordance with the recommendations presented in this report. We recommend that **HGI**, as the Geotechnical Engineer / Geologist of Record, be retained by the owner of the proposed project to observe the excavation and grading operations, foundation preparation, and test the compacted fill and utility trench backfill. If **HGI** were not selected to perform the required observation and testing of earthwork construction, **HGI** would cease to be the Geotechnical Consultant of Record for the project. A pregrading conference should be held at the site with representatives of the owner, the grading contractor, the City of Redlands, the Civil Engineer, and a representative of **HGI** in attendance. Special grading procedures and/or concerns can be addressed at that time.

Earthwork observation services allow the testing of only a small percentage of the fill placed at the site. Contractual arrangements with the grading contractor by the project owner should contain the provision that he is responsible for excavating, placing, and compacting fill in accordance with the recommendations presented in this report and the approved project grading plans and specifications. Observation by the project Geotechnical / Geologic Consultant and/or his representatives during grading should not relieve the grading contractor of his responsibility to perform the work in accordance with the recommendations presented in this report and the approved project plans and specifications.

The following recommendations may need to be modified and/or supplemented during grading as field conditions require.

Final Grading Plan Review

The project Civil Engineer should review this report, incorporate critical information on to the grading plan and/or reference this geotechnical / geologic study, by Company Name, Project No., Report No., and report date, on the grading plan. Final grading plans should be reviewed by **HGI** when they become available to address the suitability of our grading recommendations with respect to the proposed improvements.

Clearing and Grubbing

Debris from the demolition of the existing structure, grasses, weeds, brush, and other deleterious materials should be removed from the proposed building, exterior hardscape and pavement areas and areas to receive structural fill before grading is performed. Any organic material and miscellaneous / demolition debris should be legally disposed of off site. Any topsoil or highly organic soils encountered should be stripped and stockpiled for use on finished grades in landscape areas or exported from the site. Disking or mixing of organic material into the earth materials proposed to be used as structural fill should not be permitted.

Man-made objects encountered (i.e., septic tanks, leach lines, irrigation systems, underground utilities, old foundations, construction debris, etc.) should be overexcavated, exported from the site, and legally disposed of off site. Cesspools or seepage pits, if encountered (none were encountered during this study), should be abandoned and capped according to directions and supervision of San Bernardino County Department of Health, the State of California, and/or the appropriate governmental agency procedures which has jurisdiction over them before fill and/or pavement is placed over the area. If no procedures are required by the Health Department or if the following recommendations are more stringent, the cesspool or seepage pit should be pumped free of any liquid and filled with a low strength sand cement slurry to an elevation 5.0 feet below the final site grade in the area. The upper 5.0 feet of the cesspool or seepage pit should be excavated and the area backfilled with a properly compacted fill material. The location of the cesspool or seepage pit should be surveyed and plotted on the final 'As-Graded' plan prepared by the project Civil Engineer.

Wells, if encountered, should be abandoned and capped according to directions and supervision of San Bernardino County Department of Health, the State of California, and/or the appropriate governmental agency procedures which has jurisdiction over the well before fill and/or pavement is placed over the area.

Excavation Characteristics

Excavation and trenching within the subject property to the depths anticipated for the proposed development is anticipated to be relatively easy in the near-surface undocumented fills and alluvial materials on the subject site and should be accomplished with conventional earth-moving equipment since the drill rig equipped with flight augers was able to penetrate to the indicated depths. Materials were not encountered or are anticipated at shallow depths that would require heavy ripping or blasting to excavate. It is not anticipated that a significant amount of oversized rock material (i.e., 12 inches in greatest dimension) will be generated during the removal and replacement process within the alluvial materials which will require special handling during the development of the site.

Suitability of On-Site Materials as Fill

In general, the on-site earth materials present below any topsoil and/or highly organic materials are considered satisfactory for reuse as fill. Fill materials should be free of significant amounts of organic materials and/or debris and should not contain rocks or clumps greater than 12 inches in maximum dimension. It is noted that the average in-situ moisture content of the near-surface fill and alluvial earth materials on the subject site at the time this field study was performed was below the optimum moisture content for the on-site materials and that moisture will have to be added to the on-site earth materials if the earth materials are to be used as compacted fill material in the near future. No significant amount of oversized rock materials are anticipated to be generated from the cuts performed in the local materials.

The existing HMA concrete and PCC concrete that are located on the site can be crushed down to a particle size of 3.0 inches or less in maximum dimension and incorporated into the fills required to achieve the finish grades for the subject development.

Removal and Recompaction

Uncontrolled or undocumented fills and/or unsuitable, loose, or disturbed near-surface alluvial earth material in proposed areas which will support structural fills, structures (i.e., buildings, decorative block walls, retaining walls, trash enclosure walls, etc.), exterior hardscape (i.e., sidewalks, patios, curb / gutters, etc.), and pavement should be prepared in accordance with the following recommendations for grading in such areas. If overexcavation of undocumented fill or moisture sensitive, collapsible earth materials is elected not to be performed in hardscape, curb / gutter, pavement, and decorative block wall or fence areas, penetration of irrigation water with time may cause some settlement and distress to the improvements in those areas. The cost of the additional grading verses the risk of distress and cost of repairs to the structures needs to be evaluated by the project owner.

- The near-surface undocumented fill and the loose, collapsible, near-surface alluvial materials on the site are recommended to be overexcavated and recompacted. Based upon our exploratory excavations borings and laboratory test results, we anticipate that the overexcavation will extend to a depth of approximately 11 feet below existing ground surface and to a uniform elevation within the horizontal limits of the overexcavation in the areas which will receive structural fill, building structures, retaining walls, trash enclosure walls, and decorative concrete block walls. A relative compaction of 85 percent or greater should be obtained in the exposed earth material at the overexcavation depth prior to performing any scarification, moisture conditioning, and recompaction. If 85 percent relative compaction is not present, the overexcavation should be deepened until a minimum of 85 percent relative compaction is present. Moreover, the depth of the overexcavation within the perimeter of the proposed structures should be to a uniform elevation throughout the limits of the structures. It is noted that fill placed to construct slopes and/or support sidewalks, patios, retaining walls, block walls, driveways, and pavement are considered to be structural fill.

- In the proposed exterior hardscape (i.e., sidewalks, patio slabs, etc.), and pavement areas where structural fill will not be placed or cuts are proposed, the existing near-surface earth materials need only be processed to a depth of 6.0 to12 inches below existing site grades or proposed subgrade elevation, whichever is deeper unless old, undocumented fill materials are encountered at exposed grades. If undocumented fills are encountered, they will need to be overexcavated and properly compacted fill replaced to achieve proposed grades.

  Due to the collapsible nature of the near-surface alluvial earth materials on the subject site, if overexcavation and replacement is not performed under the exterior concrete slabs, hardscape, pavement, curb / gutters, etc,, there is a risk of settlement and vertical differential movement if the subgrade earth materials are allowed to become saturated. Therefore, proper drainage should be established away from such improvements and minimal precipitation or irrigation water allowed to percolate into the earth materials adjacent to the exterior concrete hardscape, pavement, curbs / gutters, etc.

- Additional overexcavation will need to be performed in areas where the exposed subgrade can not be properly processed and recompacted per the following recommendations presented in this section of this report.

- The limits of overexcavation for the building pads should extend to a distance of 5.0 feet or to the depth of the overexcavation beneath the finish pad grade for the structure, whichever is greater, beyond the structure perimeter or footing edges. The limits of overexcavation for the decorative concrete block perimeter wall footings and/or retaining wall footings should extend to a distance of 4.0 feet beyond the footing edges or to the depth of the overexcavation beneath the footing grade, whichever is greater. The limits of processing or overexcavation for exterior hardscape, curb / gutter, and pavement areas should extend to a distance of 2.0 feet beyond the edge of the exterior hardscape, curb / gutter, or pavement, or to the depth of the overexcavation beneath the finish subgrade elevation, whichever is greater.

- It is noted that localized areas, once exposed, may warrant additional overexcavation for the removal of existing undocumented fills, loose, near-surface earth material, porous, moisture sensitive alluvial earth materials, and subsurface obstructions and/or debris which may be associated with the existing structure or past usage of the site or may be not have been located during the field study performed for this report. Actual depths of removals and the competency of the exposed overexcavation bottoms should be determined by the project Geotechnical / Geologic Consultant and/or his representative during grading operations at the time they are exposed and before scarification and recompaction or the placement of fill.

- Any underground fuel and waste oil storage tanks and contaminated material, if present, should be removed in accordance with County of San Bernardino Department of Environmental Health, Hazardous Materials Management Divisions criteria and procedures. The excavations should be cleaned of loose materials. It is recommended that tank removal excavations with depths of 5.0 feet or deeper be cut back according to the 'Temporary Construction Cut' section of this report or be properly shored during construction.

- The exposed overexcavation bottom surfaces should be scarified to a depth of 6.0 to 12 inches, brought to optimum moisture content to 3.0 percent above optimum moisture content, and compacted to 90 percent or greater relative compaction before placement of fill. Maximum dry density and optimum moisture content for compacted materials should be determined according to current ASTM D1557 procedures. The scarification and recompaction of the exposed overexcavation bottoms in alluvial materials may be deleted upon approval by the project Geotechnical / Geologic

Consultant, and/or his representative when in-place density test results in the undisturbed alluvial materials indicate a relative compaction of 90 percent or greater.

Import Material

Import fill should be 'Non-Expansive' as defined in Section 1803.5.3, 'Expansive Soil,' in the 2016 CBC (i.e., Expansion Index ≤ 20) as determined by current ASTM D4829 procedures and have strength parameters equivalent to or greater than the on-site earth materials. Import fill material should be approved by the project Geotechnical / Geologic Consultant prior to it being brought on-site.

The existing pavement materials can be crushed down to a maximum practical size of 3.0 inches and incorporated into the fills materials required to achieve the finish subgrade elevations for the project.

Fill Placement Requirements

Fill material, whether on-site material or import, should be approved by the project Geotechnical / Geologic Consultant and/or his representative before placement. Fill material should be free from vegetation, organic material, debris, and oversize material (i.e., 3 inches in maximum dimension). Approved fill material should be placed in horizontal lifts not exceeding 6.0 to 12 inches in compacted thickness or in thicknesses the grading contractor can demonstrate that he can achieve adequate compaction and watered or aerated to obtain optimum moisture content to 3.0 percent above optimum moisture content. Each lift should be spread evenly and should be thoroughly mixed to ensure uniformity of earth material moisture. Fill soils should be compacted to 90 percent or greater relative compaction. Maximum dry density and optimum moisture content for compacted materials should be determined in accordance with current ASTM D1557 procedures.

Compaction Equipment

It is anticipated that the compaction equipment to be used for the project will include a combination of rubber-tired, track-mounted, sheepsfoot, and/or vibratory rollers to achieve compaction. Compaction by rubber-tired or track-mounted equipment, by itself, may not be sufficient. Adequate water trucks, water pulls, and/or other appropriate equipment should be available to provide sufficient moisture and dust control. The actual selection of equipment and compaction procedures are the responsibility of the contractor performing the work and should be such that uniform compaction of the fill is achieved.

Shrinkage, Bulking, and Subsidence

There will be a material loss due to the clearing and grubbing operations. The following values are exclusive of losses due to clearing, grubbing, or the removal of other subsurface features and may vary due to differing conditions within the project boundaries and the limitations of this study.

Volumetric shrinkage of the near-surface earth materials (i.e., undocumented fill and near-surface alluvium) on the subject site that are excavated and replaced as controlled, compacted fill should be anticipated. It is estimated that the average shrinkage of the near-surface earth materials within the upper 10 feet of the site which will be removed and replaced will be approximately 10 to 16 percent, based on fill volumes when compacted to 90 to 95 percent of maximum dry density for the earth material type based on current ASTM D1557 procedures. For example, a 10 percent shrinkage factor would mean that it would take 1.10 cubic yards of excavated material to make 1.0 cubic yard of compacted fill at 90 percent relative compaction. A higher relative compaction would mean a larger shrinkage value.

A subsidence factor (loss of elevation due to compaction of existing undocumented fill and/or the near-surface alluvial earth materials in-place) of 0.09 to 0.14 foot per foot of compacted earth material should be used in areas where the existing earth materials are compacted in-place to 90 to 95 percent relative compaction and to a depth of 12 inches.

Subsidence of the site due to settlement from the placement of less than 3.0 feet of fill (not including the depth of overexcavation and replacement) during the planned grading operation is expected to be minimal.

Although the above values are only approximate, they represent the recommended estimate of some of the respective factors to be used to calculate lost volume that will occur during grading.

Abandonment of Existing Underground Lines

Abandonment of existing underground irrigation, utility, or pipelines, if present within the zone of construction, should be performed by either excavating the lines and filling in the excavations with documented, properly compacted fill or by filling the lines with a low strength sand / aggregate / cement slurry mixture. Filled lines should not be permitted closer than 3.0 feet below the bottom of proposed footings and/or concrete slabs on-grade. The lines should be cut off at a distance of 5.0 feet or greater from the area of construction. The ends of the lines should be plugged with 5.0 feet or more of concrete exhibiting minimal shrinkage characteristics to prevent water or fluid migration into or from the lines. Capping of the lines may also be needed if the lines are subject to line pressures. The slurry should consist of a fluid, workable mixture of sand, aggregate, cement, and water. Plugs should be placed at the ends of the line prior to filling with the slurry mixture. Cement should be Portland cement conforming to current ASTM C150 specifications. Water used for the slurry mixture should be free of oil, salts, and other impurities which would have an adverse effect on the quality of the slurry. Aggregate, if used in the slurry, mixture should meet the following gradation or a suitable equivalent:

SIEVE SIZE	PERCENT PASSING
1.5"	100
1.0"	80-100
3/4"	60-100
3/8"	50-100
No. 4	40-80
No. 100	10-40

The sand, aggregate, cement, and water should be proportioned either by weight or by volume. Each cubic yard of slurry should not contain less than 188 pounds (2.0 sacks) of cement. Water content should be sufficient to produce a fluid, workable mix that will flow and can be pumped without segregation of the aggregate while being placed. The slurry should be placed within 1.0 hour of mixing. The contractor should take precautions so that voids within the line to be abandoned are completely filled with slurry.

Local ordinances relative to abandonment of underground irrigation, utility, or pipelines, if more restrictive, supersede the above recommendations.

Temporary Roads

Temporary roads created during grading should be removed in their entirety or replaced as documented compacted fill as part of the rough grading of the tract.

Protection of Work

During the grading process and prior to the completion of construction of permanent drainage controls, it is the responsibility of the grading contractor to provide good drainage and prevent ponding of water and damage to the in progress or finished work on the site and/or to adjoining properties.

Observation and Testing

During grading, observation and testing should be conducted by the project Geotechnical / Geologic Consultant and/or his representatives to verify that the grading is being performed according to the recommendations presented in this report. The project Geotechnical / Geologic Consultant and/or his representative should observe and test the overexcavation bottoms and the placement of fill and should take tests to verify the moisture content, density, uniformity and degree of compaction obtained. The contractor should notify the project Geotechnical / Geologic Consultant when cleanout and/or overexcavation bottoms are ready for observation and prior to scarification and recompaction. Typically, one (1) in-place density test should be performed for every 2.0 vertical feet of fill material, or one (1) test for every 500 cubic yards of fill, which ever requires the greater number of tests. In-place density and moisture content tests should be performed during the placement of the fill materials during the grading operations in general accordance with the following current ASTM test procedures:

> Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth) - ASTM D6938.
>
> Test Method for Density and Unit Weight of Soil in Place by Sand Cone Method - ASTM D1556.
>
> Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock - ASTM D2216.

> Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method - ASTM D4959.
>
> Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method - ASTM D4643.

Where testing demonstrates insufficient density, additional compaction effort, with the adjustment of the moisture content when needed, should be applied until retesting shows that satisfactory relative compaction has been obtained. The results of observations and testing services should be presented in a formal 'Grading Report' following completion of the grading operations. Grading operations undertaken at the site without the project Geotechnical / Geologic Consultant and/or his representative present may result in exclusions of the affected areas from the grading report for the project. The presence of the project Geotechnical / Geologic Consultant and/or his representative will be for the purpose of providing observations and field testing and will not include supervision or directing of the actual work of the contractor or the contractor's employees or agents. Neither the presence and/or the non-presence of the project Geotechnical / Geologic Consultant and/or his field representative nor the field observations and testing will excuse the contractor for defects discovered in the contractor's work. If **HGI** does not perform the observation and testing of the earthwork for the project and is replaced as Geotechnical / Geologic Consultant of record for the project, the work on the project should be stopped until the replacement Geotechnical / Geologic Consultant has reviewed the previous reports and work performed for the project, agreed in writing to accept the recommendations and prior work performed by **HGI** for the subject project, or has performed their own studies and submitted their revised recommendations. If **HGI** were not selected to perform the required observation and testing of earthwork construction, **HGI** would cease to be the Geotechnical Consultant of Record for the project.

Earth Material Expansion Potential

The preliminary expansion potential of the on-site earth materials is discussed in the subsequent foundation and floor slab recommendation sections of this report. Upon completion of grading for the building pad areas, near-surface samples should be obtained for expansion potential testing to verify the preliminary expansion test results and the foundation / slab-on-grade recommendations presented in this report.

Earth Material Corrosion Potential

The preliminary corrosion potential of the on-site earth material is discussed in the subsequent corrosion recommendation sections of this report. Upon completion of grading for the building pad areas, near-surface samples should be obtained for corrosion potential testing to verify the preliminary chemical test results and the recommendations presented in this report for protection of concrete and bare metal which will be in direct contact with the on-site earth materials and to present preliminary evaluation of the potential for corrosion of bare metal, if desired, which will be in direct contact with the on-site earth materials.

2016 CBC SEISMIC DESIGN CRITERIA

Per the **California Building Standards Commission**, *2016 California Building Code* (CBC), California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Section 1613, 'Earthquake Loads,' the followings coefficients and factors relevant to seismic mitigation and design for new construction include:

- **Site Class**
  Categorizing the upper 30 meters (±100 ft.) of earth materials into one (1) of the Site Classes 'A,' 'B,' 'C,' 'D,' 'E,' and 'F' that are based on average shear wave velocities, Standard Penetration Test blow counts, or undrained shear strength.

- **Occupancy Category**
  Relationship between the number of lives placed at risk by a failure of the structure as determined from Figure C1-1, 'Approximate Relationship between Number of Lives Placed at Risk by a Failure and Occupancy Category,' in Chapter C1 of ASCE 7-10.

- **Mapped, Maximum Considered Earthquake (MSC), 5.0 Percent Damped, Spectral Response Acceleration Parameters at Short Period and at 1-Second Period**
  Mapped, Maximum Considered Earthquake (MSC), 5.0 percent damped, spectral response acceleration parameters at short period (0.2 second) and at long period (1-second), S_s and S_1, respectively, for Site Class 'B' are determined from Java Ground Motion Parameter Calculator - Version 5.0.9a available at the USGS web site (http://earthquake.usgs.gov/research/hazmaps/design/).

- **Site Coefficients**
  Short period site coefficient (at 0.2 second period), F_a, and long-period site coefficient (at 1.0 second period), F_v, are based on 'Site Class' and the 'Mapped Spectral Response Acceleration at Short Period and at 1-Second Period,' S_a and S_1, respectively.

- **Seismic Design Category**
  A classification assigned to a structure based on its 'Risk Category' and the severity of the design earthquake ground motion at the site (i.e., Short Period Response Acceleration (S_{DS}) and Long Period Response Acceleration (S_{D1}) Parameters).

Based on our understanding of local geologic conditions, the 'Site Class' judged applicable to this site is 'D', with a soil profile name of 'Stiff Soil' per Table 20.3-1, 'Site Classification,' in Chapter 20 of ASCE 7-10 with an average Shear Wave Velocity of 600 to 1,200 feet/second (ft./s) or an average Standard Penetration Test value of 15 to 50 blows per foot of penetration in the upper 100 feet (30.48 m) of the site.

The following table presents supplemental coefficients and factors relevant to seismic mitigation and design for new construction built according to the 2016 CBC based on a 2-percent probability of being exceeded in the next 50 years (2,475 years mean return time).

Site Location	Latitude: 34.0556° N Longitude: 117.1860° W
Occupancy Category[1]	I, II, or III
Site Class[2]	D
Mapped, Maximum Considered Earthquake (MCE), 5.0 Percent Damped, Spectral Response Acceleration Parameter at Short Period (S_s)[3] (0.2 Second) for Site Class 'D.'	1.717
Mapped, Maximum Considered Earthquake (MCE), 5.0 Percent Damped, Spectral Response Acceleration Parameter at 1-Second (S_1)[3] for Site Class 'D.'	0.779
Site Coefficients (F_a)[3] for Site Class 'D.'	1.0
Site Coefficients (F_v)[3] for Site Class 'D.'	1.5
The MSC, 5.0 Percent Damped, Spectral Response Acceleration Parameter at Short Periods Adjusted for Site Class 'D' Effects (S_{MS})[3].	1.717
The MSC, 5.0 Percent Damped, Spectral Response Acceleration Parameter at 1-Second Adjusted for Site Class 'D' Effects (S_{M1})[3]	1.168
Design, 5.0 Percent Damped, Spectral Response Acceleration Parameter at Short Periods (S_{DS})[3] for Site Class 'D.'	1.145
Design, 5.0 Percent Damped, Spectral Response Acceleration Parameter at 1-Second (S_{D1})[3] for Site Class 'D.'	0.779
Seismic Design Catagory[4]	E
Model Magnitude Earthquake (M)[5]	7.5
Average Shear Wave Velocity in the Top 30m of the Site for Site Class 'D.'[5]	274 m/s
Peak Ground Acceleration (PGA)[3]	0.677
Site Coefficient (F_{PGA})[4]	1.0
$PGA_M = F_{PGA} * PGA$[5]	0.677g

1. Determined from Figure C1-1, 'Approximate Relationship between Number of Lives Placed at Risk by a Failure and Occupancy Category,' in Chapter C1 of **ASCE 7-10**, 2010 Edition.
2. Per Table 20.3-1, 'Site Classification,' in Chapter 20 of **ASCE 7-10**, 2010 Edition.
3. Java Ground Motion Parameter Calculator - Version 5.1.0 (2-10-2011) available at **USGS** web site (http://earthquake.usgs.gov/research/hazmaps/design/). Data based on **ASCE 7-10**, 2010 Edition, 'Standard, Minimum Design Loads for Buildings and Other Structures.'
4. Per Table 11.6-1, 'Seismic Design Category Based on Short Period Response Acceleration Parameters' and Table 11.6-2, 'Seismic Design Category Based on 1-S Period Response Acceleration Parameters' in Chapter 11 of **ASCE 7-10**, 2010 Edition.
5. Per Table 11.8-1, 'Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA,' in Chapter 11 of **ASCE 7-10**, 2010 Edition.
6. Per Section 11.8.3 in Chapter 11 of **ASCE 7-10**, 2010 Edition.

Actual shaking intensities at the site from any seismic source may be substantially higher or lower than estimated for a given earthquake magnitude, due to complex and unpredictable effects from variables such as:

- Near-source directivity effects.
- Direction, length, and mechanism of fault rupture (strike-slip, normal, reverse).
- Depth and consistency of unconsolidated sediments.
- Topography.
- Geologic structure underlying the site.
- Seismic wave reflection, refraction, and interference.

FOUNDATION DESIGN RECOMMENDATIONS

General

The recommendations presented in the subsequent paragraphs for foundation design and construction are based on geotechnical characteristics and 'Non-Expansive' conditions for the supporting earth materials as defined in Section

1803.5.3, 'Expansive Soil,' in the 2016 CBC and should not preclude more restrictive structural requirements. Foundations for the proposed structures may consist of conventional column and continuous wall footings founded upon undisturbed, documented, properly, compacted fill.

The Structural Engineer for the project should determine the actual footing width, depth, and reinforcing to resist design vertical, horizontal, and uplift forces under static and seismic conditions. Reinforcement recommendations presented in this report are considered the minimum for the earth material conditions present on the site and are not intended to supersede the design of the project Structural Engineer or the criteria of the governing agencies for the project. The project Structural Engineer may design a 'Slab-on-Ground Foundation' system based on the current **Wire Reinforcement Institute, Inc.** procedures or a 'Post-Tension Slab-on-Ground' system based on the current **Post Tensioning Institute** as an alternative to conventional reinforced concrete foundations and cast-on-grade concrete floor slabs. Geotechnical parameters for the design of a 'Slab-on-Ground Foundation' system or a 'Post-Tension Slab-on-Ground' system can be submitted upon request, if needed.

Foundation Size

Continuous footings should have a width of 12 inches or greater. Footings supporting a roof only shall be as required for supporting one (1) floor. Continuous footings should be continuously reinforced with a minimum of one (1) No. 4 steel reinforcing bar located near the top and one (1) No. 4 steel reinforcing bar located near the bottom of the footings to minimize the effects of slight differential movements which may occur due to minor variations in the engineering characteristics or seasonal moisture change in the supporting soils. Column footings should have a width of 18 inches by 18 inches or greater and be suitably reinforced, based on structural requirements. The continuous footings should extend across doorway and garage entrances and should be founded at the same depths and reinforced the same as the adjacent footings.

Depth of Embedment

Exterior and interior footings supported in undisturbed, documented, properly compacted fill should extend to a depth of 12 inches or greater below lowest adjacent finish grade. Footings should extend to a depth of 12 inches or greater into the bedrock material underlying the unsuitable on-site earth materials due to the expansion potential of the supporting earth materials. Frost is not considered a design factor for foundations in the City of Redlands, California, since there will not be any significant frost penetration in the winter months.

Footing Setback

Embedment of footings on or near existing or planned slopes should be determined by a setback distance measured from the bottom outside edge of the footing to the slope face in accordance with Section 1808.7, 'Foundations on or Adjacent to Slopes,' in the 2016 CBC or the current City of Redlands, California codes and ordinances, whichever is greater.

Bearing Capacity

Provided the recommendations for site earthwork and for footing width and depth of embedment are incorporated into the project design and construction, the allowable bearing value for design of continuous and column footings for the total dead plus frequently-applied live loads is 2,000 pounds per square foot (psf) for footings that are 12 inches in width and a depth of embedment of 12 inches or greater below lowest adjacent finished grade in accordance with Table 1806.2, 'Presumptive Load-Bearing Values,' in the 2016 CBC for footings founded in undisturbed, documented, properly, compacted fill material (Class 4 Material). For eccentrically loaded footings and/or overturning moments, the resultant force should be in the middle one-third of the footing and the average bearing value across the footing should not exceed the recommended allowable bearing value. The allowable bearing value has a factor of safety of 3.0 or greater and may be increased by 33.3 percent for short durations of live and/or dynamic loading such as wind or seismic forces.

Settlement

Footings designed according to the recommended bearing value, the assumed maximum wall and column loads, and founded in undisturbed, documented, properly, compacted fill material are not expected to exceed a total settlement of 1.0 inch or a differential settlement of 0.25 inch between similarly sized and loaded footings.

Lateral Capacity

Resistance to lateral loads can be provided by a combination of friction acting at the base of the foundation and passive earth pressure on the sides of the footings and stem walls. Foundation design parameters, based on undisturbed, documented, properly compacted fill (Class 4 Material) for resistance to static lateral dead forces are as follows:

ALLOWABLE LATERAL BEARING PRESSURE
(Equivalent Fluid Pressure), Passive Case

Material Type	Bearing Pressure
Undisturbed, Documented, Compacted, 'Non-Expansive' Fill**	150 pcf*
Undisturbed, Existing, On-Site Soil	***

* Pounds per square foot per foot of depth (pcf).

** Per Table 1806.2, 'Presumptive Load-Bearing Values,' for a Class 4 Material (SW, SP, SM, SC, GM, and GC) in the 2016 CBC with a relative compaction of 85% or greater.

*** Materials are to be removed and replaced as properly compacted fill to support foundations.

ALLOWABLE LATERAL SLIDING RESISTANCE BETWEEN SOIL AND CONCRETE

Material Type	Coefficient of Friction
Undisturbed, Documented, Compacted, 'Non-Expansive' Fill*	0.25
Undisturbed, Existing, On-Site Soil	**

* Per Table 1806.2, 'Presumptive Load-Bearing Values,' for a Class 4 Material (SW, SP, SM, SC, GM, and GC) in the 2016 CBC with a relative compaction of 85% or greater.

** Materials are to be removed and replaced as properly compacted fill to support foundations.

The above values are allowable design values and have safety factors of 2.0 or greater incorporated into them and may be used in combination without reduction in evaluating the resistance to lateral loads. The recommended lateral resistance assumes a horizontal surface for the earth material mass extending to a distance of 10 feet or greater from the face of the footing, or three (3) times the height of the surface generating passive pressure, whichever is greater. The allowable values may be increased by 33.3 percent for short durations of live and/or dynamic loading, such as wind or seismic forces. For the calculation of the allowable lateral bearing pressure (passive earth resistance), the upper 1.0 foot of material should be neglected unless confined by a concrete slab or pavement. The largest recommended allowable lateral bearing pressure (passive earth resistance) is 15 times the recommended design value for the appropriate CBC class of material.

Interim Foundation Plan Review

It is recommended that **HGI** review the foundation plans for the structures as they become available. The purpose of this review is to determine if these plans have been prepared in accordance with the recommendations contained in this report. This review will also provide **HGI** an opportunity to submit additional recommendations as conditions warrant.

Final Foundation Design Recommendations

Final foundation recommendations should be made upon completion of grading and be included in the 'Report of Grading' prepared by the Geotechnical / Geologic Consultant for the project.

Foundation Excavations

Foundation excavations should be observed by the project Geotechnical / Geologic Consultant and/or his representative prior to placement of forms, reinforcing steel, or placement of concrete for the purpose of verification of the recommendations presented in this report and for compliance with the project plans and specifications. The foundation excavations should be trimmed neat, level, and square. Any loose or sloughed material and debris should be removed from the foundation excavations prior to placement of reinforcing steel and removed again prior to the placement of concrete. Earth materials removed from the foundation excavations should not be placed in slab-on-grade, hardscape, and/or pavement areas unless compacted to 90 percent or greater relative compaction. The maximum dry density and optimum moisture content for the earth material should be determined in accordance with current ASTM D1557 procedures.

SLAB-ON-GRADE FLOOR RECOMMENDATIONS

The recommendations for concrete slabs on-grade, both interior and exterior, excluding Portland Cement Concrete (PCC) pavement, are based on geotechnical characteristics and 'Non-Expansive' conditions for the supporting earth material as defined in Section 1803.5.3, 'Expansive Soil,' in the 2016 CBC. The expansion potential of the slab subgrade areas should be verified at the completion of grading of the building pad areas. Concrete slabs should be designed to minimize cracking as a result of shrinkage. Joints (isolation, contraction, and construction) should be placed in accordance with the current **American Concrete Institute (ACI)** or **Portland Cement Association (PCA)** guidelines. Special precautions should be taken during placement and curing of concrete slabs. Excessive slump (high water / cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could result in excessive shrinkage, cracking, or curling in the slabs. It is recommended that concrete proportioning, placement, and curing be performed in accordance with ACI recommendations and procedures.

Interior Floor Slabs

Interior concrete floor slabs-on-grade should be 4.0 inches or greater in thickness and be placed on properly prepared subgrade per the 'Earthwork Recommendations' section of this report. The concrete for the floor slab should have a compressive strength of 2,500 pounds per square inch (psi) or greater at 28 days. Slab reinforcement should consist of a minimum of No. 3 reinforcing bars placed 30 inches on center in both directions, or an equivalent substitute. The amount of reinforcing in the floor slab should be increased as necessary based on the structural loads placed on the floors. The reinforcing should be placed at mid-depth to 1.5 inches below the top surface of the slab to minimize cracking. The concrete section, reinforcing steel, and/or design concrete compressive strength should be increased appropriately for anticipated excessive or concentrated floor loads. A

Modulus of Subgrade Reaction (k_s) of 150 pounds per square inch per inch of deflection is recommended for the design of structural slabs cast on grade for excessive floor loads. A compacted sand or gravel bedding layer beneath lightly loaded floor slabs is not needed but may be desirable to enhance the design section for heavy floor loads. If gravel bedding is used, it should consist of a well graded, crushed aggregate. The sand or gravel layer should be compacted to 90 percent or greater of maximum dry density, as determined by current ASTM D1557 procedures.

If a vapor barrier / moisture retarder is used under the floor slab and it is placed on well graded, crushed, gravel material, it is recommended that a 1.0 inch thick layer of sand or other approved granular material be placed beneath the vapor barrier / moisture retarder to prevent punctures from angular gravel fragments and projections. If open graded gravel (capillary break) is placed beneath the vapor barrier or retarder, the gravel layer should be 6.0 inches or greater in thickness. If open graded gravel is used, a separation fabric such as Mirafi 140N series, or an equivalent substitute, should be used in-leu of a sand cushion to protect the vapor barrier / moisture retarder from punctures.

Subgrade soils should be moisture conditioned to optimum moisture content to 3.0 percent above optimum moisture content to a depth of 12 inches and proof compacted to 90 percent or greater relative compaction based on current ASTM D1557 procedures immediately before placing the gravel material, the moisture barrier, or pouring concrete.

Vapor Barrier / Moisture Retarder Recommendations

HGI does not practice in the field of moisture vapor transmission evaluation / mitigation. Therefore, it is recommended that a qualified person or firm be engaged or consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. This person or firm should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate in accordance with ACI, PCA, ASTM, PTI, the California Building Code, and/or the International Residential Code.

In heated / air conditioned areas in a structure where moisture sensitive floor coverings are anticipated over the floor slab, the use of a vapor barrier / moisture retarder beneath the slab should be considered. Typically, a vapor retarder is not utilized under the floor slabs in garages, utility buildings, and other unheated accessory structures, driveways, walks, patios, and/or other flatwork not likely to be enclosed and heated at a later date. The use or non-use of a vapor barrier / moisture retarder, the thickness of the vapor barrier / moisture retarder, the use of a granular layer over the vapor barrier / moisture retarder, the thickness of the granular materials, the type of granular material, etc. should be determined by the Structural Engineer who is designing the floor slab in conjunction with the Architect who is specifying the use and the type of floor coverings to be placed over the floor slab, and/or a person or firm that practices in the field of moisture vapor transmission evaluation / mitigation. The vapor barrier / moisture retarder recommendations provided by the supplier of the flooring materials should also be incorporated into the project plans.

EXTERIOR CONCRETE FLATWORK

Exterior concrete slabs cast on finish subgrade (i.e., pedestrian walkways, patios, sidewalks, etc., with the exception of PCC pavement) should be 4.0 inches or greater in thickness and be underlain by 12 inches or greater of earth material that has been prepared in accordance with the 'Earthwork Recommendation' section of this report. Reinforcing in the slab, the design compressive strength of the concrete, and the use of a compacted sand or gravel base beneath the slabs should be according to the current codes and ordinances of the City of Redlands, California. Subgrade earth materials should be moisture conditioned to optimum moisture content to 3.0 percent above optimum moisture content to a depth of 12 inches or greater and proof compacted to 90 percent or greater relative compaction based on current ASTM D1557 procedures immediately before placing aggregate base material, placing reinforcing steel, or placing the concrete.

RETAINING WALL RECOMMENDATIONS

Low height retaining walls may be needed to achieve finish grades for the proposed building pads, driveways, parking areas, and/or landscape areas. Retaining walls should be designed in accordance with the recommendations in the following sections. If earth reinforced walls, crib wall, keystone walls, etc. are used for the development of the subject site, the design requirement of the proprietary retaining wall system should supercede the following recommendations if there are any conflicts.

Static Lateral Earth Pressures

Retaining walls backfilled with 'Non-Expansive' granular soil (i.e., Expansion Index (EI) ≤ 20 and Unified Soil Classifications of SP, SW, SM, GP, GW, and GM) within a zone extending upward and away from the heel of the footing at a slope of 0.5H:1V (Horizontal to Vertical) or flatter for level backfill and 0.7H:1V for a 2H:1V slope behind the retaining wall can be designed to resist static lateral earth pressures equivalent to those recommended in the following table:

LATERAL EARTH PRESSURE						
Condition	Level Backfill and Soil Classification*			2H:1V Sloped Backfill and Soil Classification***		
	SP, SW, GP, GW	GM	SM	SP, SW, GP, GW	GM	SM
Active	30 pcf**	40 pcf	45 pcf	40 pcf	62 pcf	81 pcf
At-Rest	60 pcf	60 pcf	60 pcf	87 pcf	110 pcf	120 pcf

* Per Table 1610.1, 'Lateral Soil Load,' in the 2016 CBC.

** Equivalent fluid Pressure, pounds per square foot per foot of depth (pcf).

*** Based on a moist unit weight of 125 pcf and an Angle of Internal Friction of 38 degrees for SP, SW, GP, and GW backfill soils, 31 degrees an for GM backfill soils, and 28 for an Angle of Internal Friction of 28 for SM backfill soils.

The designer of the retaining wall should specify the type of backfill material to be used in the active / at-rest zone behind the retaining wall. Any expansive soils which may be encountered on the subject site should not be used as backfill for retaining walls. Retaining walls that are free to deflect 0.001 radian at the top should be designed for the above-recommended active condition. Retaining walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The above values assume well-drained backfill and that a buildup of hydrostatic pressure will not occur. Surcharge loads, dead and/or live (i.e., construction loads, etc.), acting on the backfill within a horizontal distance behind the retaining wall, equivalent to or less than the vertical height of the retaining wall, should also be considered in the design. Uniform surcharge pressures should be applied as an additional uniform (rectangular) pressure distribution. The lateral earth pressure coefficient for a uniform vertical surcharge load behind the retaining wall is 0.50. Seismic and wind loads should also be added to the design loads on the retaining walls, if applicable.

Seismic Lateral Earth Pressure

In accordance with Section 1803.5.12, 'Seismic Design Categories D through F,' in the 2016 CBC for the habitable structures, seismic loads should also be added to the design loads on the retaining walls retaining more than 6.0 feet in height. Recommended seismic lateral earth pressures can be provided upon request.

Foundation Design

Retaining wall footings should be founded to the same depths below lowest adjacent finished grade and offsets from the face of slopes, and into undisturbed, observed and tested, compacted fill. The foundations may be designed for the same average allowable bearing value across the footing (as long as the resultant force is located in the middle one-third of the footing), and with the same allowable static and seismic allowable lateral bearing pressure, allowable passive earth pressure, and allowable sliding resistance as recommended in the 'Foundation Design Recommendations' section of this report. Retaining walls should be designed for a factor of safety of 1.5 against lateral sliding and overturning per Section 1807.2.3, 'Safety Factor,' in the 2016 CBC. When using the allowable lateral pressure and allowable lateral sliding resistance, a factor of safety of 1.0 may be used. If ultimate values are used for design, an approximate factor of safety (i.e., 1.5) should be achieved.

Foundation Size: Continuous footings should have a width of 12 inches or greater. Continuous footings should be continuously reinforced with a minimum of one (1) No. 4 steel reinforcing bar located near the top and one (1) No. 4 steel reinforcing bar located near the bottom of the footings to minimize the effects of slight differential movements which may occur due to minor variations in the engineering characteristics or seasonal moisture change in the supporting expansive earth materials.

Depth of Embedment: Footings should extend to a depth of 12 inches or greater below lowest adjacent finish grade.

Footing Setback: Embedment of footings on or near existing or planned slopes should be determined by a setback distance measured from the bottom outside edge of the footing to the slope face in accordance with Section 1808.7, 'Foundations on or Adjacent to Slopes,' in the 2016 CBC or the current City of Redlands, California building codes, whichever is greater.

Bearing Capacity: Provided the recommendations for site earthwork and for footing width and depth of embedment are incorporated into the project design and construction, the allowable bearing value for design of retaining wall footings for the total dead plus frequently-applied live loads is 2,000 pounds per square foot (psf) for footings that are 12 inches in width and a depth of embedment of 12 inches below lowest adjacent finish grade in accordance with Table 1806.2, 'Presumptive Load-Bearing Values,' in the 2016 CBC for footings founded in undisturbed, documented, properly, compacted fill material (Class 4 Material). For eccentrically loaded footings and/or overturning moments, the resultant force should be in the middle one-third of the footing and the average bearing value across the footing should not exceed the recommended allowable bearing value. The allowable bearing values have a factor of safety of 3.0 or greater and may be increased by 33.3 percent for short durations of live and/or dynamic loading such as wind or seismic forces.

Settlement: Footings designed according to the recommended bearing values are not expected to exceed a total settlement of 1.0 inch or a differential settlement of 0.5 inch between similarly sized and loaded footings.

Lateral Capacity

Resistance to lateral loads can be provided by a combination of friction acting at the base of the foundation and passive earth pressure on the sides of the footings and stem walls. Foundation design parameters, based on undisturbed, documented, properly compacted fill (Class 4 Material) for resistance to static lateral dead forces per Table 1806.2, 'Presumptive Load-Bearing Values,' in the 2016 CBC are as follows:

Allowable Lateral Bearing Pressure
(Equivalent Fluid Pressure), Passive Case:

Undisturbed, Documented, Compacted, 'Non-Expansive' Fill - 150 pcf*
Undisturbed, On-Site, 'Non-Expansive,' Alluvial Soil** - 150 pcf
Undisturbed, Existing, On-Site Soil - ****

*	Pounds per square foot per foot of depth (pcf).
**	Per Table 1806.2, 'Presumptive Load-Bearing Values,' for a Class 4 Material (SW, SP, SM, SC, GM, and GC) in the 2016 CBC.
***	Materials are to be removed and replaced as properly compacted fill to support foundations.

Allowable Lateral Sliding Coefficient of Friction Between Soil and Concrete:

Undisturbed, Documented, Compacted, Non-Expansive' Fill** - 0.25
Undisturbed, On-Site, 'Non-Expansive,' Alluvial Soil** - 0.25
Undisturbed, Existing, On-Site Soil - ***

**	Per Table 1806.2, 'Presumptive Load-Bearing Values,' for a Class 4 Material (SW, SP, SM, SC, GM, and GC) in the 2016 CBC.
***	Materials are to be removed and replaced as properly compacted fill to support foundations.

The above values are allowable design values and have safety factors of 2.0 or greater incorporated into them and may be used in combination without reduction in evaluating the resistance to lateral loads. The recommended lateral resistance assumes a horizontal surface for the earth material mass extending to a distance of 10 feet or greater from the face of the footing, or three (3) times the height of the surface generating passive pressure, whichever is greater. The allowable values may be increased by 33.3 percent for short durations of live and/or dynamic loading, such as wind or seismic forces. For the calculation of the allowable lateral bearing pressure (passive earth resistance), the upper 1.0 foot of material should be neglected unless confined by a concrete slab or pavement. The largest recommended allowable lateral bearing pressure (passive earth resistance) is 15 times the recommended design value for the appropriate class of material.

Subdrain

A subdrain system should be constructed behind, and at the base of retaining walls to allow drainage and to prevent the buildup of excessive hydrostatic pressures. The subdrain system should be designed by the project Civil Engineer. The use of water-stops, impermeable barriers, or other dampproofing or waterproofing methods should be considered for any retaining walls where moisture migration through the retaining wall is considered critical to the performance and/or appearance of the retaining walls. A waterproofing consultant should be retained to provide specific waterproofing recommendations for the project, if required.

Typical subdrains may include weep holes with a continuous free draining gravel gallery, perforated pipe surrounded by free draining filter rock, or another approved system. The option of providing an ungrouted, open coarse of block at the bottom of a retaining wall is not a recommended drainage option since the openings in this coarse are so often covered by landscape soil, hardscape, and or pavement. Gravel galleries and/or filter rock, if not designed and graded for the on-site and/or import materials, should be enclosed in a geotextile fabric such as Mirafi 140N series, or an equivalent substitute, to prevent infiltration of fine soil particles into the subdrain and clogging of the system. Before placement of the fabric, the top of the footing should be cleared of loose soil materials, large stones, and/or other debris. Any large depressions or holes should be filled with a concrete slurry or a suitable equivalent to permit close contact of the fabric with the surrounding surface. The fabric should be placed smoothly without folds or excessive wrinkles. Successive sheets of the fabric should be placed with an overlap of 24 inches or more in the direction of the flow of the water in the pipe with the upstream layer overlapping the downstream layer. The fabric should be folded over the top of the free draining granular material producing an overlap of 12 inches or more. The perforated pipes should be Schedule 40 or stronger and 4.0 inches or greater in diameter. Perforations may be either bored 0.25-inch diameter holes or 0.1875-inch (3/16-inch) wide slots placed on the bottom one-third of the pipe perimeter. If the pipe is bored, a minimum of 10 holes per linear foot should be uniformly placed along the pipe. If slots are used, they should not exceed 2.0 inches in length and should not be closer than 2.0 inches on center along the length of the pipe. The total length of the slots should not be less than 50 percent of the pipe length and should be uniformly spaced along the length of the pipe. Pipe perforations should be placed downward. Gravel filters should have a volume of 3.0 cubic feet or greater per linear foot of pipe. Subdrains should maintain a positive flow gradient and have outlets that drain in a non-erosive manner.

Prefabricated drainage products such as 'Miradrain,' or a suitable equivalent, may also be used for the purpose of providing drainage behind retaining walls when installed in accordance with the manufacturers recommendations.

Backfill

Backfill directly behind retaining walls (if backfill width is less than 3.0 feet) may consist of 0.5- to 1.5-inch diameter, rounded to subrounded gravel with less than 5.0 percent passing the 0.5 inch sieve enclosed in a geotextile fabric such as Mirafi 140N series, or an equivalent substitute, or a clean sand (Sand Equivalent Value greater than 50) water jetted into place to obtain compaction. If water jetting is used, the subdrain system should be in place. Even if water jetting is used, the sand should be densified to 90 percent or greater relative compaction. If the specified density is not obtained by water jetting, mechanical methods will have to be used. If other types of soil or gravel are used for backfill, mechanical compaction methods will have to be used to obtain a relative compaction of 90 percent or greater of maximum dry density. Backfill directly behind retaining walls should not be compacted by wheel, track or other rolling by heavy construction equipment unless the retaining wall is designed for the surcharge loading. If gravel, clean sand, or other imported backfill is used behind retaining walls in unpaved areas, the upper 12 to 18 inches of backfill should consist of typical on-site material compacted to 90 percent or greater relative compaction to prevent the influx of surface run-off into the granular backfill and into the subdrain system. Maximum dry density and optimum moisture content for backfill materials should be determined according to current ASTM D1557 procedures.

V-Drain Design

A V-drain should be constructed directly behind retaining walls which have a sloping backfill to intercept surface water and drain it from the back of the retaining wall. The V-drain should be designed and constructed in accordance with the current typical standards of the City of Redlands, California. The V-drain should direct water from the back of the retaining wall to an adequate down drain and discharge it in a non-erosive manner.

Observation and Testing

During retaining wall construction, observation and testing should be conducted by the project Geotechnical / Geologic Consultant and/or his representatives to verify that the work is being performed according to the recommendations presented in this report.

The foundation excavations should be observed by the project Geotechnical / Geologic Consultant and/or his representative prior to placement of forms, reinforcing steel, or placement of concrete for the purpose of verification of the recommendations presented in this report and for compliance with the project plans and specifications. The foundation excavations should be trimmed neat, level, and square. Any loose or sloughed material and debris should be removed from the foundation excavations prior to placement of reinforcing steel and removed again prior to the placement of concrete.

The placement and construction of the subdrain system behind the retaining walls should be observed by the project Geotechnical / Geologic Consultant and/or his representatives to verify that the work is being performed according to the recommendations presented in this report.

During backfill of the retaining walls, observation and testing should be conducted by the project Geotechnical / Geologic Consultant and/or his representatives to verify that the backfilling is being performed according to the recommendations presented in this report. The project Geotechnical / Geologic Consultant and/or his representative should observe the placement of fill and should take tests to verify the moisture content, density, uniformity and degree of compaction obtained. Where testing demonstrates insufficient density, additional compaction effort, with the adjustment of the moisture content when needed, should be applied until retesting shows that satisfactory relative compaction has been obtained. The results of observations and testing services should be presented in a formal report following completion of the construction operations. Retaining wall backfill operations undertaken at the site without the project Geotechnical / Geologic Consultant and/or his representative present may result in exclusions of the affected areas from the final report for the project.

The presence of the project Geotechnical / Geologic Consultant and/or his representative will be for the purpose of providing observations and field testing and will not include supervision or directing of the actual work of the contractor or the contractor's employees or agents. Neither the presence and/or the non-presence of the project Geotechnical / Geologic Consultant and/or his field representative nor the field observations and testing will excuse the contractor for defects discovered in the contractor's work.

CORROSION POTENTIAL EVALUATION

The recommendations for corrosion protection should be verified at the completion of grading of the building pads on the subject site. Bulk samples of the near surface, on-site earth materials were obtained during the field study to evaluate the potential for corrosivity. Results from the tests are included in the 'Summary of Laboratory Test Results' presented in Appendix 'A.'

Concrete Corrosion Potential

A preliminary test on a sample of near-surface, on-site earth material suggest a soluble sulfate concentration of 0.0021 percent. Earth materials with a water soluble sulfate (SO_4) concentration of less than 0.10 percent are considered to be Category S, Class S0 in accordance with Table 19.3.1.1, 'Exposure Categories and Classes,' in **American Concrete Institute (ACI)** 318-14. Therefore the requirements in Table 19.3.2.1, 'Requirements for Concrete by Exposure Class,' in **ACI** 318-14 are applicable. The referenced **ACI** Table 19.3.2.1 should be used to determine the type cement, the maximum water cement ratio, and the minium compressive strength to be used for normal weight concrete which comes in direct contact with the on-site earth materials (i.e., foundations, floor slabs, driveway slabs, sidewalks, patios, curbs / gutters, etc.). The applicable portion of the referenced **ACI** Table 19.3.2.1, as presented on Figure No. 4, should be used to determine the type cement, the maximum water cement ratio, and the minium compressive strength to be used for normal weight concrete which comes in direct contact with the on-site earth materials (i.e., storm drain pipe / box culvert, driveway slabs, sidewalks, curbs / gutters, etc.). A lower water / cement ratio or higher compressive strength may be required for design of concrete for water tightness or for protection against freezing and thawing, or for corrosion protection of concrete reinforcement per Section 1904, 'Durability Requirements,' in the 2016 CBC, if applicable.

Experience in the southern California area has shown that even though the earth materials do not contain levels of soluble sulfate which would require the use of sulfate resistant cement, maximum water cement ratios, or minimum compressive strength for concrete, concrete corrosion and erosion problems still occur. These problems are the result of concentrations of soluble sulfate, chloride, and other salts and/or acids present in groundwater, irrigation water, rain water, and potable water sources, and in fertilizers or amendments used to promote plant growth (i.e., some domestic water sources contain levels of dissolved sulfate which would be a Class S1 exposure to concrete which comes in contact with it). Therefore, it may be wise to use a concrete designed for a Category S, Class S1 criteria that comes into contact with surface run-off or other sources of water. Higher strength, lower water / cement ratio, and denser concrete may also be effective in reducing the potential for corrosion to occur and preventing damage due to salt or acid exposure. The use of sulfate resistant concrete for non-structural elements (i.e., driveway slabs, sidewalks, patios, curbs / gutters, etc.), is considered to be a value / risk assessment and decision to be made by the owner / developer.

Metallic Corrosion Potential

The life of buried metals depends on type of material, thickness, and construction details. Since **HGI** does not practice metallic corrosion engineering, if corrosion protection is considered to be a design issue, an engineer specializing in corrosion should be consulted regarding the potential damage due to corrosion. The corrosion engineer should recommend appropriate types of piping and/or protective measures where needed.

A preliminary minimum resistivity test on a sample of the near-surface, on-site, earth material of 5,561 ohm-cm suggest a mild to moderate corrosive environment for buried ferrous metal in direct contact with the on-site earth materials when the earth materials are wet. Soils with a minimum resistivity of less than 1,000 ohm-cm indicates a severe corrosive environment and a minimum resistivity of 2,000 ohm-cm or greater indicates a mild to moderate corrosive environment for buried ferrous metal in direct contact with the soils when the soils are wet.

A preliminary test on a sample of near-surface, on-site, earth material suggests a soluble chloride concentration of 23 parts per million (ppm). Earth materials with greater than 300 and 500 ppm of soluble chloride are considered to be aggressive to buried ferrous and copper material respectively, in direct contact with the earth materials.

Earth material pH is a general indicator of the corrosivity of earth materials. The measured pH of a sample of near-surface, on-site, earth material of 8.2 indicates a non-corrosive environment to copper and ferrous metals when in direct contact with the on-site earth materials.

Sulfide in soils is a general indicator of the corrosivity of earth materials. The measured sulfide of the samples of near-surface, on-site, earth material tested as part as part of this report from the finish building pads was 'Negative' which indicates a non-corrosive environment to copper and ferrous metals when in direct contact with the on-site earth materials.

Salt Crystallization Exposure

Damage of concrete, concrete masonry units, slump stone block, etc. surface can occur when evaporation of moisture takes place at the surface of the materials. As evaporation takes place, salts (i.e,. carbonates, chloride, sulfur, sodium, potassium, etc.) are deposited in or form on the surfaces. As the salts crystalize, they can exert extreme pressures in the pore spaces of the materials they are deposited in and/or on. The formation of the crystals within the pore spaces of the material can result in what is generally called 'salt crystallization damage.' This results in the scaling and/or etching of the surface of the material on which they are deposited. The damaging effects of this phenomenon can be greatly reduced and/or even eliminated by the following or other such methods: 1) either using a higher strength concrete or a denser, low porosity product; 2) seal the surface of the material with a water proofing substance which will prevent the evaporation of the moisture from within the cementitious product. If 'salt crystallization damage' is considered to be an issue, an engineer or chemist specializing in this area should be consulted regarding the potential damage due evaporation and the deposition of salts. The engineer or chemist should recommend appropriate types of materials or protective measures where needed.

PRELIMINARY PAVEMENT RECOMMENDATIONS

The following are preliminary recommendations for the structural pavement section for the proposed parking areas, and driveway areas for the subject development. The Hot Mix Asphalt (HMA) concrete pavement sections have been determined in general accordance with current **California Department of Transportation (CALTRANS)** design procedures using the CalFP Ver. 1.1 'Hot Mix Asphalt

Empirical Design' computer program developed by the **CALTRANS, Office of Pavement Design** and are based on a an assumed Traffic Index (TI) of 5.5 for a 20 year design life and an assumed R-Value of at least 40 based on past experience in the vicinity of the site and visual textural classification of the on-site earth material and/or import materials which are anticipated to be at subgrade elevation.

Portland Cement Concrete (PCC) pavement sections are based on an equivalent structural numbers as the recommended HMA concrete pavement section and a compressive strength of 2,500 psi or greater at 28 days for the concrete.

The preliminary recommendations for the pavement sections should consist of the following:

RECOMMENDED PAVEMENT SECTIONS

Site Area	Traffic Index*	Subgrade R-Value**	Pavement Section
Driveway and Parking Areas for Autos and Light Weight Vehicles Only.	≤5.5	≥40	3.5" Hot Mix Asphaltic (HMA) Concrete over 5.0" Aggregate Base (AB) or 5.5" PCC @ 2,500 psi over properly prepared subgrade.

* Traffic Index was assumed for the project.
** R-Value was assumed for the project.

It is noted that the City of Redlands minimum pavement sections may override the above pavement recommendations without prior City review and approval.

HMA concrete pavement materials should be as specified in Section 39, 'Hot Mix Asphalt,' in the current CALTRANS 2010 'Standard Specifications' with the 7-18-

2014 Revisions, or an equivalent substitute. Aggregate base should conform to Class 2 Material, 1-1/2" Maximum or 3/4" Maximum, as specified in Section 26-1.02B, 'Class 2 Aggregate Base,' in the current, CALTRANS 2010 'Standard Specifications' with the 7-18-2014 Revisions, or an equivalent substitute.

Portland Cement Concrete sections are based on a compressive strength of 2,500 psi or greater at 28 days for the concrete. Higher strength design for the concrete can permit thinner pavement sections. Lower strength design for the concrete will require thicker pavement sections. Joints (longitudinal, transverse, construction, and expansion), jointing arrangement, joint type, pavement and/or joint reinforcing, as well as drainage, crowning, finishing and curing of PCC pavement should be in accordance with current Portland Cement Association (PCA) recommendations.

The subgrade earth material, including utility trench backfill, should be compacted to 90 percent or greater relative compaction to a depth of 1 foot or greater below the finish pavement subgrade elevation. The aggregate base material should be compacted to 95 percent or greater relative compaction. If asphaltic concrete and/or PCC pavement is placed directly on subgrade, the upper 1.0 foot of the subgrade should be compacted to 95 percent or greater relative compaction. Maximum dry density and optimum moisture content for subgrade and aggregate base materials should be determined according to current ASTM D1557 procedures. The asphalt concrete pavement should be densified to 95 percent or greater of the density obtained by current California Test 304 and 308 procedures (Hveem compacted laboratory samples).

If semi-trailers are to be parked on the asphalt concrete pavement, such that a considerable load is transferred from small, steel wheels, it is recommended that a strip of rigid Portland Cement concrete pavement with a thickness of 6.0 inches or greater be provided in these areas. This will provide for the distribution of loads to the subgrade without causing deformation of the pavement surface. Special consideration should also be given to areas where truck traffic will negotiate small radius turns and/or in areas utilized by solid tired forklifts or other material handling equipment. HMA concrete pavement in these areas should utilize stiffer emulsions or the areas should be paved with Portland Cement concrete. Where HMA concrete pavement abuts concrete aprons, drives, walks, or curb and gutter sections, a thickened edge transition zone is recommended for the HMA concrete section to minimize the effects of impact loading as vehicles transition from PCC paving to HMA concrete paving. This thickened edge should consist of an increased thickness of 2.0 inches for parking areas and 4.0 inches for areas of heavy truck usage. This thickened edge should extend to a distance of 3.0 feet or greater from the edge of pavement and then gradually taper back to the design pavement thickness. If pavement subgrade earth materials are prepared at the time of grading of the building site and the areas are not paved immediately, additional observations and testing will have to be performed before placing aggregate base material, asphaltic concrete, or PCC pavement to locate areas that may have been damaged by construction traffic, construction activities, and/or seasonal wetting and drying. In the proposed pavement areas, earth material samples should be obtained at the time the subgrade is graded for Resistance (R-Value) testing according to current California Test 301 procedures to verify the pavement design recommendations.

Because the full design thickness of the HMA concrete is frequently not placed prior to construction traffic being allowed to use the parking lots, rutting and pavement failures can occur prior to project completion. To reduce this occurrence, it is recommended that either the full-design pavement section be placed prior to use by the construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

Surface water infiltration beneath pavements could significantly reduce the pavement design life. To limit the need for additional long-term maintenance of the pavement or pre-mature failure, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cutoff wall, deepened curbs, or equivalent. Pavement cut-off barriers should be considered where pavement areas are located downslope of any landscape areas that are to be irrigated. The cut-off barrier should extend to a depth of at least 4.0 inches below the pavement section aggregate base material.

Gradation is not the only quality guidelines for aggregate base material. The longevity and performance of pavements utilizing aggregate base material for support is dependent upon the quality of the material which composes the aggregate base. CALTRANS specifications do not specifically exclude the use of material other than a natural, crushed rock and rock dust for Class 2 Aggregate Base material as the 'Standard Specifications for Public Works Construction' (2012 Edition of the 'Greenbook' with the 2014 Cumulative Supplement), Section 200-2.2, does for Crushed Aggregate Base material. Often times, reclaimed Portland Cement concrete, Hot Mix Asphalt concrete, lean concrete base, and cement treated base are crushed, combined with broken stone, crushed gravel, natural rough surfaced gravel, and sand per the current Section 26-1.02B, 'Class 2 Aggregate Base,' of the current CALTRANS 2010 'Standard Specifications,' with the 7-18-2014 Revisions, and graded to produce a Class 2 Aggregate Base material per CALTRANS gradation specifications. Bricks, concrete masonry units, tile, glass, ceramics, porcelain, wood, plastic, metal, etc. **are not** an acceptable reclaimed material for use in a Class 2 Aggregate Base material per the current CALTRANS

2010 'Standard Specifications' with the 7-18-2014 Revision. If a reclaimed material is proposed for use on the project as a Class 2 Aggregate Base, the reclaimed materials should not exceed 50 percent of the total volume of the aggregate used. The aggregate base material should be tested prior to delivery to the subject project site for the following quality requirements per the current, appropriate CALTRANS test procedures:

TEST	TEST METHOD NO.	QUALITY REQUIREMENT	
		OPERATING RANGE	CONTRACT COMPLIANCE
Resistance (R-Value)	Calif. Test 301	--	78 Minimum
Sand Equivalent	Calif. Test 217	25 Minimum	22 Minimum
Durability Index	Calif. Test 229	--	35 Minimum

If a reclaimed material or a pit run aggregate is proposed for use on the project as a 'Greenbook' Crushed Miscellaneous Base (CMB), the materials should be tested for the following quality requirements prior to delivery to the subject project, per the current 'Greenbook,' 2012 Edition with the 2014 Cumulative Supplement, Section 200-2.4.3, and appropriate procedures as well as the required gradation and other requirements:

TEST	TEST METHOD NO.	QUALITY REQUIREMENT
Resistance (R-Value)	Calif. Test 301	78 Minimum[1]
Sand Equivalent	Calif. Test 217	35 Minimum
Percent Wear[2] 100 Revolutions 500 Revolutions	ASTM C131	 15 Maximum 52 Maximum

<table>
<tr><th>TEST</th><th>TEST METHOD NO.</th><th>QUALITY REQUIREMENT</th></tr>
<tr><td colspan="3">1. R-Value requirement may be waived if Sand Equivalent is 40 or more.
2. The percentage wear requirements may be waived if the material has a minimum Durability Index of 40 in accordance with CALTRANS Test Method 229.</td></tr>
</table>

A 'Greenbook' CMB may contain broken or crushed asphalt concrete or Portland Cement concrete and may contain crushed aggregate base or other rock materials. The CMB may contain no more than 3.0 percent brick retained on the # 4 sieve by dry weight of the total sample.

Samples of the proposed aggregate base using reclaimed material should be sampled from the manufacturer's stockpiles and tested prior to delivery to the project. The samples should be obtained at a time as near the delivery to the project as possible but would allow enough time to complete the testing and report the results before delivery to the site. Samples should again be obtained and tested for quality compliance from the materials delivered to the project. In addition, per the current CALTRANS 2010 'Standard Specifications' with the 7-18-2014 Revisions, an aggregate grading and Sand Equivalent test shall not represent more than 500 cubic yards or one (1) days production if less than 500 cubic yards.

Concrete gutters should be provided at flow lines in paved areas. Pavements should be sloped to permit rapid and unimpaired flow of runoff water. In addition, paved areas should be protected from moisture migration and ponding from adjacent water sources. Saturation of aggregate base and/or subgrade materials could result in pavement failure and/or premature maintenance. The gutter material and construction methods should conform to the current standards of the City of Redlands, California.

POST-GRADING CRITERIA

Earth materials generated from the excavation of foundations, utility trenches, etc., to be used on-site, should be moisture conditioned to optimum moisture content to 3.0 percent above optimum moisture content and compacted to 90 percent or greater of the maximum dry density for the material type as determined by current ASTM D1557 procedures when it is to be placed under floor slabs, under hardscape areas, and/or in paved areas. The placement of the excess material should not alter positive drainage away from structures and/or off the lot and should not change the distance from the weep screed on the structure to the finished adjacent earth material grade per the 'Finish Surface Drainage Recommendations' presented in a subsequent section of this report.

UTILITY TRENCH RECOMMENDATIONS

Utility trenches within the zone of influence of foundations or under building floor slabs, exterior hardscape, and/or pavement areas should be backfilled with documented, compacted earth material. Utility trenches within the building pad and extending to a distance of 5.0 feet beyond the building exterior footings should be backfilled with on-site or similar earth material. Where interior or exterior utility trenches are proposed to pass beneath or parallel to building, retaining wall, and/or decorative concrete block perimeter wall footings, the bottom of the trench should not be located below a 1H:1V (Horizontal to Vertical) plane projected downward from the outside bottom edge of the adjacent footing unless the utility lines are designed for the footing surcharge loads.

Trench Excavation

It is recommended that utility trench excavations be designed and constructed in accordance with current OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet in vertical depth based on a description and field verification of the earth material types encountered. Trenches over 20 feet in vertical depth should be designed by the Contractor's Engineer based on site specific geotechnical analyses. For planning purposes, we recommend that the following OSHA earth material type designations and temporary slope inclinations be used:

EARTH MATERIAL	OSHA SOIL TYPE*	TEMPORARY SLOPE INCLINATION (H:V)**
Undocumented Fill	C	1.5:1
Compacted Fill	C	1.5:1
Alluvium	C	1.5:1

* Type 'C': Cohesive soils with an unconfined compressive strength of 0.5 tsf or less: or Granular soils including sands, gravels, loamy, clayey or silty sands, etc.

** Steepest allowable slopes for excavations less than 20 feet in vertical height. Slopes for excavations greater than 20 feet in vertical height should be designed by a Registered Professional Engineer with experience in Geotechnical Consulting and Soil Mechanics.

Excavations of less than 5.0 feet in depth may also be subject to collapse due to water, vibrations, previously disturbed earth materials, or other factors and may require protection for workers such as temporary slopes, shoring, or a shielding protective system. The excavations should be observed by a qualified, competent person (as defined in the current OSHA regulations) looking for signs of potential cave-ins on a daily basis before start of work, as needed throughout the work shifts, and after every rainstorm or other hazard-increasing occurrence.

Surcharge loads (i.e., spoil piles, earthmoving equipment, trucks, etc,) should not be allowed within a horizontal distance measured from the top of the excavation slope equivalent to 1.5 times the vertical depth of the excavation (for medium stiff or dense earth materials). Excavations should be initially observed by the project Geotechnical / Geologic Consultant and/or his representative to verify the recommendations presented or to make additional recommendations to maintain stability and safety. Moisture variations, differences in the cohesive or cementation characteristics, or changes in the coarseness of the deposits may require slope flattening or, conversely, permit steepening upon review and appropriate testing by the project Geotechnical / Geologic Consultant and/or his representative. The excavations should be observed by a qualified, competent person (as defined in the current OSHA regulations) looking for signs of potential problems on a daily basis before start of work, as needed throughout the work shifts, and after every rainstorm or other hazard-increasing occurrence. Deep utility trenches may experience caving which will require special considerations to stabilize the walls and expedite trenching operations. Surface drainage should be controlled along the top of the construction slopes to preclude erosion of the slope face. If excavations are to be left open for long periods, the slopes should be sprayed with a protective compound and/or covered to minimize drying out, raveling, and/or erosion of the slopes.

Utility Line Foundation Preparation

If the utility trench excavation bottom is in material that is not suitable for support of the utility pipe, the material should be removed to a minimum depth of 1.0 foot below the bottom of the pipe and replaced with concrete slurry, sand, or crushed gravel meeting the following appropriate gradation limits or some other suitable equivalent as specified by the utility designer.

SIEVE SIZE	CRUSHED ROCK OR GRAVEL (PERCENT PASSING)
1"	100

SIEVE SIZE	CRUSHED ROCK OR GRAVEL (PERCENT PASSING)
3/4“	90-100
½“	30-60
3/8“	0-20
No. 4	0-5

SIEVE SIZE	SAND (PERCENT PASSING)
3/8"	100
No. 4	75-100
No. 30	12-50
No. 100	5-20
No. 200	0-15

Most of the granular native earth materials encountered on the subject site **are not** expected to meet the above granular earth material criteria.

We recommend, that where the bottom of the pipe foundation excavation is loose or soft, the foundation earth materials be removed to firm materials as determined by the Engineer. This condition would likely only apply where fill underlies the pipe in localized areas along a utility alignment. If firm material is not encountered within 24 inches of the bottom of the pipe zone, the contractor may then elect to stabilize the trench bottom with 24 inches of crushed rock as described above. Alternately, soft or loose material may be excavated to firm earth material and the overexcavation replaced with select earth material.

The bottom of the utility trench excavation should be proof compacted to 90 percent or greater relative compaction prior to placement of compacted fill. Maximum dry density and optimum moisture content for compacted materials should be determined according to current ASTM D1557 procedures.

Prior to placement of trench slurry or crushed rock, the bottom need only be cleaned of loose materials created by the excavation process. Where the bottom of the trench contains rocks or hard objects protruding above a depth of 6.0 inches below the pipe bottom, such objects should be removed or broken and any resulting cavities filled to produce a smooth surface.

Bedding Requirements

It is recommended that the pipe be bedded on either clean sand, gravel, crushed rock or any approved suitable material in order to provide a smooth, firm, and uniform foundation for the pipe. The pipe bedding material, thickness, shaping, and placement should satisfy the design requirements as determined by the design Civil Engineer and/or in accordance with Section 306-1.2.1 of the 2012 Edition of the 'Greenbook' with the 2014 Cumulative Supplement.

Trench Zone Backfill

The excavated earth materials from the trench may be used as backfill in the trench zone unless more restrictive specifications are required by the design engineer or the permitting agency. The trench backfill material should consist of approved earth materials free of trash debris, vegetation or other deleterious matter, and oversize particles (i.e., 12 inch in maximum dimension). Trench zone backfill should be compacted to 90 percent or greater relative compaction. Maximum density and optimum moisture content for compacted materials should be determined according to current ASTM D1557 procedures.

Trench backfill material should be placed in a lift thickness appropriate for the type of backfill material and compaction equipment used. Backfill material should be brought to optimum moisture content to 3.0 percent above optimum moisture content and compacted to 90 percent or greater relative compaction by mechanical means. Jetting or flooding of the backfill material will **not** be considered a satisfactory method for compaction. Maximum dry density and optimum moisture content for backfill material should be determined according to current ASTM D1557 procedures.

FINISH SURFACE DRAINAGE RECOMMENDATIONS

Positive drainage should be established away from the tops of slopes, the exterior walls of structures, the back of retaining walls, trash enclosure walls, decorative concrete block walls, etc. Finish surface gradients in unpaved areas should be provided next to tops of slopes and buildings to guide surface water away from foundations, hardscape, pavement, and from flowing over the tops of slopes. The surface water should be directed toward adequate drainage facilities. Ponding of surface water should not be allowed next to structures or on pavements. Design criteria for finish lot drainage away from structures and off the lot should be determined by the project Structural Engineer designing the foundations and slabs in conjunction with the project Civil Engineer designing the precise grading for lot drainage, respectively, in accordance with the 2016 CBC and/or the current City of Redlands, California codes and ordinances and the earth material types and expansion characteristics for the earth materials contained in this report. Finished landscaped and hardscape or pavement grades adjacent to the proposed structures should maintain a vertical distance below the bottom elevation of the weep screed per the 2016 CBC and/or the current City of Redlands codes and ordinances. Landscape plants with high water needs and trees should be planted at a distance away from the structure equivalent to or greater than the width of the canopy of the mature tree or 6.0 feet, whichever is greater. Downspouts from roof drains should discharge to a permanent all-weather surface which slopes away from the structure. Downspouts from roof drains should not discharge into planter areas immediately adjacent to the building unless there is positive drainage out of the planter and away from the structure in accordance with the recommendations of the project foundation and slab designer and/or the project Civil Engineer designing the precise grades for the lot drainage.

PLANTER RECOMMENDATIONS

Planters around the perimeter of the structures should be designed so that adequate drainage is maintained and minimal irrigation water is allowed to percolate into the earth materials underling the buildings. This should include enclosed or trapped planter areas that are created as a result of sidewalks. Planters with solid bottoms, independent of the underlying earth material, are recommended within a distance of 6.0 feet from the buildings. The planters should drain directly onto surrounding paved areas or into a designed subdrain system. If planters are raised above the surrounding finished grades or are placed against the building structure, the interior walls of the planter should be waterproofed.

INFILTRATION RECOMMENDATIONS

Location of Shallow Percolation Tests

The shallow percolation test boring locations were located within the proposed infiltration area, in the existing parking lot. The approximate percolation test locations are shown on the ‘Exploratory Excavation Location Plan,’ Plate No. 1, presented in Appendix ‘A.’

Earth Material Characteristics of the Subject Site

- The earth material characteristics for the subject site are defined as moderately favorable.

- Clayey, moderately favorable soil conditions are anticipated for the infiltration system.
- There was no visible evidence of shallow groundwater or impervious bedrock materials.
- Tests performed agreed with the visual evidence.
- The existing pavement surface in the infiltration area is sloping at an approximate 3.5 percent gradient.

Number of Exploratory Borings

- Four (4) exploratory borings and two (2) shallow percolation borings were drilled in the proposed infiltration area.
- The materials underlying the subject site consisted of artificial fill over native clayey sands and gravelly clayey sands (Old Axial Valley Deposits). The earth material was generally moist and loose to medium dense in consistency. The infiltration borings were terminated at approximately 5.67 feet below existing grade, and the deepest exploratory boring was excavated to 42.0 feet.

Earth Material Profile

- The earth materials encountered in the exploratory borings in the infiltration area are described on the 'Subsurface Exploration Logs,' Plate Nos. 11 and 12, presented in Appendix 'A.'
- No low permeability layers were observed.
- The alluvial soils in the exploratory borings were classified in general accordance with the Unified Soil Classification System as SC.
- All colors described on the boring logs were moist earth material colors. There was no reduction-oxidation mottling observed in the exploratory borings.
- No roots were noted in our percolation test holes.

- There were no wet or saturated earth material encountered in the subsurface exploration borings.

- No groundwater was encountered on the site.

Percolation Testing Procedures

Test Borings:

- The exploratory borings were performed by using a truck-mounted drill rig equipped with 8-inch outside diameter, hollow stem augers. The exploratory excavations were explored to a depth 5.67 feet below the existing ground surface at the excavation locations. The bottom of the borings were in natural, undisturbed earth material.

- Slotted PVC pipe, 3.25 inches in diameter, was installed in the boring excavation through the center of the hollow stem auger prior to removing the augers. Gravel was placed around the outside of the pipe after removal of the augers from the boring.

Pre-Soak:

- Soaking Period: The test borings were pre-soaked with water beginning in the afternoon on January 4, 2018, and ending when the percolation testing began on January 5, 2018.

- Soaking Method: A 2-inch hose was used to fill the holes from a 300-gallon water tank that was carried on a pickup truck. The hose was placed into the pipe and each hole was filled to the approximate surface of the ground to a few inches below.

Percolation Measurement:

- Testing was performed on January 5, 2018.

- Each boring was filled to the approximate surface to commence the percolation testing. After setting the initial water level, the drop in the water depth was measured and recorded at 25 minute intervals. In percolation test P-2, more than half the wetted depth percolated through the test hole over two timed 25 minute intervals, and therefore the test was run for an additional one hour with a refill of the percolation hole after each 10 minute reading. Percolation test P-1 percolated for one 25 minute interval and was found to have under 6 inches of water seep away. Percolation test P-1 was then tested for a period of approximately six (6) hours with a refill and measurement after each 30 minute reading. Calculations were based on the above recorded readings at approximate 30-minute and 10-minute intervals.

Percolation Test Results

Detailed percolation test results, in general accordance with San Bernardino County Technical Guidance Document Appendices, are included in Appendix 'C' as Plate Nos. 13 and 14. Percolation Rates were converted to Infiltration Rates utilizing the Porchet Method, the average and steady state infiltration rates given in (cm/hr) are listed below:

INFILTRATION TEST RESULTS

TEST BORING NO.	BOTTOM OF TEST DEPTH (ft.)	SOIL CLASS-IFICATI-ON	AVERAGE INFILTRATION RATE (cm / hr)	STEADY STATE INFILTRATION RATE (cm/hr)
P-1	5.67	SC	2.276	2.362
P-2	5.67	SC	17.447	13.744

The earth materials within the test borings were observed to be older alluvial soils that were classified in general accordance with the Uniform Soils Classification system as SC and SC/SM. Percolation test P-1 contained silty to clayey, fine to medium sand. The clayey, fine grained nature of the deposits caused the percolation rates to be slow to moderate. Percolation test P-2 contained slightly clayey, fine to coarse grained sand, with the majority of the boring containing gravel up to 4 inches in dimension. The coarser nature and the location of the gravels (generally in the lower portion of the test boring) allowed the percolation rates to be fast to moderate. In the design phase, the slowest percolation rate should be utilized in the infiltration design. No groundwater or impermeable layers were encountered in the percolation test borings. A small amount of materials caved around the slotted pipe, particularly in test hole P-1 over the 6 hour time period, but did not appear to interfere with the percolation rates or test.

Caution should be used in determining a percolation rate for infiltration systems. Eventual siltation can drastically reduce the percolation rate over time. We recommend that suitable methods to prevent siltation be incorporated in the project design.

LIMITATIONS

REVIEW, OBSERVATION, AND TESTING

The recommendations presented in this report are contingent upon review of final plans and specifications for the project by **HGI**. The project Geotechnical / Geologic Consultant should review and verify in writing the compliance of the final grading plan and the final foundation plans with the recommendations presented in this report.

It is recommended that **HGI** be retained to provide continuous Geotechnical / Geologic Consulting services during the earthwork operations (i.e., rough grading, utility trench backfill, subgrade preparation for slabs-on-grade and pavement areas, finish grading, etc.) and foundation installation process. This is to observe compliance with the design concepts, specifications and recommendations and to allow for design changes in the event that subsurface conditions differ from those anticipated prior to start of construction. If **HGI** is replaced as Geotechnical / Geologic Consultant of record for the project, the work on the project should be stopped until the replacement Geotechnical / Geologic Consultant has reviewed the previous reports and work performed for the project, agreed in writing to accept the recommendations and prior work performed by **HGI** for the subject project, or has submitted their revised recommendations.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our understanding of the project requirements based on an evaluation of subsurface earth material conditions encountered at the subsurface exploration locations and the assumption that earth material conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations may be influenced by undisclosed or unforeseen variations in earth material conditions that may occur in intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the **HGI** so that we may make modifications, if necessary.

CHANGE IN SCOPE

HGI should be advised of any changes in the project scope of proposed site grading so that it may be determined if recommendations contained herein are valid. This should be verified in writing or modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the State-of-the-Art and/or government codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two (2) years without a review by **HGI** verifying the validity of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with the standard of care and skill ordinarily exercised under similar circumstances by members of the geologic / geotechnical professions currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our surveys and exploratory excavations were made, and that our data, interpretations, and recommendations are based solely on information obtained by us. We will be responsible for those data, interpretations, and recommendations, but should not be responsible for interpretations by others of the information presented and/or developed. Our services consist of professional consultation and observation only, and other warranties, expressed or implied, are not made or intended in connection with work performed by **HGI** or by the proposal for consulting or other services or by the furnishing of oral or written reports or findings.

CLIENT'S RESPONSIBILITY

It is the responsibility of the client and/or the client's representatives to ensure that information and recommendations contained herein are brought to the attention of the Engineers and Architect for the project and incorporated into project plans and specifications. It is further their responsibility to take measures so that the contractor and his subcontractors carry out such recommendations during construction.

APPENDIX A

FIELD EXPLORATION

The field study performed for this report included a visual reconnaissance of existing surface conditions of the subject site and surrounding area. Site observations were conducted on January 4th and 5th, 2018 by a representative of **HGI**. The aerial distribution of the earth materials observed is shown on the 'Exploratory Excavation Location Plan,' Plate No. 1, presented in this Appendix.

A study of the property's subsurface condition was performed to evaluate underlying earth strata and the presence of groundwater. Four (4) exploratory borings and (2) percolation test excavations were performed on the on the subject site on January 4, 2018. Locations of the exploratory excavations were determined in the field by pacing, tape measuring, and sighting from the adjacent existing streets, adjacent structures, and topographic features as shown on the Reference No. 1, 'Site Plan,' noted on the first page of the cover letter for this. Approximate locations of the exploratory excavations are denoted on the 'Exploratory Excavation Location Plan,' Plate No. 1, presented in this Appendix. Approximate elevations at the locations of the exploratory excavations were determined from the Google Earth Website (http://www/google.com/earth). Locations and elevations of the exploratory excavations should be considered accurate only to the degree implied by the method used in determining them.

The exploratory borings were performed by using a truck-mounted drill rig equipped with 8-inch outside-diameter, hollow-stem augers. The exploratory excavations were explored to depths ranging from approximately 5.67 to 42.0 feet below existing ground surface at the excavation locations. Bulk and relatively undisturbed samples of encountered earth materials were obtained at various depths in the exploratory excavations and returned to our laboratory for testing and verification of field classifications. Bulk samples were obtained from cuttings developed during the excavation process and represent a mixture of earth materials within the depth indicated on the logs. Relatively undisturbed samples of encountered earth materials were obtained by driving a thin-walled, steel sampler lined with 1-inch high, 2.416-inch inside diameter brass rings. The sampler was driven with successive drops of a 140-pound weight having a free fall of approximately 30 inches. Blow counts for each successive 6.0 inches of penetration, or fraction thereof, are shown on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in this Appendix. Ring samples were retained in close-fitting moisture-proof containers and returned to our laboratory for testing. Standard Penetration Tests were also performed at various depths in the borings. The test was performed in general accordance with current American Society of Testing Materials (ASTM) D1586 procedures using a standard penetration sampler (2.0-inch outside diameter, 1.375-inch inside diameter) driven with a 140 weight dropping 30 inches. The blow counts to drive the sampler for three (3) successive 6.0 inch intervals are recorded on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in this Appendix. The standard penetration resistance ('N' value) is the sum of the blow counts for the last two (2) 6.0 inch intervals.

Groundwater observations were made during, and at the completion of the excavation process and are noted on the 'Subsurface Exploration Log' presented in this Appendix, if encountered.

The exploratory excavations were logged by a representative of **HGI** for the existing pavement section thickness, fill material, natural earth material, and subsurface conditions encountered. Earth materials encountered in the exploratory excavations were visually described in the field in general accordance with the current Unified Soils Classification System (USCS), ASTM D2488, visual-manual procedures, as illustrated on the attached, simplified 'Subsurface Exploration Legend,' Plate No. 2, presented in this Appendix. The visual textural description, color of the earth material at natural moisture content, apparent moisture condition of the earth materials, and apparent relative density or consistency of the earth materials, etc., were recorded on the field logs. The 'Relative Density' of granular soils (SP, SW, SM, SC, GP, GW, GM, GC) is given as very loose, loose, medium dense, dense, or very dense and is based on the number of blows to drive the sampler 1.0 foot or fraction thereof. The 'Consistency' of silts or clays (ML, CL, MH, CH) is given as very soft, soft, medium stiff, stiff, very stiff, or hard and is also based on the number of blows to drive the sampler 1.0 foot or fraction thereof. The field log for each excavation contains factual information and interpretation of earth material conditions between samples. The 'Subsurface Exploration Log' presented in this Appendix represent our interpretation of the field log contents and results of laboratory observations and tests performed on samples obtained in the field from the exploratory excavations.

Perforated pipe was installed in the two (2) borings in the proposed infiltration area. The pipe was installed for use in performing percolation tests in this area of the subject site. The remaining exploratory boring excavations were backfilled with excavated earth materials and with reasonable effort to restore the areas to their initial condition before leaving the site but were not compacted to a relative compaction of 90 percent or greater. In an area as small and deep as a boring excavation, consolidation and subsidence of backfill earth material may result in time, causing a depression of the excavation areas. The client is advised to observe exploratory excavation areas periodically and, when needed, backfill noted depressions.

Percolation tests were performed in the infiltration area in general accordance with **County of San Bernardino**, May 19, 2011, *Technical Guidance Document Appendices, Appendix VII., Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations.*

LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected, relatively undisturbed ring and bulk samples obtained from exploratory excavations during the field study. Tests were performed in general accordance with generally accepted American Society for Testing and Materials (ASTM), State of California - Department of Transportation (CALTRANS), Environmental Protection Agency (EPA) or other suitable test methods or procedures. The remaining samples obtained during the field study will be discarded 30 days after the date of this report. This office should be notified immediately if retention of samples will be needed beyond 30 days. A brief description of the tests performed is presented below:

CLASSIFICATION

The field classification of earth material materials encountered in the exploratory excavations was verified in the laboratory in general accordance with the current Unified Soils Classification System, ASTM D2488, 'Standard Practice for Determination and Identification of Soils (Visual-Manual Procedures).' The final classification is shown on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in this Appendix.

IN-SITU MOISTURE CONTENT AND DRY DENSITY

The in-situ moisture content and dry density were determined in general accordance with current ASTM D2216 (Moisture Content) and D2937 (Drive Cylinder) procedures, respectively, for selected undisturbed samples obtained. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry density is determined in pounds per cubic foot and the moisture content is determined as a percentage of the oven dry weight of the earth material. Test results are shown on the 'Subsurface Exploration Log,' Plate Nos. 3a through 6, presented in this Appendix.

EXPANSION TEST

A laboratory expansion test was performed on a selected sample of near-surface earth material in general accordance with the current ASTM D4829 procedures. In this testing procedure, a remolded sample is compacted in two (2) layers in a 4-inch inside diameter mold to a total compacted thickness of approximately 1.0 inch by using a 5.5-pound weight dropping 12 inches and with 15 blows per layer. The sample should be compacted at a saturation between 48 and 52 percent. After remolding, the sample is confined under a pressure of 144 pounds per square foot (psf) and allowed to soak for 24 hours. The resulting volume change due to the increase in moisture content within the sample is recorded and the Expansion Index (EI) calculated. The test results are summarized in the 'Summary of Laboratory Test Results,' Plate No. 7, presented in this Appendix.

SOLUBLE SULFATE TEST

The concentration of soluble sulfate was determined on a selected sample of near-surface earth material in general accordance with current EPA 300.0 procedures. The test results are summarized in the 'Summary of Laboratory Test Results,' Plate No. 7, presented in this Appendix.

SIEVE ANALYSIS

The percent by weight finer than a No. 200 sieve (silt and clay content) was determined for a selected sample of earth material in general accordance with current ASTM D1140 procedures. The test is performed by taking a known weight of an oven dry sample of earth material, washing it over a No. 200 sieve, and oven drying the earth material retained on the No. 200 sieve. The dry weight of earth material retained on the No. 200 sieve is measured and the resulting percentage retained is calculated based on the original total dry earth material sample weight. The percent passing the No. 200 sieve is determined by subtracting the percent retained from 100. The test results are summarized in the 'Summary of Laboratory Test Results,' Plate No. 7, presented in this Appendix.

CHEMICAL AND MINIMUM ELECTRICAL RESISTIVITY

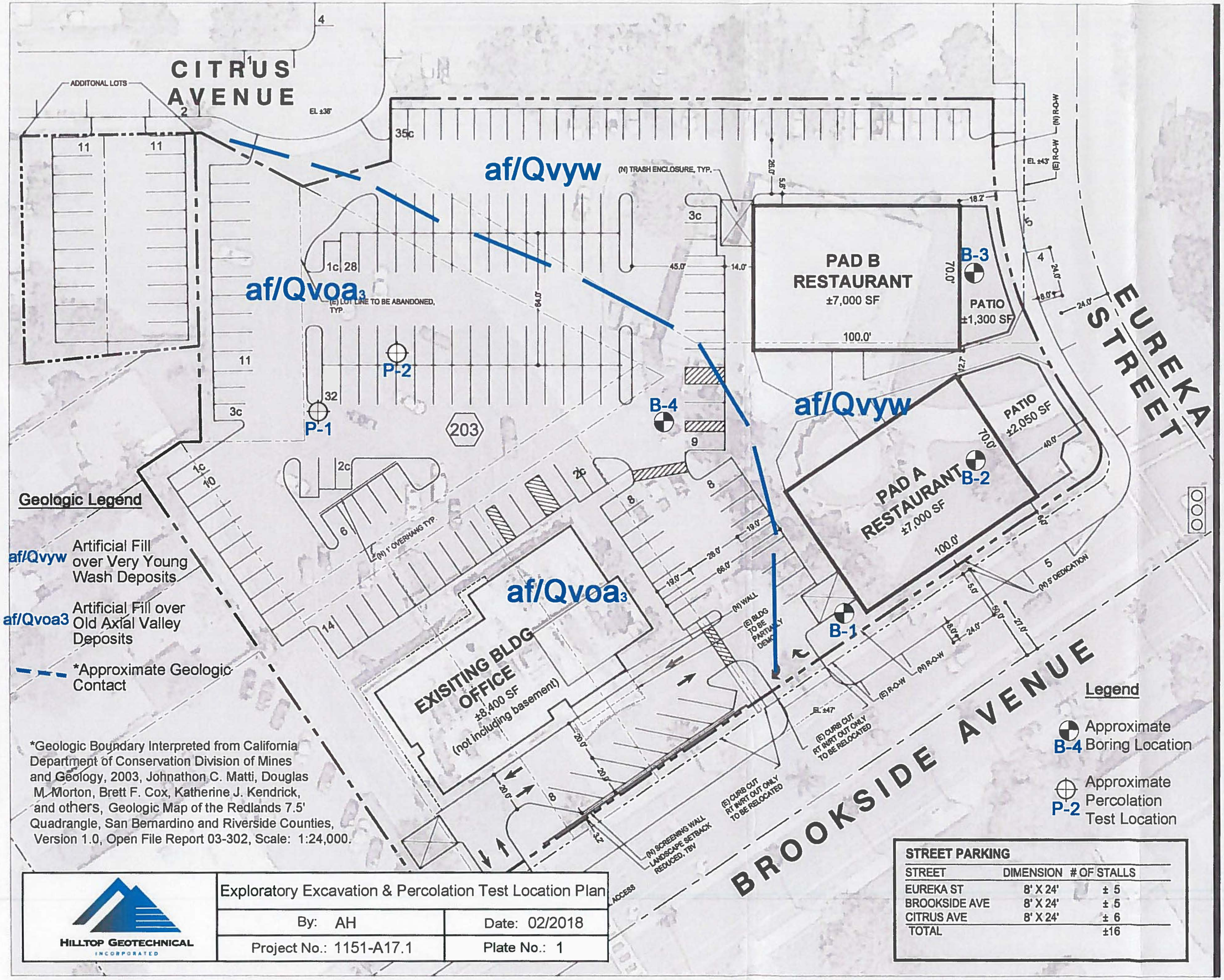
The concentration of soluble chloride, pH, as well as other chemical constituents and the minimum electrical resistivity were determined for a selected sample of near-surface earth material. The pH test was performed in general accordance with current EPA 9045C procedures. The Chloride test was performed in general accordance with current EPA 300.0 procedures. The test results are summarized in the 'Summary of Laboratory Test Results,' Plate No. 8, presented in this Appendix.

CONSOLIDATION TESTS

Hydroconsolidation or the Collapse Potential, I_C, of the on-site earth material behavior under load were made on the basis of consolidation tests that were performed on selected relatively undisturbed ring samples of the alluvial soils in general accordance with current ASTM D5333 procedures. The consolidation apparatus is designed to receive a 1-inch high, 2.416-inch diameter ring sample. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore water. A load of 1,600 pounds per square foot (psf) was applied normal to the face of the specimen at field moisture condition and the sample was allowed to consolidate. Upon completion of the consolidation process, water was added to the test apparatus to create a submerged condition and to measure the collapse (hydroconsolidation) or expansion potential of the sample. The resulting change in sample thickness was recorded. The test results are summarized in the 'Summary of Laboratory Test Results,' Plate No. 8, presented in this Appendix.

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE CONTENT RELATIONSHIP TEST

A maximum dry density / optimum moisture content relationship determination was performed on a sample of near-surface earth material in general accordance with current ASTM D1557 procedures using a 4-inch diameter mold. Samples were prepared at various moisture contents and compacted in five (5) layers using a 10-pound weight dropping 18 inches and with 25 blows per layer. A plot of the compacted dry density versus the moisture content of the specimens was constructed and the maximum dry density and optimum moisture content determined from the plot. The test results are summarized in the 'Maximum Dry Density / Optimum Moisture Content Relationship Test Results,' Plate No. 9, presented in this Appendix.



Geologic Legend

af/Qvyw	Artificial Fill over Very Young Wash Deposits
af/Qvoa3	Artificial Fill over Old Axial Valley Deposits
— —	*Approximate Geologic Contact

*Geologic Boundary Interpreted from California Department of Conservation Division of Mines and Geology, 2003, Johnathon C. Matti, Douglas M. Morton, Brett F. Cox, Katherine J. Kendrick, and others, Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, Version 1.0, Open File Report 03-302, Scale: 1:24,000.

Legend

B-4	Approximate Boring Location
P-2	Approximate Percolation Test Location

STREET PARKING

STREET	DIMENSION	# OF STALLS
EUREKA ST	8' X 24'	± 5
BROOKSIDE AVE	8' X 24'	± 5
CITRUS AVE	8' X 24'	± 6
TOTAL		±16

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Exploratory Excavation & Percolation Test Location Plan	
By: AH	Date: 02/2018
Project No.: 1151-A17.1	Plate No.: 1

GreenbergFarrow
19000 MacArthur Blvd, Suite 250
Irvine, CA 92612
t: 949 296 0450 f: 949 296 0479

PROJECT INFORMATION

VICINITY MAP

SITE AREA

NET SITE AREA	±2.75 AC
ADDITIONAL LOTS	±0.21 AC
STREET DEDICATIONS	±0.07 AC
GROSS SITE AREA	±3.03 AC

BUILDING AREA

PAD A	±7,000 SF
PAD B	±7,000 SF
EXISTING BLDG* (not incl basement)	±8,000 SF
TOTAL BLDG AREA	±22,000 SF
SITE COVERAGE	±18.7% (±8,145 SF/AC)

* EXISTING BLDG AREA IS ±8,400 SF NOT INCLUDING BASEMENT. PROPOSING PARTIALLY DEMO.

PARKING SUMMARY

USER	RATIO REQUIRED	SPACES REQ'D	SPACES PROV'D
PAD A - RESTAURANT	**	78	
PAD A - RESTAURANT	**	78	
EX. BLDG - MEDICAL OFFICE	1 SP/ 200 SF	40	
STANDARD			148
COMPACT	25% ALLOWED	(±23.15 %)	47
HANDICAPPED			8
TOTAL		196	203
TOTAL RATIO PROVIDED			9.23 SP/1000 SF
STREET PARKING AVAILABLE			±16 STALLS

** PARKING REQUIRED FOR SIT DOWN RESTAURANTS BASED ON 1 SP/3 SEATS OR 1 SP/50 SF OF SERVING AREA WHICHEVER IS LARGER PLUS 1 SP/2 EMPLOYEES. SERVING AREA SSUMED 50% OF BLDG AREA.

ZONING INFORMATION

JURISDICTION	CITY OF REDLANDS, CA
EXISTING ZONING:	A-P DISTRICT - ADMINISTRATIVE & PROFESSIONAL OFFICE DISTRICT W/CIVIC DESIGN DISTRICT OVERLAY
PROPOSED ZONING:	C-3 - GENERAL COMMERCIAL

PROJECT NOTES

1. THIS PLAN IS BASED ON A PDF OF THE ASSESSOR'S PARCEL MAP AND AN AERIAL.
2. THIS PLAN IS A CONCEPTUAL SITE PLAN IS FOR PLANNING PURPOSES ONLY.

DRAWING ISSUE/REVISION RECORD

DATE	NARRATIVE	INITIALS
01.29.2016	PREPARING SP-1	II
02.19.2016	PREPARING SP-2	MM
12.08.2016	PREPARING SP-3	II
12.11.2016	PREPARING SP-4	II
02.21.2017	PREPARING SP-4 PARKING STUDY	II/KQ
03.06.2017	PREP SP-5	II

CLIENT	VANTAGE ONE
CLIENT REPRESENTATIVE	TOM ROBINSON
PROJECT MANAGER/DESIGNER	II
SITE DEV. COORDINATOR	FRANK CODA

REDLANDS, CA
NWC BROOKSIDE AVE & EUREKA ST

GFA PROJECT NUMBER	20160103.0

SCALE: 1"=20'-0"
0 10 20 40 60 80

SP-5

SUBSURFACE EXPLORATION LEGEND

UNIFIED SOIL CLASSIFICATION SYSTEM
Visual-Manual Procedure (ASTM D2488-09a)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
Coarse-Grained Soils* More than 50 % Retained on No. 200 Sieve	Gravels 50 % or more of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels	GW	Well Graded Gravels and Gravel-Sand Mixtures, Little or no Fines
			GP	Poorly Graded Gravels and Gravel-Sand Mixtures, Little or no Fines
		Gravels with Fines	GM	Silty Gravels, Gravel-Sand-Silt Mixtures**
			GC	Clayey Gravel, Gravel-Sand-Clay Mixtures**
	Sands More than 50 % of Coarse Fraction Passes No. 4 Sieve	Clean Sands	SW	Well Graded Sands and Gravely Sands, Little or no Fines
			SP	Poorly Graded Sands and Gravelly Sands, Little or no Fines
		Sands with Fines	SM	Silty Sands, Sand-Silt Mixtures**
			SC	Clayey Sands, Sand-Clay Mixtures**
Fine Grained Soils* 50 % or more Passes No. 200 Sieve	Silts and Clays Liquid Limits 50 % or less		ML	Inorganic Silts, Sandy Silts, Rock Flour
			CL	Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			OL	Organic Silts and Organic silty Clays of Low Plasticity
	Silts and Clays Liquid Limits Greater than 50 %		MH	Inorganic Silts, Micaceous or Diatomaceous silts, Plastic Silts
			CH	Inorganic Clays of High Plasticity, Fat Clays
			OH	Organic Clays of Medium to High Plasticity
Highly Organic Soils			PT	Peat, Muck, or Other Highly Organic Soils

CONSISTENCY / RELATIVE DENSITY

CRITERIA

Reference: 'Foundation Engineering', Peck, Hansen, Thornburn, 2nd Edition.

Standard Penetration Test
Granular Soils

Penetration Resistance, N, (Blows / Foot)	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
> 50	Very Dense

Standard Penetration Test
Cohesive Soils

Penetration Resistance, N, (Blows / Foot)	Consistency	Unconfined Compressive Strength, (Tons / Sq. Ft.)
< 2	Very Soft	< 0.25
2 - 4	Soft	0.25 - 0.5
5 - 8	Firm (Medium Stiff)	0.5 - 1.0
9 - 15	Stiff	1.0 - 2.0
16 - 30	Very Stiff	2.0 - 4.0
> 31	Hard	> 4.0

* Based on material passing the 3-inch sieve.

** More than 12% passing the No. 200 sieve; 5% to 12% passing No. 200 sieve requires use of duel symbols (i.e., SP-SM., GP-GM, SP-SC, GP-GC, etc.); Border line classifications are designated as CH/Cl, GM/SM, SP/SW, etc.

U.S. Standard Sieve Size 12" 3" 3/4" #4 #10 #40 #200

Unified Soil Classification Designation	Boulders	Cobbles	Gravel		Sand			Silt and Clay
			Coarse	Fine	Coarse	Medium	Fine	

Moisture Condition

Dry	Absence of moisture, dusty, dry to the touch.
Moist	Damp but no visible moisture.
Wet	Visible free water, usually below the water table.

Material Quantity

Trace	< 5 %
Few	5 - 10%
Little	15 - 25%
Some	30 - 45 %

Other Symbols

C - Core Sample
S - SPT Sample
B - Bulk Sample
CK - Chunk Sample
R - Ring Sample
N - Nuclear Gauge Test
∇ - Water Table

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(Revised 11-23-2015)

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

Project Name: Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA

Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1346
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	42.0

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
	R	21	SC			af		ARTIFICIAL FILL:
1		37						Clayey fine to caorse sand, trace gravel, slightly porous; Red brown; Moist; Very dense.
		38		113.8	4.6			
2			SC					Clayey fine to medium sand trace coarse, trace silt, trace fine rootlets; Red brown; Moist; Medium dense.
3	S	6						
		6						
4		9						
5			SC					Clayey fine to coarse sand, trace silt, trace metal fragments; Red brown; Moist; Medium dense.
	R	12						
6		19						
		19		114.0	3.9			
7			SM			Qvyw		VERY YOUNG WASH DEPOSITS:
8	S	1						Silty fine sand, trace medium sand; Pale brown; Moist; Loose to medium dense.
		3						
9		3						
10	R	4						Trace clay, slighty porous.
11		8						
		9		115.0	5.4			
12			SM					Gravelly to silty fine to coarse sand; Gray brown; Moist; Medium dense. 3 fragments of rock larger than sampler.
13	S	11						
		14						
14		13						
15			SP/SM					Slightly silty fine to medium sand, trace coarse sand; Gray brown; Moist; Medium dense.
	R	14						
16		11						
		12		105.7	3.7			
17								
18								
19								
20	S	9						
21		11						
		12						
22								
23			SP					Gravelly fine to coarse sand, trace cobbles on flights; Gray brown; Moist; Dense.
24								
25								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

Plate No. 3a

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

Project Name: Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA

Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1346
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	42.0

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
			SP			Qvyw		VERY YOUNG WASH DEPOSITS: Gravelly fine to coarse sand, trace cobbles on flights; Gray brown; Moist; Dense. Cobble encountered inside augers. No sample taken.
26								
27								
28			SP/SM					Slightly silty, fine to coarse sand, trace gravel; Gray orange brown; Moist; Dense to very dense.
29								
30	R	18 24 26						
31								
32								
33								
34								
35	R	50						
36								
37								
38								
39								
40	R	36 50/2"						
41								
42								Bottom of boring 42.0 feet due to refusal on cobbles. No groundwater encountered. Boring was backfilled with excavated materials.
43								
44								
45								
46								
47								
48								
49								
50								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

HILLTOP GEOTECHNICAL
INCORPORATED

SUBSURFACE EXPLORATION LOG
BORING NO. B-2

Project Name:	Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA				
Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1343
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	21.5

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
1 2	S/B	9 11 10	SC			af		ARTIFICIAL FILL: Clayey fine to coarse sand; Red brown; Moist; Medium dense.
3 4	R	11 13 13		101.8	14.8			
5 6	S	3 3 2	SM					Silty fine to medium sand, trace coarse sand; trace clay brick in sample; Dark red brown; Moist; Loose.
7 8 9	R	3 3 3	SC	108.9	8.6	Qvyw		VERY YOUNG WASH DEPOSITS: Clayey fine to medium sand, a little silt; Red brown; Moist; Loose to medium dense.
10 11 12	S	4 8 8						
13 14 15 16 17 18	S	7 10 13	SM					Silty fine to medium sand, trace coarse sand, trace clay; Red brown; Moist; Medium dense.
19 20 21	R	18 24 25	SP/SM	115.3	4.2			Slightly silty fine to coarse sand, trace gravel; Light reddish gray brown; Moist; Dense.
22 23 24 25								Bottom of boring 21.5 feet. No groundwater encountered. Boring backfilled with excavated materials.

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

HILLTOP GEOTECHNICAL
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SUBSURFACE EXPLORATION LOG
BORING NO. B-3

Project Name:	Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA				
Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1342
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	26.5

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
1	R	8 17 17	SM	122.8	3.7	af		ARTIFICIAL FILL: Silty fine to coarse sand, trace gravel; Brown; Moist; Medium dense.
2			SC					Clayey fine to coarse sand, trace silt; Red brown; Moist; Medium dense.
3 4	S	5 13 13						
5			SP/SM			Qvyw		VERY YOUNG WASH DEPOSITS: Slighty silty fine to medium sand; Dark brown; Moist; Medium dense.
6	R	9 11 12		114.6	3.8			
7			SP/SM					Slightly silty fine sand, trace medium sand; Gray brown; Moist; Loose.
8 9	S	4 5 5						
10			SP					Fine to coarse sand, a little gravel, trace silt; Gray brown; Moist; Dense to medium dense.
11	R	15 22 25		127.4	2.7			
12								
13 14	S	8 5 5						
15 16	R	4 8 9						
17 18 19								
20 21	S	12 19 14						
22								
23 24 25			SC					Clayey fine to coarse sand, trace gravel; Red brown; Moist; Dense.

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

Plate No. 5a

HILLTOP GEOTECHNICAL
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SUBSURFACE EXPLORATION LOG
BORING NO. B-3

Project Name:	Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA				
Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1342
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	26.5

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
	R	10 19 37	SC			Qvyw		VERY YOUNG WASH DEPOSITS: Clayey fine to coarse sand, trace gravel; Red brown; Moist; Dense.
26								
27								Bottom of boring 26.5 feet. No groundwater encountered. Boring was backfilled with excavated materials.
28								
29								
30								
31								
32								
33								
34								
35								
36								
37								
38								
39								
40								
41								
42								
43								
44								
45								
46								
47								
48								
49								
50								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

HILLTOP GEOTECHNICAL
INCORPORATED

SUBSURFACE EXPLORATION LOG
BORING NO. B-4

Project Name:	Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA				
Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1344
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	21.5

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
			SM			af		3" Inches Hot Mix Asphalt (HMA) / 0 Base ARTIFICIAL FILL: Silty fine to medium sand, trace clay; Red brown; Moist; Very loose to loose.
1	R	2						
		1						
2		1						
	R	2						
3		2						
		2		102.8	10.5			
4								
5	S	2						
6		2						
		3						
7			SC			Qvoa3		VERY OLD AXIAL VALLEY DEPOSITS: Clayey fine to coarse sand; Red brown; Moist; Medium dense.
8	R	5						
		6						
9		10		116.0	9.5			
10	S	7						
11		8						
		7						
12			SP/SM					Slightly silty fine to medium sand, trace coarse sand; Gray red brown; Moist; Medium dense.
13	R	6						
		7						
14		7		113.9	5.5			
15	S	10						
16		17						
		12						
17								
18								
19			SP/SM					Slightly silty fine to coarse sand, trace gravel; Light reddish gray brown; Moist; Dense.
20	R	12						
21		11						
		12	SM					Silty fine sand, trace clay; Red brown; Moist; Medium dense.
22								Bottom of boring 21.5 feet. No groundwater encountered. Boring backfilled with excavated materials.
23								
24								
25								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

SUMMARY OF LABORATORY TEST RESULTS

EXPANSION INDEX TEST RESULTS (ASTM D4829 Test Method)

SAMPLE NO.	MOISTURE CONTENT PRIOR TO TEST (to 0.1%)	DRY DENSITY PRIOR TO TEST (to 0.1 pcf)	SATURATION PRIOR TO TEST (to 0.1% between 48% & 52%)*	MOISTURE CONTENT AFTER TEST (to 0.1%)	EXPANSION INDEX	EXPANSION POTENTIAL**
B-2, 0-4.0'	7.4	119.7	49.0	13.0	5	Non-Expansive

* Assumes a 2.70 Specific Gravity for the earth material.

** As defined in Section 1803.5.3, 'Expansive Soil,' in the 2016 California Building Code (CBC) (i.e., Non-Expansive: EI ≤20; Expansive: EI >20).

SOLUBLE SULFATE TEST RESULTS (EPA 300.0 Test Procedure)*

SAMPLE	SOLUBLE SULFATE CONTENT (%)	CLASS**
B-2, 0-4.0'	<0.0021	S0

* Test performed by **A & R Laboratories**.

** Per Table 19.3.1.1, 'Exposure Categories and Classes,' in **American Concrete Institute** (ACI) 318-14.

PERCENT PASSING #200 SIEVE TEST RESULTS (ASTM D1140 Test Method)

SAMPLE	EARTH MATERIAL DESCRIPTION	PERCENT PASSING #200 SIEVE
B-2, 0-4.0'	Clayey, fine to coarse sand; Red brown (SC)	29.7

SUMMARY OF LABORATORY TEST RESULTS

CHEMICAL / MINIMUM ELECTRICAL RESISTIVITY TEST RESULTS

SAMPLE	RESISTIVITY Minimum (ohm-cm)	pH*	SULFIDE	CHLORIDE (ppm)*
B-2, 0-4.0'	5,561	8.21	Neg.**	23

* Test performed by **A & R Laboratories** in accordance with EPA 300.0 procedures.

** Neg. - Negative.

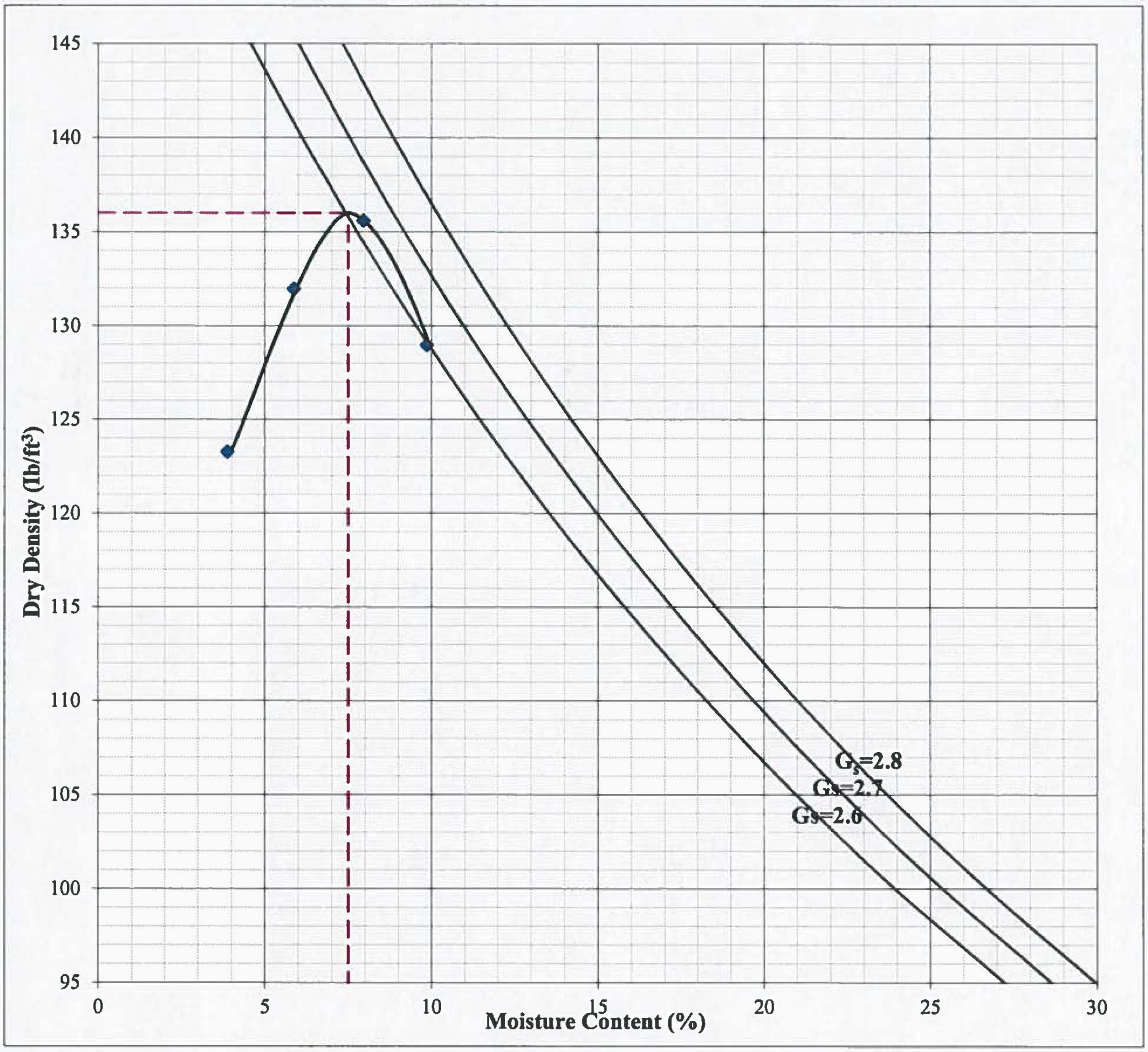
COLLAPSE POTENTIAL TEST RESULTS (ASTM D5333 Test Method)

SAMPLE	SETTLEMENT AT 1,600 PSF LOAD (%)	COLLAPSE / SWELL* (%)	COLLAPSE INDEX, (I_C), (%)	DEGREE OF COLLAPSE**
B-1, 10.0'	1.1	7.0	7.0	Moderately Severe
B-1, 10.0'	1.1	8.1	8.1	Moderately Severe
B-2, 2.5'	1.0	0.7	0.7	Slight

* Percent collapse (-) or swell (+) measured when water added at 1,600 psf load during test procedure.

** Per Table 1, 'Classification of Collapse Index, I_C,' in ASTM Standard Test Method D5333-03.

None - 0%
Slight - 0.1 - 2.0%
Moderate - 2.1 - 6.0%
Moderately Severe - 6.1 - 10.0%
Severe - >10.0%



Maximum Dry Density (lb/ft^3)	136.0
Optimum Moisture Content (%)	7.5
Procedure	B

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MAXIMUM DRY DENSITY / OPTIMUM MOISTURE CONTENT RELATIONSHIP TEST RESULTS
(ASTM D1557 Test Method)

SAMPLE: B-2, 0-4.0'

SOIL DESCRIPTION:	Clayey, fine to coarse sand; Red brown (SC)		
BY:	SS	DATE:	2/2018
JOB NO.:	1151-A17.1	PLATE NO.:	9

APPENDIX B

TECHNICAL REFERENCES

American Society of Civil Engineers, 2010, *Minimum Design Loads for Buildings and Other Structures*: ASCE Standard No. 7-10.

Blake, Thomas F., 2000, *Preliminary Fault-Data for EQFAULT, EQSEARCH and FRISKSP.*

Blake, Thomas, F., *Computer Services and Software, Users Manuals, FRISKSP v. 4.00, EQSEARCH v. 3.00, and EQFAULT v. 3.00.*

California Building Standards Commission, Effective January 1, 2017, *2016 California Building Code*: California Code of Regulations, Title 24, Part 2, Volume 1 of 2 and Volume 2 of 2 (Based on 2012 International Building Code).

California Department of Conservation, Division of Mines and Geology, *Guidelines to Geologic/Seismic Reports*: CDMG Note 42.

California Department of Conservation, Division of Mines and Geology, *Guidelines for Preparing Engineering Geologic Reports*: CDMG Note 44.

California Department of Conservation, California Geological Survey, Interim Revision 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones[1] Maps*, ([1]Name Changed from 'Special Studies Zones,' January 1, 1994.): Special Publication 42.

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

California Department of Conservation, Division of Mines and Geology, 1976, *Geologic Hazards in Southwestern San Bernardino County, California*: Special Report 113.

California Department of Conservation, Division of Mines and Geology, 1990, *Index to Fault Evaluation Reports Prepared 1976-1989 Under the Alquist-Priolo Special Studies Zone Act*: CDMG Open-File Report 90-9.

TECHNICAL REFERENCES

California Department of Conservation, Division of Mines and Geology, 2003, Morton, D.M., Miller, F.K., (Digitally Prepared by Cossette, P.M., Bovard, K.R.); *Preliminary Geologic Map of the San Bernardino 30' x 60' Quadrangle, California*; Version 1.0, Scale 1:100,000.

California Department of Conservation, Division of Mines and Geology, 1986 (Second Printing 1998), Bortugno, E.J. and Spittler, T.E., *Geologic Map of the San Bernardino Quadrangle*: 'Regional Geologic Map Series, San Bernardino Quadrangle-Map No. 3a (Geology),' Sheet 1 of 5 through 5 of 5, Scale: 1:250,000.

California Department of Conservation, Division of Mines and Geology, Effective January 1, 1977, *State of California Special Studies Zones, Redlands Quadrangle, Revised Official Map*: Scale: 1:24,000.

California Department of Conservation, Division of Mines and Geology, 1999, *Seismic Shaking Hazard Maps of California*: Map Sheet 48.

Frankel, A., Harmeson, S., Mueller, C., Barnhard, Y., Leyendecker, E.V., Perkins, D., Hanson, S., Dickman, N., and Hopper, M., 1997, *Uniform Hazard Spectra, Deaggregation, and Uncertainty*: Proceedings of FHWA/NCEER Workshop on the National Representation of Seismic Ground Motion for New and Existing Highway Facilities: NCEER Technical Report 9700010, pp. 39-73 [Text to accompany gridded values for the California probabilistic seismic hazard model, downloadable data at http://www.geohazards.cr.usgs.gov/eq/data/CNUmap1r.asc].

Ishihara, K., 1985, *Stability of Natural Deposits During Earthquakes*: Proceedings: 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1. pp. 321-376.

Ishihara, K., 1993, *Liquefaction and Flow Failures During Earthquakes*: Geotechnique, Vol.43, No. 3, pp. 351-415.

Liao, S.S.C. and Whitman, R.V., 1986, *Overburden Correction Factors for SPT in Sand*: Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 3, pp. 373-377.

Matti, J.C. and Morton, D.M., 1993, *Paleogeographic Evolution of the San Andreas Fault in Southern California: a Reconstruction Based on a New Cross-fault Correlation, in the San Andreas Fault System*: Displacement, Palinspastic

TECHNICAL REFERENCES

Reconstruction and Geologic Evolution, Edited by R.E. Powell, R.J. Weldon, and J.C. Matti, Mem. Geologic Society of America, Bulletin 178, pp. 107-160.

Navel Facilities Engineering Command, September 1986, *Foundations & Earth Structures*: Design Manual 7.02, Change 1.

Pradel, D., 1998, *Procedures to Evaluate Earthquake-Induced Settlement in Dry Sandy Soils*: ASCE Geotechnical Journal, April 1998, pp. 364-368.

Public Works Standards, Inc., 2012 Edition with 2014 Cumulative Supplement, *The "Greenbook", Standard Specifications for Public Works Construction.*

San Bernardino County Planning Department, *San Bernardino Land Use Plan, GENERAL PLAN, Hazard Overlays*, Sheet FH31 B Redlands, Plot Date: 03/09/2010, Scale: 1:14,400.

San Bernardino County Planning Department, *San Bernardino Land Use Plan, GENERAL PLAN, Geologic Hazard Overlays*, Sheet FH31 C Redlands, Plot Date: 03/09/2010, Scale: 1:14,400.

Seeber, L. and Armbruster, J.G., 1995, *The San Andreas Fault System Through the Transverse Ranges as Illuminated by Earthquakes Abstract*: J. Geophys. Res., 100, 8285.

Seed, H.B. and Idriss, I.M., 1971, *Simplified Procedure for Evaluating Soil Liquefaction Potential*: Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM9, pp. 1249-1274, September 1971.

Southern California Earthquake Center, March 1999, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California.*

Southern California Earthquake Center, June 2002, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California.*

TECHNICAL REFERENCES

Spotilla, J. and Sieh, K., 1997, *Characterizing Seismonic Sources Associated with Uplift of the San Bernardino Mountains: Progress Report to Southern California Earthquake Center*: 4 p., (http://www.scec.org/research/97progreports).

State of California, Department of Transportation, 2010 with Revisions Dated February 21, 2014, *Standard Specifications.*

Tokimatsu, K. and Seed, H.B., 1987, *Evaluation of Settlements in Sands Due to Earthquake Shaking*: Journal of Geotechnical Engineering Division, ASCE, Volume 113, No. 8, August, pp.861-878.

U.S. Department of the Interior, U.S. Geological Survey, 1991, Matti, J.C. and Carson, S.E., *Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California - A Regional Evaluation*: U.S. Geological Survey Bulletin 1898.

U.S. Department of the Interior, U.S. Geological Survey, 1987, *Recent Reverse Faulting in the Transverse Ranges, California*: Text and Plates, U.S. Geological Survey Professional Paper 1339.

U.S. Department of the Interior, U.S. Geological Survey, 1985, *Evaluating Earthquake Hazards in the Los Angeles Region-An Earth-Science Perspective*: U.S. Geological Survey Professional Paper 1360.

U.S. Department of the Interior, U.S. Geological Survey, 1985, Matti, J.C., Morton, D.M., and Cox, B.F., *Distribution and Geologic Relations of Fault Systems in the Vicinity of the Central Transverse Ranges, Southern California*: U.S. Geological Survey Open-File Report 85-365.

U.S. Department of the Interior, U.S. Geological Survey, 2003, Morton, D.M., and Miller, F.K. (Digitally Prepared by Cossette, P.M. & Bovard, K.R.), *Preliminary Geologic Map of the San Bernardino 30'x60' Quadrangle, California*: Digital Version 1.0, U.S. Geological Survey Open-File Report 03-293, Scale: 1:100,000, Sheet 1 of 5 through Sheet 5 of 5.

U.S. Department of the Interior, U.S. Geological Survey, 2003, Matti, J.C., Morton, D.M., and Cox, B.F., Carson, S.E., and Kendrick, K.J., *Geologic Map of the Redlands 7.5' Quadrangle, San Bernardino and Riverside Counties, California*: U.S. Geological Survey Open-File Report 03-302, Scale: 1:24,000.

TECHNICAL REFERENCES

U.S. Department of the Interior, U.S. Geological Survey, 1985, Carson, S.E. and Matti, J.C., *Contour Map Showing Minimum Depth to Groundwater, Upper Santa Ana River Valley, California, 1973-1979*: U.S. Geological Survey, Miscellaneous Field Studies Map, MAP MF-1802, Sheet 1 of 2 and 2 of 2, Scale: 1:48,000.

U.S. Department of the Interior, U.S. Geological Survey, Design Maps Web Site (https://earthquake.usgs.gov/designmaps/us/application.php).

U.S. Department of the Interior, U.S. Geological Survey, Geologic Hazards Science Center's 2008 NSHMP PSHA Interactive Deaggregation Web Site (https://geohazards.usgs.gov/deaggint/2008/).

U.S. Department of the Interior, U.S. Geological Survey, 1967, Photorevised 1988, *Redlands Quadrangle, California San Bernardino Co.:* 7.5-Minute Series (Topographic), Scale: 1:24,000.

U.S. Federal Emergency Management Administration (FEMA), Effective August 28, 2008, *Flood Insurance Rate Map*: Map Nos. 06071C8712 H, 06071C8716 H. Site specific information obtained through FEMA Website, Map Service Center, (http://msc.fema.gov/).

Weldon, R.J., and Sieh, K.E., 1985, *Holocene Rate of Slip and Tentative Recurrence Interval for Large Earthquakes on the San Andreas Fault in Cajon Pass, Southern California*: Geological Society of America Bulletin 96, pp. 793-812.

Wesnousky, Steven G., Prentice, Carol S., and Sieh, Kerry E., 1991, *An Offset Holocene Stream Channel and the Rate of Slip along the Northern Reach of the San Jacinto Fault Zone, San Bernardino Valley, California*: Geological Society of America Bulletin, V 103.

APPENDIX C

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PERCOLATION SUBSURFACE EXPLORATION LOG
PERCOLATION NO. P-1

Project Name: Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA

Project No.	1151-A17.1	Date:	1/4/2018	Logged By:	AH
Type of Rig:	Hollow-Stem Auger	Drive Wt.:	140 lb	Elevation:	± 1338
Drill Hole Dia.:	8 in.	Drop:	30 in.	Depth of Boring (ft.):	5.7

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
			SC			af		2.0 inches Hot Mix Asphalt (HMA) / 0 Base Clayey fine to medium sand, trace coarse sand; Red brown; Moist.
1								
2			SC/SM			Qvoa3		OLD AXIAL VALLEY DEPOSITS: Slightly silty to clayey, fine to medium sand; Red brown; Moist.
3								
4								
5								
6								Bottom of boring at 5.7 feet. No groundwater encountered. Boring converted to percolation test, and backfilled with excavated material after testing.
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

HILLTOP GEOTECHNICAL
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PERCOLATION SUBSURFACE EXPLORATION LOG
PERCOLATION NO. P-2

Project Name: Proposed Restaurant Pads and Building Renovation, 216 Brookside Avenue, Redlands, CA
Project No. 1151-A17.1 Date: 1/4/2018 Logged By: AH
Type of Rig: Hollow-Stem Auger Drive Wt.: 140 lb Elevation: ± 1339
Drill Hole Dia.: 8 in. Drop: 30 in. Depth of Boring (ft.): 5.7

Depth (ft.)	Sample Type	Penetration Resistance	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
			SC			af		1.75 inches Hot Mix Asphalt (HMA), No base Clayey fine to medium sand, trace coarse sand; Red brown; Moist.
1								
2			SC			Qvoa3		OLD AXIAL VALLEY DEPOSITS: Gravelly, fine to coarse sand; Red brown; Moist. Largest gravel sized 4".
3								
4								
5								
6								Bottom of boring at 5.7 feet. No groundwater encountered. Boring converted to percolation test, and backfilled with excavated material after testing.
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

S - SPT Sample R - Ring Sample B - Bulk Sample N - Nuclear Gauge Test D - Disturbed Sample

N.R. - No Recovery

Plate No. 11

HILLTOP GEOTECHNICAL
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SHALLOW PERCOLATION DATA SHEET

Project Name:	Proposed Restaurants and Building Renovations, Redlands	Project Number:	1151-A17.1
Test Hole Number:	P-1	Date Tested:	1/5/18
Depth of Boring in feet:	5.67	Tested By:	AH
Diameter of Boring in inches:	0.67	Hours Presaturation	16

Depth of Bottom (ft)	Time Initial	Time Final	Time Interval (minutes)	Depth of Water - Initial (ft)	Depth of Water - Final (ft)	Change in Water Level (ft)	$H_{average}$ (ft)	Rate, It (In/Hr)	Rate, It (Cm/Hr)
5.67	10:17	10:42	25.0	0.166	0.604	0.438	5.285	0.752	1.910
4.67	10:42	11:12	30.0	0.000	0.479	0.479	4.431	0.808	2.053
4.63	11:12	11:42	30.0	0.250	0.521	0.271	4.240	0.476	1.210
4.56	11:42	12:12	30.0	0.292	0.563	0.271	4.133	0.488	1.239
4.54	12:12	12:42	30.0	0.250	0.917	0.667	3.957	1.250	3.174
4.38	12:42	13:12	30.0	0.396	1.000	0.604	3.677	1.210	3.074
4.29	13:12	13:42	30.0	0.396	0.875	0.479	3.655	0.965	2.452
4.16	13:42	14:12	30.0	0.333	0.792	0.459	3.598	0.938	2.384
4.16	14:12	14:42	30.0	0.354	0.771	0.417	3.598	0.853	2.165
4.00	14:42	15:12	30.0	0.250	0.708	0.458	3.521	0.955	2.426
3.90	15:12	15:42	30.0	0.354	0.771	0.417	3.338	0.913	2.319
3.83	15:42	16:12	30.0	0.250	0.708	0.458	3.351	0.999	2.537

Average Steady State Rate (In/Hr):	**0.930**	Average Steady State Rate (Cm/Hr):	**2.362**	Average Rate (In/Hr):	**0.896**	Average Rate (Cm/Hr):	**2.276**

Plate No. 12

HILLTOP GEOTECHNICAL
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SHALLOW PERCOLATION DATA SHEET

Project Name:	Proposed Restaurants and Building Renovations, Redlands	Project Number:	1151-A17.1
Test Hole Number:	P-2	Date Tested:	1/5/18
Depth of Boring in feet:	5.67	Tested By:	AH
Diameter of Boring in inches:	0.67	Hours Presaturation	16

Depth of Bottom (ft)	Time Initial	Time Final	Time Interval (minutes)	Depth of Water - Initial (ft)	Depth of Water - Final (ft)	Change in Water Level (ft)	$H_{average}$ (ft)	Rate, It (In/Hr)	Rate, It (Cm/Hr)
5.58	10:21	10:46	25.0	0.250	4.458	4.208	3.226	11.401	28.958
5.58	10:46	11:11	25.0	0.146	3.040	2.894	3.987	6.460	16.409
4.38	11:11	11:21	10.0	0.229	1.875	1.646	3.323	10.853	27.567
4.38	11:22	11:32	10.0	0.313	1.646	1.333	3.396	8.619	21.891
4.33	11:32	11:42	10.0	0.208	1.333	1.125	3.560	6.968	17.697
4.33	11:42	11:52	10.0	0.188	1.188	1.000	3.642	6.065	15.405
4.29	11:52	12:02	10.0	0.167	1.125	0.958	3.644	5.807	14.750
4.29	12:02	12:12	10.0	0.188	1.021	0.833	3.686	4.997	12.693
4.27	12:12	12:22	10.0	0.208	1.000	0.792	3.666	4.775	12.127

Average Steady State Rate (In/Hr):	**5.411**	Average Steady State Rate (Cm/Hr):	**13.744**	Average Rate (In/Hr):	**6.869**	Average Rate (Cm/Hr):	**17.447**