



Appendix D

Geotechnical Evaluation



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June 14, 2023

Project No. 644-23036
23-06-064

Donlon Builders, Inc.
2681 Saturn Street
Brea, California 92821

Project: Proposed Industrial/Warehouse Building
322 Jurupa Avenue
Rialto, California

Subject: Geotechnical Update & Change of Consultant

Ref: Geotechnical Investigation report prepared by LGC Geotechnical, Inc. (LGC)
dated May 13, 2022; Project No. 22041-01

In accordance with your request, we have reviewed the above referenced Geotechnical Investigation report prepared for the project by LGC Geotechnical, Inc. (LGC) as it relates to the design and construction of the new industrial/warehouse building proposed for the project site located at 322 Jurupa Avenue in the City of Rialto, California. It is our understanding that the proposed industrial/warehouse building will be of concrete tilt-up construction and will be supported by conventional shallow spread footings and concrete slabs on grade.

Sladden Engineering has been retained to perform geotechnical testing services during the construction phase of this project. Prior to our involvement, we reviewed the Geotechnical Investigation report prepared by LGC along with the project plans. Based upon our report review and our understanding of the proposed construction, we concur with the recommendations included within the referenced LGC report. **Based upon our involvement during site grading and building construction, Sladden Engineering agrees to accept responsibility as the geotechnical consultant of record on this project.**

The referenced report includes recommendations pertaining to the design and construction of commercial/industrial building foundations. Based upon our review of the referenced report and our understanding of the proposed construction, it is our opinion that the structural values and remedial grading recommendations included in the LGC report remain applicable for use in the design and construction of the proposed warehouse building, except as amended herein.

The proposed industrial/warehouse building may be supported upon conventional shallow spread footings. Conventional shallow spread footings should be bottomed into properly compacted engineered fill soil a minimum of 18 inches below lowest adjacent grade. Continuous footings should be at least 12 inches wide and isolated pad footings should be at least 2 feet wide. Continuous footings and isolated pad footings may be designed utilizing allowable bearing pressures of 2000 psf and 2500 psf, respectively. Allowable increases of 250 psf for each additional 1 foot of width and 250 psf for each additional 6 inches of depth may be utilized, if desired. The maximum allowable bearing pressure should be 3000 psf. The recommended allowable bearing pressures are intended for dead and long-term live loads and may be increased by one-third when considering transient wind and seismic loading.

Static settlement resulting from the anticipated foundation loads should be tolerable provided that the recommendations included in this report are considered in foundation design and construction. The ultimate static settlement is expected to be less than 1.0 inch when using the recommended allowable foundation bearing pressures. As a practical matter, differential static settlement between footings can be assumed as one-half of the total static settlement.

Lateral forces may be resisted by friction along the base of the foundations and passive resistance along the sides of the footings. A friction coefficient of 0.35 times the normal dead load forces is recommended for use in design. Passive resistance may be estimated using an equivalent fluid weight of 250 pcf. If used in combination with the passive resistance, the frictional resistance should be reduced by one third.

The bearing soil is non-expansive and falls within the "very low" expansion category in accordance with 2022 California Building Code (CBC) classification criteria. Slab thickness and reinforcement should be determined by the structural engineer. We recommend a minimum floor slab thickness of 4.0 inches and minimum reinforcement of #3 bars at 24 inches on center in both directions for office areas. A minimum floor slab thickness of 6.0 inches and minimum reinforcement of #4 bars at 24" on center is recommended for warehouse areas. All slab reinforcement should be supported on concrete chairs to ensure that reinforcement is placed at slab mid-height.

Slabs with moisture sensitive surfaces should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 10-mil Visqueen, or equivalent. All laps within the membrane should be sealed and at least 2 inches of clean sand should be placed over the membrane to prevent damage during concrete placement and to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to placing a thin leveling course of sand across the pad surface prior to placement of the membrane.

Minor retaining walls may be necessary to accomplish the proposed construction. Lateral pressures for use in retaining wall design may be estimated using an equivalent fluid weight of 35 pcf for level free-draining native backfill conditions. For walls that are to be restrained at the top, the equivalent fluid weight should be increased to 55 pcf for level free-draining native backfill conditions. Back drains should be provided for the full height of the walls. Seismic pressures should be incorporated into any retaining walls greater than 6 feet in height.

The seismic design category for a structure may be determined in accordance with Section 1613 of the 2022 CBC or ASCE7-16. According to the 2022 CBC, Site Class D may be used to estimate design seismic loading for the proposed structures. The 2022 CBC Site Specific Seismic Hazard Analyses are attached. The project Structural Engineer should verify that all design parameters provided are applicable for the subject project.

Due to the presence of undocumented fill soil throughout the project site, significant remedial grading is recommended. To provide firm and uniform foundation bearing conditions, the artificial fill soil and the primary foundation bearing soil should be over-excavated and re-compacted. Over-excavation should extend to competent native soil or to a minimum depth of 5 feet below the bottom of the footings, whichever is deeper. Once adequate removals have been verified, the exposed native soil should be scarified, moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction. Removal up to 15 feet in depth may be required to accomplish the complete removal of undocumented fill soil. Once cleaned of unsuitable material, the previously removed material may be placed in thin lifts at near optimum moisture content and compacted to at least 90 percent relative compaction. Removals should extend at least 5 feet laterally beyond the building limits.

We appreciate the opportunity to provide service to you on this project. If you have any questions regarding this letter or the referenced report, please contact the undersigned.

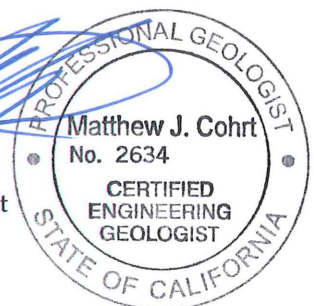
Respectfully submitted,
SLADDEN ENGINEERING

Brett L. Anderson
Principal Engineer

SER/ra



Matthew J. Cohrt
Principal Geologist



Copies: pdf/ Addressee

SITE-SPECIFIC GROUND MOTION ANALYSIS (ASCE 7-16)

Project: 322 Jurupa Avenue, Rialto, California
Project Number: 644-23036
Client: Donlon Builders, Inc.
Site Lat/Long: 34.0500/-117.3755
Controlling Seismic Source: San Jacinto

REFERENCE	NOTATION	VALUE	REFERENCE	NOTATION	VALUE	REFERENCE	NOTATION	VALUE
Site Class	C, D, D default, or E	D measured	Fv (Table 11.4-2)[Used for General Spectrum]	F _v	1.7			
Site Class D - Table 11.4-1	F _a	1.0	Design Maps	S _s	1.647	0.2*(S _{D1} /S _{DS})	T ₀	0.132*
Site Class D - 21.3(ii)	F _v	2.5	Design Maps	S ₁	0.641	S _{D1} /S _{DS}	T _s	0.662*
0.2*(S _{D1} /S _{DS})	T ₀	0.195	Equation 11.4-1 - F _a *S _s	S _{MS}	1.647*	Equation 11.4-4 - 2/3*S _{M1}	S _{D1}	0.7265*
S _{D1} /S _{DS}	T _s	0.973	Equation 11.4-3 - 2/3*S _{MS}	S _{DS}	1.098*	Equation 11.4-2 - F _v *S ₁	S _{M1}	1.0897*
Fundamental Period (12.8.2)	T	Period	Design Maps	PGA	0.698			
Seismic Design Maps or Fig 22-14	T _L	8	Table 11.8-1	F _{PGA}	1.1			
Equation 11.4-4 - 2/3*S _{M1}	S _{D1}	1.0683	Equation 11.8-1 - F _{PGA} *PGA	PGA _M	0.768*			
Equation 11.4-2 - F _v *S ₁ ¹	S _{M1}	1.6025	Section 21.5.3	80% of PGA _M	0.614			
¹ - F _v as determined by Section 21.3			Design Maps	C _{RS}	0.925			
			Design Maps	C _{R1}	0.9			
RISK COEFFICIENT								
Cr - At Periods <=0.2, Cr=C _{RS}	C _{RS}	0.925				Cr - At Periods between 0.2 and 1.0 use trendline formula to complete	Period	Cr
Cr - At Periods >=1.0, Cr=C _{R1}	C _{R1}	0.9					0.200	0.925
							0.300	0.922
							0.400	0.919
							0.500	0.916
							0.600	0.913
							0.680	0.910
							1.000	0.9

* Code based design value. See accompanying data for Site Specific Design values.

Mapped values from <https://hazards.atcouncil.org/>
<https://www.seismicmaps.org/>



Sladden Engineering

PROBABILISTIC SPECTRA¹
2% in 50 year Exceedence

Project No: 644-23036

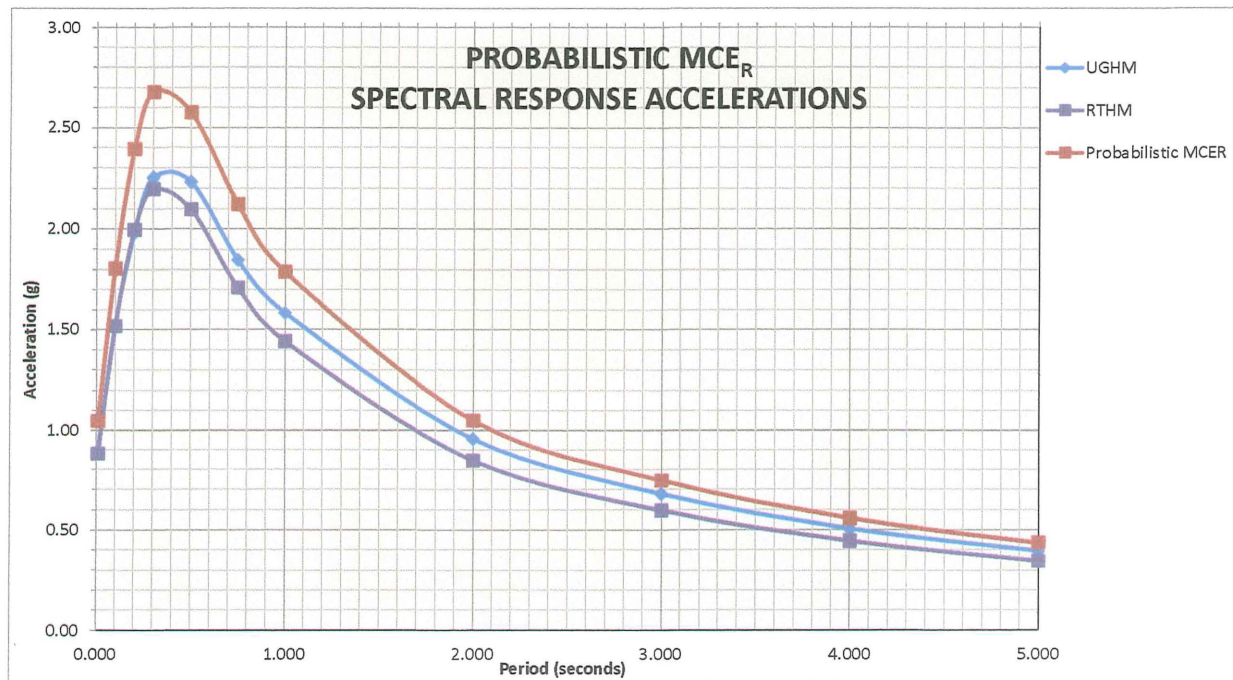
Period	UGHM	RTGM	Max Directional Scale Factor ²	Probabilistic MCE
0.010	0.895	0.879	1.19	1.046
0.100	1.523	1.516	1.19	1.804
0.200	1.979	1.993	1.20	2.392
0.300	2.251	2.194	1.22	2.677
0.500	2.229	2.093	1.23	2.574
0.750	1.847	1.710	1.24	2.120
1.000	1.583	1.441	1.24	1.787
2.000	0.957	0.846	1.24	1.049
3.000	0.676	0.594	1.25	0.743
4.000	0.505	0.444	1.25	0.555
5.000	0.395	0.345	1.26	0.435

¹ Data Sources:

<https://earthquake.usgs.gov/hazards/interactive/>
<https://earthquake.usgs.gov/designmaps/rtgm/>

² Shahi-Baker RotD100/RotD50 Factors (2014)

Probabilistic PGA: 0.895
Is Probabilistic $S_{a(max)} < 1.2F_a$? NO



DETERMINISTIC SPECTRUM

Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations¹

Controlling Source: San Jacinto

Is Probabilistic $S_{a(max)} < 1.2F_a$?

NO

Project No: 644-23036

Period	Deterministic PSa Median + 1.0 for 5% Damping	Max Directional Scale Factor ²	Deterministic MCE	Section 21.2.2 Scaling Factor Applied
0.010	0.857	1.19	1.019	1.019
0.020	0.859	1.19	1.022	1.022
0.030	0.870	1.19	1.036	1.036
0.050	0.915	1.19	1.089	1.089
0.075	1.082	1.19	1.288	1.288
0.100	1.276	1.19	1.519	1.519
0.150	1.552	1.20	1.863	1.863
0.200	1.742	1.20	2.090	2.090
0.250	1.897	1.21	2.296	2.296
0.300	1.988	1.22	2.425	2.425
0.400	2.043	1.23	2.513	2.513
0.500	1.997	1.23	2.457	2.457
0.750	1.657	1.24	2.055	2.055
1.000	1.410	1.24	1.749	1.749
1.500	1.013	1.24	1.256	1.256
2.000	0.769	1.24	0.954	0.954
3.000	0.522	1.25	0.652	0.652
4.000	0.356	1.25	0.445	0.445
5.000	0.259	1.26	0.326	0.326

Is Deterministic $S_{a(max)} < 1.5 * F_a$?

NO

Section 21.2.2 Scaling Factor:

N/A

Deterministic PGA:

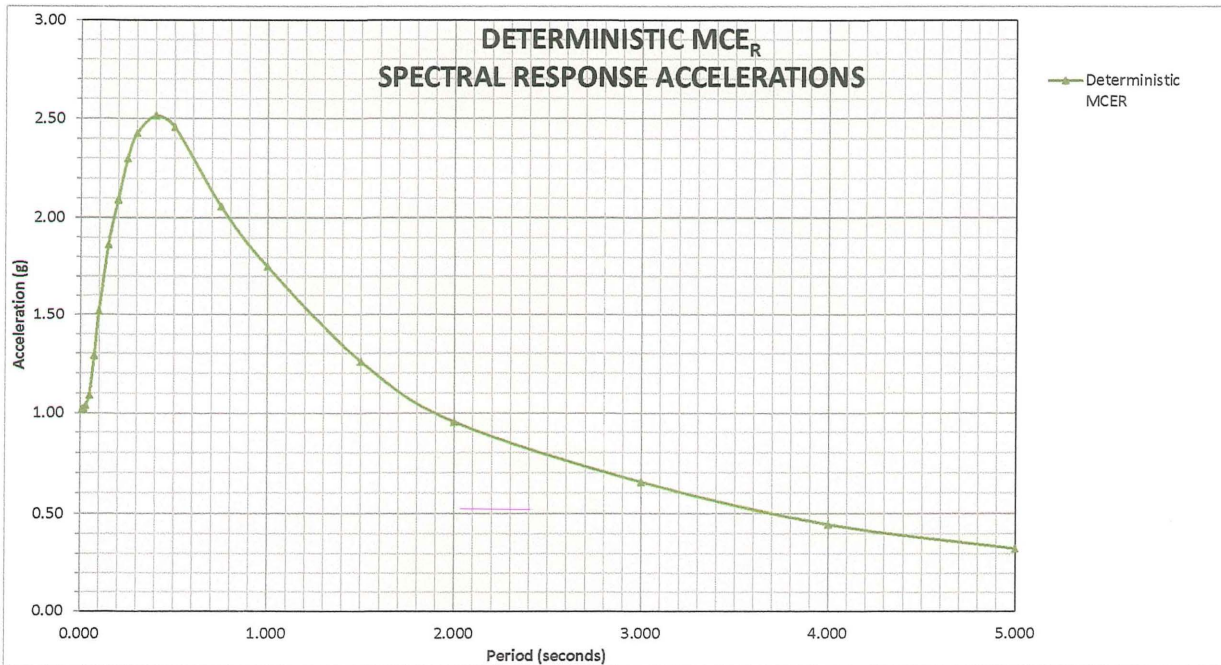
0.857

Is Deterministic PGA $\geq F_{PGA} * 0.5$?

YES

¹ NGAWest 2 GMPE worksheet and
Uniform California Earthquake Rupture
Forecast, Version 3 (UCERF3) - Time
Dependent Model

² Shahi-Baker RotD100/RotD50 Factors
(2014)



SITE SPECIFIC SPECTRA

Period	Probabilistic MCE	Deterministic MCE	Site-Specific MCE	Design Response Spectrum (Sa)
0.010	1.046	1.019	1.019	0.680
0.100	1.804	1.519	1.519	1.013
0.200	2.392	2.090	2.090	1.394
0.300	2.677	2.425	2.425	1.617
0.500	2.574	2.457	2.457	1.638
0.750	2.120	2.055	2.055	1.370
1.000	1.787	1.749	1.749	1.166
2.000	1.049	0.954	0.954	0.636
3.000	0.743	0.652	0.652	0.435
4.000	0.555	0.445	0.445	0.296
5.000	0.435	0.326	0.326	0.217

ASCE 7-16: Section 21.4

Site Specific

	Calculated Value	Design Value
SDS:	1.474	1.474
SD1:	1.304	1.304
SMS:	2.211	2.211
SM1:	1.957	1.957
Site Specific PGAm:	0.857	0.857
Site Class:	D measured	

Seismic Design Category - Short*

D

Seismic Design Category - 1s*

D

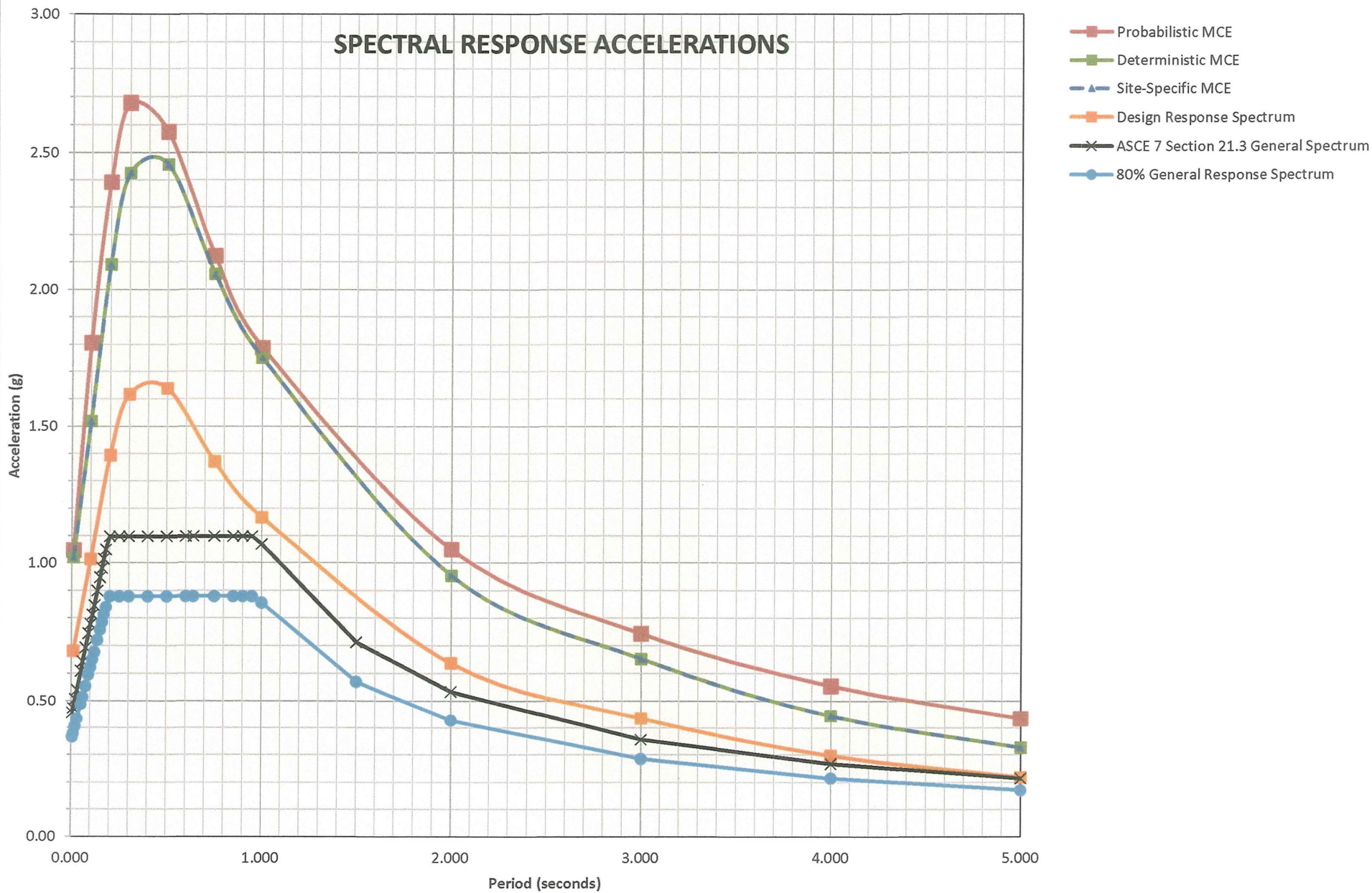
* Risk Categories I, II, or III

Period	ASCE 7 SECTION 21.3 General Spectrum	80% General Response Spectrum
0.005	0.456	0.365
0.010	0.473	0.378
0.020	0.507	0.406
0.030	0.541	0.433
0.050	0.608	0.487
0.060	0.642	0.514
0.075	0.693	0.554
0.090	0.744	0.595
0.100	0.778	0.622
0.110	0.812	0.649
0.120	0.845	0.676
0.136	0.900	0.720
0.150	0.947	0.758
0.160	0.981	0.785
0.170	1.015	0.812
0.180	1.049	0.839
0.200	1.098	0.878
0.250	1.098	0.878
0.300	1.098	0.878
0.400	1.098	0.878
0.500	1.098	0.878
0.600	1.098	0.878
0.640	1.098	0.878
0.750	1.098	0.878
0.850	1.098	0.878
0.900	1.098	0.878
0.950	1.098	0.878
1.000	1.068	0.855
1.500	0.712	0.570
2.000	0.534	0.427
3.000	0.356	0.285
4.000	0.267	0.214
5.000	0.214	0.171

Project No: 644-23036



SPECTRAL RESPONSE ACCELERATIONS



May 13, 2022

Project No. 22041-01

Mr. Benjamin Horning
Dedeaux Properties
1430 South Eastman Avenue
Commerce, CA 90023

Subject: Geotechnical Subsurface Due Diligence Evaluation, Proposed Industrial Development, Northwest of the Intersection of Jurupa Avenue and South Willow Avenue, Rialto, California

In accordance with your request, LGC Geotechnical, Inc. is providing this geotechnical subsurface due diligence report for the proposed industrial development, located northwest of the intersection of Jurupa Avenue and South Willow Avenue in the City of Rialto, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

LGC Geotechnical, Inc.



Dennis Boratyne, GE 2770
Vice President



Katie Maes, CEG 2216
Project Geologist



Branden Petersen, EIT

DJB/BPP/klr

Distribution: (1) Addressee (electronic copy)

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical due-diligence level report for the proposed industrial development located at the northwest corner of Jurupa Avenue and South Willow Avenue in the City of Rialto, California (see Site Location Map, Figure 1).

The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective. As part of this report, we have: 1) reviewed available geotechnical reports and geologic maps pertinent to the site (Appendix A); 2) performed a limited subsurface geotechnical evaluation of the site consisting of the excavation of eight small-diameter borings ranging in depth from approximately 25 to 50 feet below existing ground surface; 3) performed 6 exploratory test pits from approximately 6.5 to 12 feet below existing ground surface; 4) performed laboratory testing of select soil samples obtained during our subsurface evaluation; and 5) prepared this due-diligence level geotechnical evaluation report presenting our findings, conclusions and preliminary recommendations as it relates to the proposed development.

The findings and conclusions presented herein should be considered preliminary and will need to be re-evaluated once the project plans are available as part of a plan review report to be provided at a later date. It should be noted that LGC Geotechnical does not provide environmental consulting services.

1.2 Project Description and Background

The approximately 14.5-acre site is bound to the north by industrial buildings and vacant land, to the east by South Willow Avenue, to the south by Jurupa Avenue and west by various structures. The site is primarily comprised of one existing building in the north with associated parking and vacant land with a stockpile on the south.

Review of historic aerial photographs indicate that the current configuration of the buildings on the north was constructed by 2005. Previously, the site was undeveloped farmland until 1959 when a structure was built at the southeast corner of the site. The grading and stockpiling operation at the southern portion of the site started in 2016 (Historic Aerials, 2022).

Based on the provided information and conceptual site plans (GAA, 2022), the proposed industrial development will consist of one 94,000 square foot warehouse and associated parking. The proposed industrial building is anticipated to be an at-grade concrete tilt-up structure with estimated maximum column and wall loads of approximately 150 kips and 10 kips per linear foot, respectively. Please note no structural loads or preliminary grading plans were provided to us at the time of this report.

The preliminary recommendations given in this report are based upon the proposed layout and estimated structural loading information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated

project plans and the actual structural loads when they become available, in order to either confirm or modify the recommendations provided herein. This may include but is not limited to additional subsurface explorations, laboratory testing and analysis to provide a design level geotechnical report.



FIGURE 1
Site Location Map

PROJECT NAME	Dedeaux - Willow Avenue, Rialto
PROJECT NO.	22041-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	May 2022

1.3 Subsurface Exploration

A geotechnical field evaluation was performed by LGC Geotechnical. This program consisted of drilling and sampling eight small-diameter borings. The borings were drilled by CalPac Drilling under subcontract to LGC Geotechnical. The depth of the borings ranged from approximately 25 to 50 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were performed using a truck-mounted drill rig equipped with 6-inch diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged for laboratory testing at select depths. At the completion of drilling, the borings were backfilled with cuttings and where applicable the surface was capped with quick-crete.

Boring logs are presented in Appendix B and their approximate locations are depicted on Figure 2.

1.4 Laboratory Testing

Representative driven and bulk samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ unit weight and moisture content, collapse, laboratory compaction, expansion index, and corrosion (sulfate, chloride, pH, and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 98 pounds per cubic foot (pcf) to 128 pcf, with an average of 113 pcf. Field moisture contents ranged from approximately 2 percent to 18 percent, with an average of 5 percent.
- Three collapse tests were performed. The stress vs. deformation plots are provided in Appendix C.
- Two Expansion Index (EI) tests were performed. Results were EI values of 0 and 3, corresponding to "Very Low" expansion potential.
- Two laboratory compaction tests of near surface samples indicate a maximum dry density ranging from 130 to 135 pcf and an optimum moisture content ranging from 6.0 to 8.5 percent.
- Corrosion testing indicated soluble sulfate contents of approximately 0.01 percent, chloride content of 40 parts per million (ppm), pH value of 8.01, and minimum resistivity value of 3,400 ohm-cm.

A summary of the laboratory test results is presented in Appendix C.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located in the northeastern portion of the broad San Bernardino Basin that is bound to the north by the San Gabriel Mountains and to the south by the Santa Ana Mountains. Regional topography is dominated by the presence of the northwest trending faults that define the mountains and hills of the Southern California region. Structurally, the site is located on the northeastern portion of the 'Perris block' of the Peninsular Ranges Geomorphic Province of Southern California. The 'Perris block' is an area of relatively low relief, consisting of a block of basement rock bounded by the Elsinore fault zone to the southwest and the San Jacinto fault zone to the northeast (USGS, 2003). The Santa Ana River flows southwest from the San Gabriel Mountains, less than a mile from the site.

2.2 Site-Specific Geology

Based on our subsurface exploration and review of pertinent geologic literature and maps, the site is generally underlain by undocumented artificial fill soils and Quaternary-aged old alluvial fan deposits (USGS, 2003). The alluvial fan deposit emanated from the San Gabriel Mountains, locally dissected by tributary drainage to the nearby Santa Ana River.

It should be noted that geotechnical explorations are only representative of the location where they are performed, and varying subsurface conditions may exist outside of each location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform, and that soil is homogeneous within the project area. A brief description of the materials encountered during drilling is presented in the following section, and the approximate boring locations are depicted on the Boring Location Map (Figure 2). For details on the stratigraphy at the exploration locations, refer to the boring logs provided in Appendix B.

2.2.1 Undocumented Artificial Fill (Map Symbol – afu)

Undocumented artificial fill was observed in the field explorations up to approximately 15 feet below existing grade in our borings. The stockpiled fill was observed to consist of dry to moist sand with varying amounts of silt and gravel.

2.2.2 Quaternary Old Alluvial Fan Deposits (Map Symbol – Qof)

Quaternary Old Alluvial Fan deposits, generally dated Late to Middle Pleistocene, were encountered at the site below the undocumented artificial fill. Where observed, the fan deposits generally consisted of medium dense to very dense sands with varying amounts of silt and gravel, to the maximum explored depth of approximately 50 feet below existing grade.

2.3 Groundwater

Groundwater was not encountered in our borings to an approximate depth of 50 feet below existing grade. Wells present at nearby sites show groundwater being present at depth greater than 100 feet below existing grade (WDL, 2022). High groundwater is estimated at 100 feet below existing grade.

It should be noted that higher localized and seasonal perched groundwater conditions may accumulate below the surface and should be expected throughout the design life of the proposed improvements. In general, groundwater conditions below any given site may vary over time depending on numerous factors including seasonal rainfall and local irrigation, among others.

2.4 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 34.051314 degrees north and longitude -117.374863 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 1. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 1
Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
S _s (Risk-Targeted Spectral Acceleration for Short Periods)	1.661g	From SEAOC, 2022
S ₁ (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.647g	From SEAOC, 2022
F _a (per Table 1613.2.3(1))	1.000	For Simplified Design Procedure of Section 12.14 of ASCE 7, F _a shall be taken as 1.4 (Section 12.14.8.1)
F _v (per Table 1613.2.3(2))	1.7	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S _{MS} for Site Class D [Note: S _{MS} = F _a S _s]	1.661g	-
S _{M1} for Site Class D [Note: S _{M1} = F _v S ₁]	1.010g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S _{DS} for Site Class D [Note: S _{DS} = (2/3)S _{MS}]	1.107g	-
S _{D1} for Site Class D [Note: S _{D1} = (2/3)S _{M1}]	0.733g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C _{RS} (Mapped Risk Coefficient at 0.2 sec)	0.925	ASCE 7 Chapter 22
C _{R1} (Mapped Risk Coefficient at 1 sec)	0.899	ASCE 7 Chapter 22
*Since site soils are Site Class D and S ₁ is greater than or equal to 0.2, the seismic response coefficient C _s is determined by Eq. 12.8-2 for values of T ≤ 1.5T _s and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for T _L ≥ T > T _s , or Eq. 12.8-4 for T > T _L . Refer to ASCE 7-16.		

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 7.19 at a distance of approximately 10.38 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.93 at a distance of approximately 12.34 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.774g (SEAOC, 2022). The design PGA is equal to 0.516g (2/3 of PGA_M).

2.5 Faulting

The subject site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CDMG, 1977; CGS, 2018). A fault is considered “active” if evidence of surface rupture in Holocene time (the last approximately 11,000 years) is present. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site. The closest known active faults that may impact the site include the San Jacinto, San Andreas, Cucamonga, and Elsinore Faults, among others.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching and shallow ground rupture, soil liquefaction, and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A discussion of these secondary effects is provided in the following sections.

2.5.1 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that loose, saturated, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the San Bernardino County Land Use Plan for liquefaction susceptibility (San Bernardino, 2007), the site is not located within a zone that is considered susceptible to liquefaction. Due to the lack of groundwater in the upper 50 feet the potential for liquefaction is considered very low.

2.5.2 Lateral Spreading

Lateral spreading is a type of liquefaction induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a

subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the very low potential for liquefaction, the potential for lateral spreading is also considered very low.

2.6 Expansion Potential

Based on the results of our recent laboratory testing, site soils are anticipated to have “Very Low” expansion potential. Final expansion potential of site soils should be determined at the completion of earthwork operations. Results of expansion testing at finish grades will be utilized to confirm final foundation recommendations.

3.0 CONCLUSIONS

Based on the results of our subsurface geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations contained in the following sections are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- From a geotechnical perspective, onsite soils are anticipated to be suitable for use as general compacted fill, provided they are screened of construction debris, significant organics and any oversized material (8 inches in greatest dimension).
- Groundwater was not encountered in our field evaluation. Records indicate groundwater levels recorded in the area are at depths greater than 100 feet below the ground surface.
- The subject study area is not located within a mapped State of California Earthquake Fault Zone (i.e., Alquist-Priolo Act Earthquake Fault Zone), and based upon our review of published geologic mapping, no known active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered very low.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Site soils are generally not susceptible to liquefaction due to a lack of groundwater and dense to very dense alluvial soils in the upper 50 feet. The site is also not located in an area with soils that are susceptible to liquefaction according to the San Bernardino County Use Plan (San Bernardino, 2007).
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “Very Low” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Based on the corrosion test results, soils are not considered corrosive per the Caltrans criteria (Caltrans, 2021). This should be confirmed after grading has been completed.
- Oversized material (material larger than 8 inches in maximum dimension) should be anticipated during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E.
- Undocumented fill (existing stockpiles) and 5 feet of the underlying native soils shall be temporarily removed and placed as engineered fill. Temporary removal and recompaction will range from approximately 10 to 20 feet across the entire site. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order.
- From a geotechnical perspective, the proposed industrial building can be supported on a slab on grade and shallow foundation system provided recommended earthwork removal and recompaction is performed. Additional field work, laboratory testing and analysis may have to be performed once actual building loads are known to further evaluate and confirm this.

- On-site soils will most likely be suitable for backfill of site retaining walls. Soils that will be used for retaining wall backfill should be tested and approved by the geotechnical consultant prior to the backfill of site walls.
- Based on our preliminary study, we recommend performing additional field work consisting of infiltration testing be performed.
- Final design level recommendations utilizing the site plans and structure loads should be provided as part of a comprehensive geotechnical report.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2019 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, grading and construction of the proposed new improvements, including the industrial structures, subsurface utilities, and vehicular pavement areas.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2019 CBC/City of Rialto requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations may be revised within future grading plan review reports or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

Prior to grading, areas to be developed should undergo the stripping and clearing of vegetation and clearing of surface obstructions from the site. Vegetation and debris

should be removed and properly disposed of offsite. Holes resulting from removals of buried obstructions, which extend below proposed remedial and/or finish grades, should be replaced with suitable compacted fill material.

If cesspools or septic systems are encountered they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements.

4.1.2 Removal and Recomposition Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the site soils be temporarily removed and recompacted according to the criteria outlined below. Updated recommendations may be required based on additional field work, changes to building layouts and actual structural loads.

Existing Northern Re-Development (Hollywood Delivery Service Building)

It is our understanding that the office portion of the existing industrial building in the northern portion of the site will be demolished, and new parking stalls will be created. Within pavement and hardscape areas, removal and recompaction should extend to a depth of at least 2 feet below the existing grade or 1 foot below finished subgrade (i.e., below planned aggregate base/asphalt concrete), whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 2 feet beyond the edges of the proposed pavement and hardscape improvements.

New Southern Development

Undocumented fill (existing stockpile) and 5 feet of the underlying native soils shall be temporarily removed prior to placement of engineered fill. Therefore, temporary removal and recompaction will range from approximately 10 to 20 feet across the entire southern site. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger. See Figure 2.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above-noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas and areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and applicable Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "C" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 projection from the bottom of the excavation or 5 feet, whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters can be provided.

4.1.4 Subgrade Preparation

In general, areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project requirements. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and any oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of Very Low expansion potential (expansion index of 20 or less based on ASTM D4829). Import for retaining wall backfill should meet the criteria outlined in the paragraph below. Source samples should be

provided to the geotechnical consultant for laboratory testing a minimum of three working days prior to any planned importation.

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. The majority of the on-site soils should be suitable for retaining wall backfill due to their low fines content (i.e., silt and clay content) and very low expansion potential. Samples of retaining wall backfill should be sampled prior to construction to confirm the findings of the investigation.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base), the City of Rialto or Caltrans Class 2 aggregate base.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than approximately 2 to 4 inches in maximum dimension and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned asphalt areas (i.e., not within building pad areas).

4.1.6 Placement and Compaction of Fills

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Soils are present that will require additional moisture in order to achieve the required compaction. Drying and/or mixing the very moist soils may also be required prior to reusing the materials in compacted fills. Generally, soils are present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by LGC Geotechnical. Oversized material as previously defined should be removed from site fills, if encountered.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction, or in accordance with the City of Rialto requirements, per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content), unless otherwise noted in the pavement recommendations section (see Sections 4.5 & 4.6).

If gap-graded $\frac{3}{4}$ -inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with vibration with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric (Mirafi 140N or approved alternative) to prevent the migration of fines into the rock backfill.

4.1.7 Trench and Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other material greater than 6 inches in diameter and organic matter. If trenches are shallow or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 or greater (per California Test Method [CTM] 217) may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least the recommended minimum relative compaction (per ASTM D1557).

Retaining wall backfill should consist of sandy soils as outlined in preceding Section 4.1.5. The limits of select sandy backfill should extend at minimum $\frac{1}{2}$ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 3 (rear of text). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.8 Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of shrinkage factors for the various soil types found onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

TABLE 2

Estimated Shrinkage

Soil Type	Allowance	Estimated Range
Alluvium	Shrinkage	0 to 10 %
Undocumented Fill (Stockpile)	Shrinkage	0 to 15%

Subsidence due to earthwork equipment is expected to be on the order of 0.1 feet. It should be stressed that these values are only estimates and that actual shrinkage factors are extremely difficult to predict. These values are estimates only and exclude losses due to removal of vegetation or debris. The effective change in volume of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor, and accuracy of the topographic survey. The above shrinkage estimates are intended as an aid for others in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. A balance area should be considered if export/import is either economically prohibitive or restricted by the governing agency.

4.2 Preliminary Foundation Recommendations

The proposed structures may be supported on spread or continuous footings and conventional slabs, provided earthwork is performed in accordance with the recommendations presented in this report. Since the site soils are anticipated to be "Very Low" expansion potential (EI of 20 or less per ASTM D4829), special design considerations from a geotechnical perspective are not anticipated, however, this must be verified based on as-graded conditions. Footings should be supported on properly compacted fill. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading.

Preliminary foundation recommendations are provided in the following sections. The foundation design must be performed by the structural engineer based on the following geotechnical parameters and minimum values provided.

4.2.1 Slab Design and Construction

From a geotechnical perspective, minimum slab thicknesses of 6 inches and 4 inches are recommended for new slabs in the warehouse areas and office areas, respectively. Slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Actual slab reinforcement and thickness should be determined by the structural engineer based on the imposed loading. Additional slab-on-grade recommendations can be provided for alternative building types upon request.

The foundation designer may use a modulus of vertical subgrade reaction (k) of 200 pounds per cubic inch (pounds per square inch per inch of deflection). This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed footing using the following formula:

$$k = 200 \times [(B+1)/2B]^2$$

k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)

B = foundation width (feet)

It is recommended that subgrade soils below slabs be moisture conditioned in order to maintain the recommended moisture content up to the time of concrete placement. The recommended moisture content of the slab subgrade soils should be between optimum moisture content and approximately 2 percent above optimum moisture content to a minimum depth of 12 inches. The moisture content of the slab subgrade should be verified by the geotechnical consultant within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

The following recommendations are for informational purposes only, as they are unrelated to the geotechnical performance of the foundation. The following recommendations may be superseded by the foundation engineer and/or owner. Some post-construction moisture migration should be expected below the foundation. In general, interior floor slabs with moisture sensitive floor coverings should be underlain by a minimum 10 mil thick polyolefin material vapor retarder, which has a water vapor transmission rate (permeance) of less than 0.03 perms. The need for sand and/or the sand thickness (above and/or below the vapor retarder) should be specified by the structural engineer, architect or concrete contactor. The selection and thickness of sand is not a geotechnical engineering issue and is therefore outside our purview.

4.2.2 Foundation Design Parameters

For the proposed industrial warehouse structures, minimum continuous wall and column footing widths are to be 12 inches and 24 inches, respectively, minimum foundation embedment is to extend a minimum of 18 inches below the adjacent exterior grade, and interior column footings should be embedded a minimum of 12 inches beneath the

adjacent subgrade. The following allowable bearing pressures for both continuous and column spread footings presented in Table 3 below are recommended for corresponding footing widths and embedments.

TABLE 3

Allowable Soil Bearing Pressures

Allowable Static Bearing Pressure (psf)	Minimum Footing Width (feet)	Minimum Footing Embedment* (feet)
3,000	3	2
2,500	2	1.5
2,000	1	1

* Refers to minimum depth measured below lowest adjacent grade.

These allowable bearing values indicated above (exclusive of the weight of the footings) are for total dead loads and frequently applied live loads and may be increased by $\frac{1}{3}$ for short duration loading (i.e., wind or seismic loads). The allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be on the order of 1-inch or less. Differential static settlement may be taken as half of the static settlement (i.e., $\frac{1}{2}$ -inch over a horizontal span of 40 feet).

4.2.3 Foundation Construction

The foundation is to be excavated into competent compacted artificial fill placed during grading operations. It is recommended that the foundation subgrade soils be evaluated by the geotechnical engineer prior to steel and/or concrete placement.

4.2.4 Lateral Load Resistance

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 340 pcf (maximum of 3,400 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive

resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.3 Lateral Earth Pressures for Retaining Walls

The following preliminary lateral earth pressures may be used for site retaining walls. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design.

The following lateral earth pressures are presented on Table 4 for approved select granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and Very Low expansion potential (EI of 20 or less per ASTM D4829). Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria. Most of the onsite soils should be suitable for retaining wall backfill due to the low fines content and very low expansion potential.

TABLE 4
Lateral Earth Pressures – On-site or Imported Sandy Backfill

Conditions	Equivalent Fluid Unit Weight (pcf)	Equivalent Fluid Unit Weight (pcf)
	Level Backfill	2:1 Sloped Backfill
	Approved Soils	Approved Soils
Active	35	55
At-Rest	55	70

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 3). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the

equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.35 and 0.5 may be used for the active and at-rest conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The retaining wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 12.5 pcf for level backfill conditions. This increment should be applied in addition to the provided static lateral earth pressure using a “normal” triangular distribution with the resultant acting at $H/3$ in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.2. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.4 Corrosivity to Concrete and Metal

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing of near-surface bulk samples indicated soluble sulfate contents of approximately 103 ppm, chloride content of approximately 40 parts per million (ppm), pH value of approximately 8.0, and minimum resistivity value of 3,400 ohm-cm. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils are designated to a class “S0” per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the “S0” sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.5 Preliminary Asphalt Concrete Pavement Sections

For the purposes of these preliminary recommendations, we have assumed a preliminary design R-value of 40 and calculated pavement sections for Traffic Indices of 5.0 (or less), 6.0 and 7.0. R-value testing of the street and parking subgrade will need to be performed to confirm our preliminary testing results/assumptions once the asphalt areas have been graded to finish subgrade elevations and the final Traffic Index is determined by the Civil Engineer. Determination of the Traffic Index is not the purview of the geotechnical consultant. Final street sections should be confirmed by the project civil engineer based upon the projected design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 5

Preliminary Asphalt Concrete Pavement Sections

Assumed Traffic Index	5.0 (or less)	6.0	7.0
R -Value Subgrade	40	40	40
AC Thickness	4.0 inches	4.0 inches	4.0 inches
CAB Thickness	6.0 inches	6.0 inches	8.0 inches

Increasing the thickness of asphalt or adding additional base material will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Aggregate base material (crushed aggregate base and crushed miscellaneous base) should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction, or the City of Rialto specifications, at or slightly above optimum moisture content per ASTM D1557. Earthwork recommendations are provided in Section 4.1 “Site Earthwork” and the related sub-sections of this report.

4.6 Preliminary Portland Cement Concrete Pavement Sections

For the purposes of these preliminary recommendations, we have assumed a preliminary design R-value of 40. Preliminary minimum Portland Cement Concrete (PCC) pavement street sections are provided in Table 6 for Traffic Indices of 5.0 (or less), 6.0, and 7.0 and may be utilized in the design of the truck parking/circulation areas or loading docks. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final PCC sections should be confirmed by the project civil engineer based upon the projected design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. The appropriate paving section must be selected by the project civil engineer/client based on design traffic indexes.

TABLE 6

Preliminary PCC Pavement Sections

Provided Traffic Index	5.0 (or less)	6.0	7.0
R -Value Subgrade	40	40	40
PCC Thickness	5.5 inches	6.5 inches	7.5 inches
95% Compacted Subgrade	12.0 inches	12.0 inches	12.0 inches

For preliminary planning purposes, the PCC pavement sections may consist of a minimum of concrete over subgrade soils compacted to 95 percent relative compaction (see Table 7 for section thicknesses). The concrete should have a minimum compressive strength of 3,500 psi and a minimum flexural strength of 530 psi at the time the pavement is subjected to traffic. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals not exceeding 15 feet in each direction, depth of $\frac{1}{3}$ the concrete thickness. Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint sealer and filler material should be specified by the pavement designer and should be maintained throughout the life of the pavement. The above section does not include steel reinforcement. If desired, steel reinforcement should be determined by the structural engineer.

The thicknesses shown are minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Subgrade below the PCC pavement should be compacted to a minimum of 95 percent relative compaction per ASTM D1557 near optimum moisture content (generally within optimum and 2 percent above optimum moisture content). Earthwork recommendations are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete (such as flatwork, sidewalks, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined below. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

Nonstructural and non-vehicular concrete flatwork placed on compacted subgrade may be a 4-inches (nominal) in thickness with crack control joints spaced 8 feet apart for flatwork slabs and 6 feet apart for flatwork sidewalks. Crack control joints should be sawcut or deep open tool joint to a minimum of 1/3 the concrete thickness. The compacted subgrade below the nonstructural and non-vehicular concrete flatwork should be wet down prior to placing concrete.

To reduce the potential for nonstructural concrete flatwork to separate from entryways and doorways, the owner may elect to install dowels to tie these two elements together.

4.8 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that finished grade adjacent to proposed structures be sloped away from the proposed structures and towards an approved drainage device or unobstructed swale, so that it will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.9 Geotechnical Plan Review

Project plans (grading, foundation, retaining wall, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional or modified geotechnical recommendations may be required based on the proposed layout.

4.10 Geotechnical Observation and Testing

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2019 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- During precise grading;
- Preparation of building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

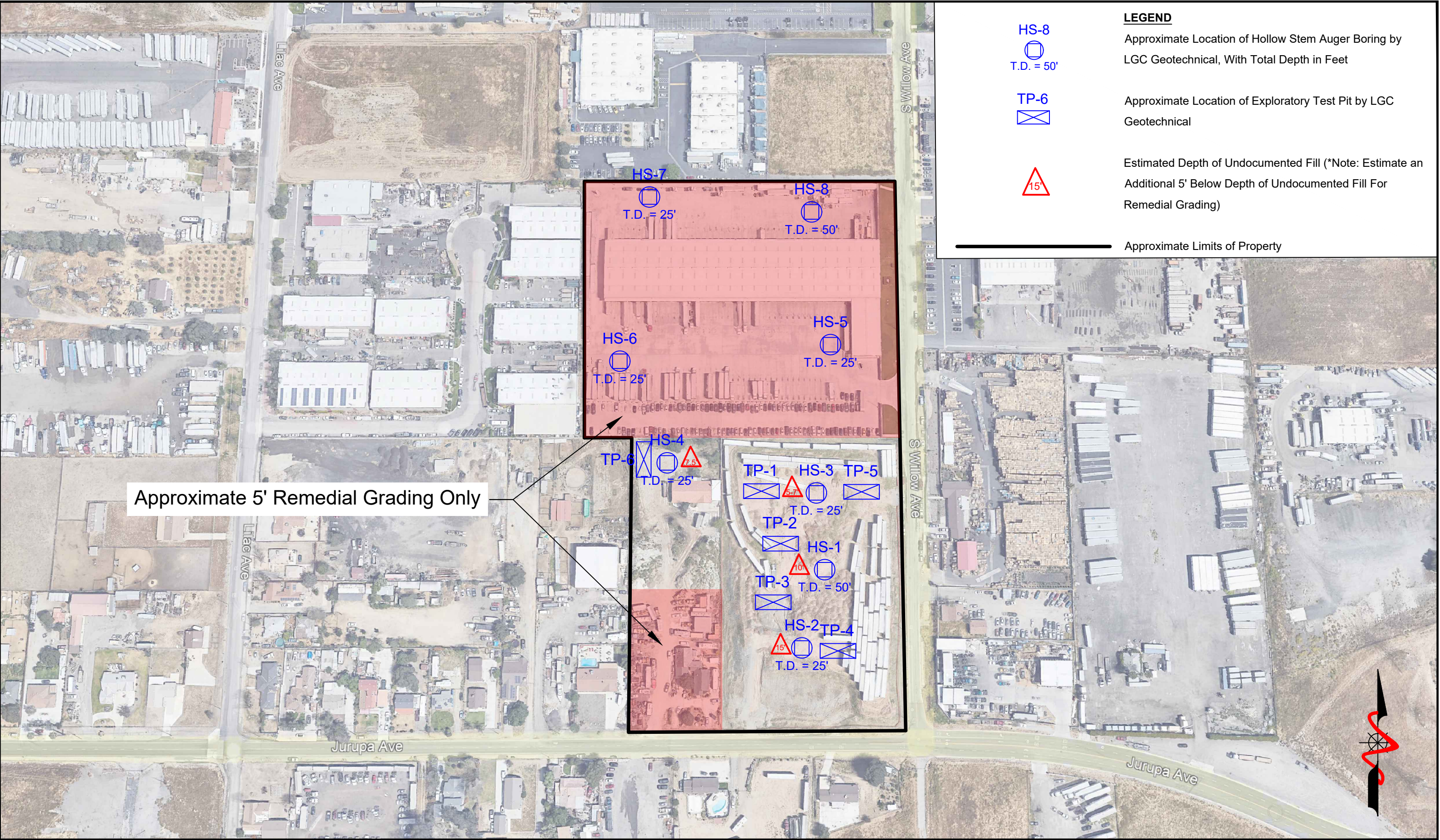
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants and incorporated into the plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.



LEGEND

- Approximate Location of Hollow Stem Auger Boring by LGC Geotechnical, With Total Depth in Feet
- Approximate Location of Exploratory Test Pit by LGC Geotechnical
- Estimated Depth of Undocumented Fill (*Note: Estimate an Additional 5' Below Depth of Undocumented Fill For Remedial Grading)
- Approximate Limits of Property

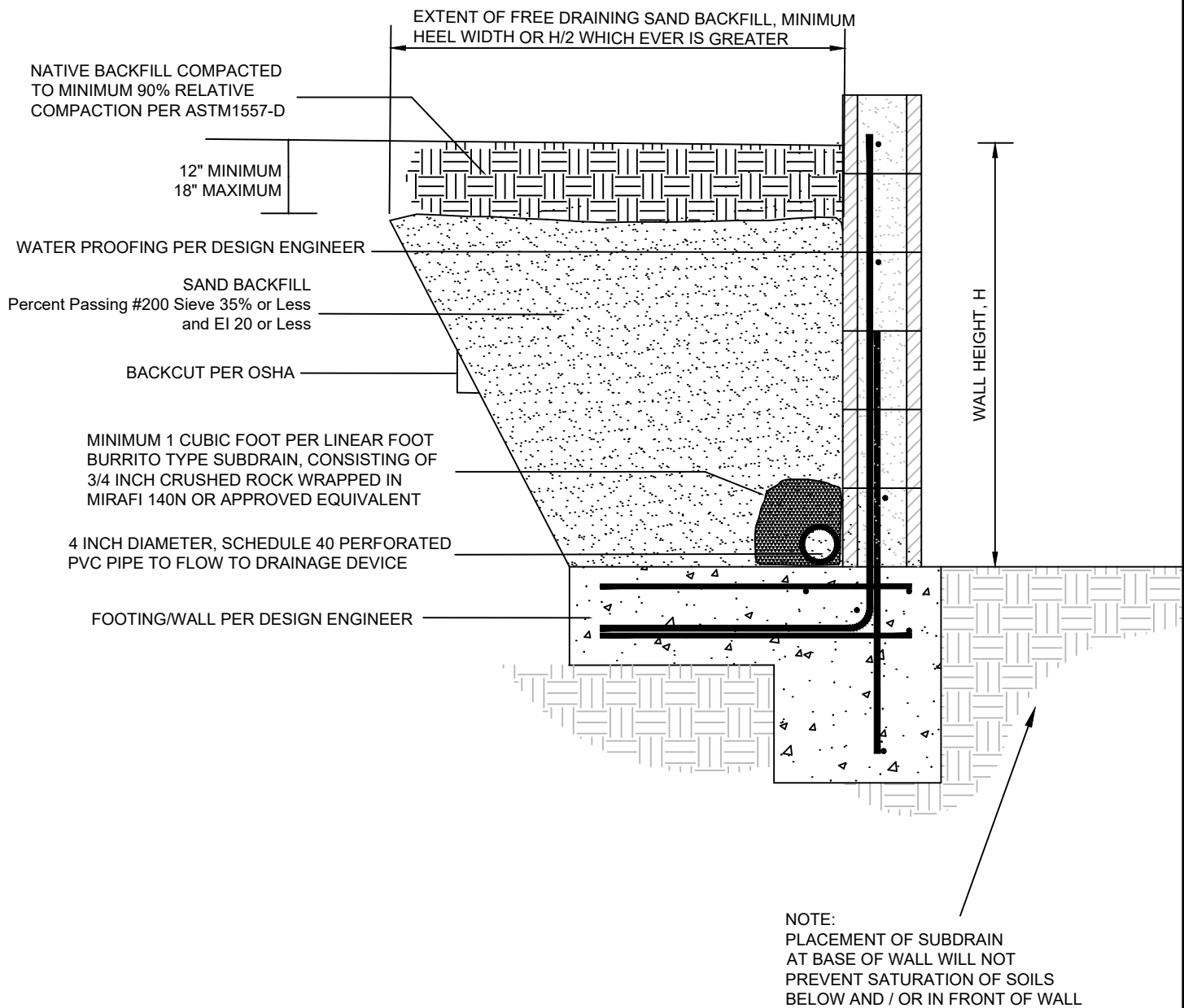


FIGURE 3
Retaining Wall
Backfill Detail

PROJECT NAME	Dedeaux - Willow Avenue, Rialto
PROJECT NO.	22041-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	May 2022

Appendix A

References

APPENDIX A

References

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Appendix B
Boring and Trench Logs

Geotechnical Boring Log Borehole HS-1

Date: 3/24/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~971' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
970	0							@0' to 10' - Undocumented Artificial Fill (afu): @0' - SAND with Silt: light brown, dry; rootlets	
			R-1	4 13 14	108.7	6.1	SM	@2.5'- Silty SAND: brown, slightly moist, medium dense	
965	5	B-1	R-2	5 6 10	108.4	4.0		@5'- Silty SAND: brown, dry, medium dense	
			R-3	8 7 8	102.7	2.7		@7.5'- Silty SAND: brown, dry, medium dense	
960	10		R-4	7 10 11	105.1	1.5	SM	@10' to T.D. - Old Alluvial-Fan Deposits (Qof): @10'- Silty SAND with Gravel: brown, dry, medium dense	
955	15		SPT-1	11 18 19		1.7	SP-SM	@15'- SAND with Silt and Gravel: grayish brown, dry, dense	
950	20		R-5	10 20 20	101.2	15.0		@20'- Silty SAND: brown, very moist, dense	
945	25		SPT-2	6 8 9		5.0		@25'- Silty SAND: grayish brown, slightly moist, medium dense	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-1

Date: 3/24/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~971' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
940	30		R-6	7 16 23	103.8	6.2	SM	@30'- Silty SAND: grayish brown, slightly moist, dense	
935	35		SPT-3	10 7 10		9.3		@35'- Silty SAND with Gravel: grayish brown, moist, medium dense	
930	40		R-7	20 27 26		6.3		@40'- Disturbed Sample: Silty SAND: grayish brown, slightly moist, dense	
925	45		SPT-4	16 15 16		18.0	ML	@45'- Silty CLAY: olive brown, very moist, hard	
920	50		R-8	12 21 24	98.4	8.9	SM	@50'- Silty SAND: grayish brown, moist, dense	
915	55							Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	
910	60								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-2

Date: 3/24/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~972' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
970	0							@0' to 15' - Undocumented Artificial Fill (afu): @0'- SAND: light brown, dry; rootlets	
			R-1	4 6 8	109.5	7.2	SM	@2.5'- Silty SAND: brown, slightly moist, medium dense	
	5		R-2	6 8 8	109.2	6.2		@5'- Silty SAND: brown, slightly moist, medium dense	
965			R-3	8 8 4	99.6	5.8		@7.5'- Silty SAND: brown, slightly moist, loose	
	10		R-4	4 8 13	107.0	7.0		@10'- Silty SAND: brown, slightly moist, medium dense	
960									
	15		R-5	8 12 16	109.4	1.6	SP-SM	@15' to T.D. - Old Alluvial-Fan Deposits (Qof): @15'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
955									
	20		SPT-1	10 17 19		1.5		@20'- SAND with Silt and Gravel: grayish brown, dry, dense	
950									
	25		R-6	8 15 18				@25'- No Recovery	
945									
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-3

Date: 3/24/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~974' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
970	0	B-1	R-1	8 8	111.0	8.0	SM	@0' to 5' - Undocumented Artificial Fill (afu): @0'- Silty SAND: light brown, dry; rootlets	CO
								@2.5'- Silty SAND: brown, moist, medium dense	
	5		R-2	6 5 6	104.9	5.2	SM	@5' to T.D. - Old Alluvial-Fan Deposits (Qof): @5'- Silty SAND: brown, slightly moist, loose	
965			R-3	11 13 11	111.5	2.5		@7.5'- Silty SAND: brown, dry, medium dense	
	10		R-4	7 9 9	110.2	3.2		@10'- Silty SAND: olive brown, dry, medium dense	
960									
	15		SPT-1	7 16 19		1.8	SP-SM	@15'- SAND with Silt and Gravel: grayish brown, dry, dense	
955									
	20		R-5	14 33 50/6"	118.4	1.5		@20'- SAND with Silt and Gravel: grayish brown, dry, very dense	
950									
	25		SPT-2	9 17 18		1.8		@25'- SAND with Silt and Gravel: grayish brown, dry, dense	
945									
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-4

Date: 3/24/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~970' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
965	0	B-1	R-1	7 9 10	115.1	2.4	SM	@0' to 7.5' - Undocumented Artificial Fill (afu): @0'- SAND with Gravel: yellowish brown, dry @2.5'- Silty SAND with Gravel: brown, dry, medium dense	
965	5		R-2	7 12 10	114.1	1.8		@5'- Silty SAND with Gravel: grayish brown, dry, medium dense	
960	10		R-3	8 10 14	113.3	1.9	SP-SM	@7.5' to T.D. - Old Alluvial-Fan Deposits (Qof): @7.5'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
955	15		R-4	9 17 17	103.6	2.3		@10'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
950	20		R-5	11 23 31	115.4	1.8		@15'- SAND with Silt and Gravel: grayish brown, dry, dense	
945	25		SPT-1	10 24 20		1.7		@20'- SAND with Silt and Gravel: grayish brown, dry, very dense	
			R-6	19 50/6"	109.9	2.0		@25'- SAND with Silt and Gravel: grayish brown, dry, very dense	
								Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-5

Date: 4/4/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~975' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
970	0	B-1						@0' to 5' - Undocumented Artificial Fill (afu): @0'- 5" of Concrete with Rebar; Silty SAND: yellowish brown, dry	
	5		R-1	5 9 11	121.1	14.7	SM	@5' to T.D. - Old Alluvial-Fan Deposits (Qof): @5'- Silty SAND: brown, very moist, medium dense	
			R-2	6 10 15	114.5	3.6		@7.5'- Silty SAND: yellowish brown, dry, medium dense	CO
965	10		R-3	7 17 32	122.4	1.9	SP-SM	@10'- SAND with Silt and Gravel: grayish brown, dry, dense	
960	15		R-4	15 28 28	120.3	2.6		@15'- SAND with Silt and Gravel: grayish brown, dry, dense	
955	20		SPT-1	17 50/6"		1.9		@20'- SAND with Silt and Gravel: grayish brown, dry, very dense	
950	25		R-5	10 20 30	121.3	4.2		@25'- SAND with Silt and Gravel: grayish brown, dry, dense	
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-6

Date: 4/4/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~976' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
975	0							@0' to 5' - Undocumented Artificial Fill (afu): @0'- 8" of Concrete with Rebar; Silty SAND: reddish brown, dry	CR EI MD
970	5		R-1	4 5 9	114.4	7.9	SM	@5' to T.D. - Old Alluvial-Fan Deposits (Qof): @5'- Silty SAND: brown, moist, medium dense	
			R-2	7 12 18	119.5	3.5	SP-SM	@7.5'- SAND with Silt: light olive brown, dry, medium dense	CO
965	10		R-3	9 14 24	115.8	2.0		@10'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
960	15		SPT-1	6 11 17		3.0		@15'- SAND with Silt: grayish brown, dry, dense	
955	20		R-4	5 10 24	115.5	16.9	SM	@20'- Silty SAND: grayish brown, very moist, medium dense	
950	25		SPT-2	11 20 25		2.1	SP-SM	@25'- SAND with Silt and Gravel: grayish brown, dry, very dense	
								Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-7

Date: 4/4/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~976' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
975	0	B-1						@0' to 5' - Undocumented Artificial Fill (afu): @0'- 10" of Concrete with no Rebar; Silty SAND with Gravel: reddish brown, dry, loose	
970	5		R-1	4 8 20	121.4	4.1	SP-SM	@5' to T.D. - Old Alluvial-Fan Deposits (Qof): @5'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
			R-2	12 23 46	122.1	3.0		@7.5'- SAND with Silt and Gravel: grayish brown, dry, very dense	
965	10		R-3	16 30 50/6"	127.9	1.7		@10'- SAND with Silt and Gravel: grayish brown, dry, very dense	
960	15		R-4	8 11 17	104.7	3.0	SM	@15'- Silty SAND: grayish brown, dry, medium dense	
955	20		SPT-1	15 13 16		2.5	SP-SM	@20'- SAND with Silt and Gravel: grayish brown, dry, dense	
950	25		R-5	19 30 50/3"	121.8	2.3		@25'- SAND with Silt and Gravel: grayish brown, dry, very dense	
	30							Total Depth = 25' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-8

Date: 4/4/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~975' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
970	0	B-1						@0' to 5' - Undocumented Artificial Fill (afu): @0'- 9.5" of Concrete with Rebar; Silty SAND: brown, dry	EI MD
			R-1	3 5 7	116.5	5.1	SM	@3'- Silty SAND: brown, slightly moist, loose	
	5		R-2	4 6 13	121.3	8.1	SM	@5' to T.D. - Old Alluvial-Fan Deposits (Qof): @5'- Silty SAND: brown, moist, medium dense	
			R-3	6 18 26	111.4	2.2	SP-SM	@7.5'- SAND with Silt and Gravel: grayish brown, dry, dense	
965	10		R-4	15 23 30	123.9	1.9	SP	@10'- SAND with Gravel: grayish brown, dry, dense	
960	15		SPT-1	8 9 11		4.0	SP-SM	@15'- SAND with Silt and Gravel: grayish brown, dry, medium dense	
955	20		R-5	7 11 20	114.4	13.8	SM	@20'- Silty SAND: olive gray, very moist, medium dense	
950	25		SPT-2	11 17 18		2.3	SP-SM	@25'- SAND with Silt and Gravel: grayish brown, dry, dense	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
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 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-8

Date: 4/4/2022	Drilling Company: Cal Pac Drilling
Project Name: Dedeaux - Rialto	Type of Rig: Truck Mounted
Project Number: 22041-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~975' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By JMN Sampled By JMN Checked By DJB DESCRIPTION	Type of Test
	30		R-6	21 25 50/6"	121.8	1.6	SP-SM	@30'- SAND with Silt and Gravel: grayish brown, dry, very dense	
945	35		SPT-3	14 26 37		2.1		@35'- SAND with Silt and Gravel: grayish brown, dry, very dense	
940	40		R-7	25 50/6"	114.9	2.3		@40'- SAND with Silt: grayish brown, dry, very dense	
935	45		SPT-4	8 16 18		11.0	ML	@45'- SILT with Sand: grayish brown, moist, hard	
930	50		R-8	10 19 32	105.2	9.6		@50'- SILT: olive brown, slightly moist, hard	
925	55							Total Depth = 50' Groundwater Not Encountered Backfilled with Cuttings on 3/24/2022	
	60								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

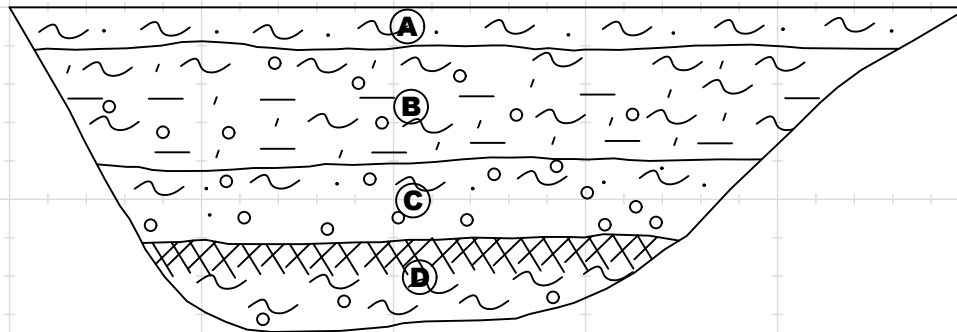
TEST TYPES:
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 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Project Name: Willow Ave, Rialto			Logged By: JMO		Trench No.: TP-1				
Project Number: 22041-01			Date: 3/29/2022		Engineering Properties:				
Equipment: Backhoe			Location:						
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)		
	A	@ 0' to 6' - <u>Undocumented Artificial Fill:</u> @ 0 - 1' Sandy SILT - yellowish brown, slightly moist to moist,	Afu	ML	B-1 @ 0-2'				
	B	@ 1 - 4' - Gravelly SAND: yellowish orange brown, dry		SP					
	C	@ 4 - 6' - Sandy Silt with Gravel: gray-orange brown, dry to slightly moist; trace Clay		ML					
	D	@ 6' to T.D. - <u>Old Alluvial-Fan Deposits:</u> @ 6 - 8' - Sandy SILT: brown/gray, dry, stiff with gravels over orange silt, slightly moist; trace Clay	Qof	ML	B-2 @ 7-8'				

GRAPHICAL REPRESENTATION BELOW:

Elevation: 975' MSL

Trend: E-W



Total Depth: 8'
Groundwater: N/A
Backfilled: 3/29/22

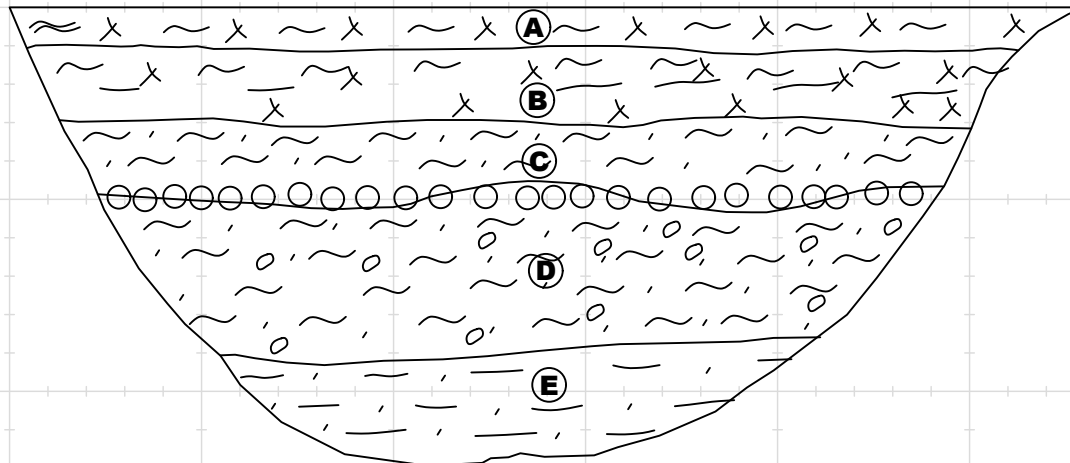
scale: 1 in = 5 ft

Project Name: Willow Ave, Rialto			Logged By: JMO			Trench No.: TP-2				
Project Number: 22041-01			Date: 3/29/2022			Engineering Properties:				
Equipment: Backhoe			Location:							
Geologic Attitudes	Unit	SOIL DESCRIPTION:				GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	@ 0' to 9' - <u>Undocumented Artificial Fill:</u>				Afu				
	B	@ 1 - 3' - Silty SAND with Gravel: lighter brown, dry to slightly moist, trace Clay					SM	B-1 @ 2-4'		
	C	@ 3 - 5' - Sandy SILT with Clay: darker brown, moist; rootlets					ML	B-2 @ 4-6'		
	D	@ 5 - 9' - Sandy SILT with Clay: orange brown, slightly moist; some Gravel								
	E	@ 9' to T.D. - <u>Old Alluvial-Fan Deposits:</u>				Qof	SP	B-3 @ 8-10'		
		@ 9 - 12' - SAND with Gravel: yellowish gray, slightly moist; medium to coarse grained								

GRAPHICAL REPRESENTATION BELOW:

Elevation: 974' MSL

Trend: N-S



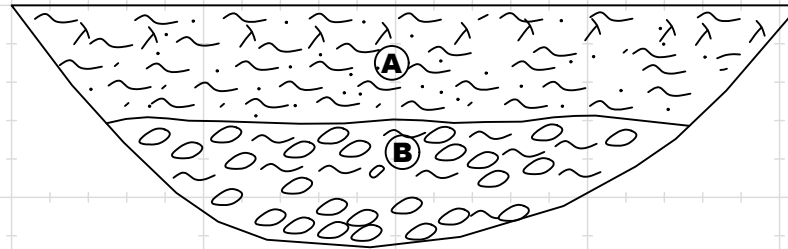
Total Depth: 12'
Groundwater: N/A
Backfilled: 3/29/22

scale: 1 in = 5 ft

Project Name: Willow Ave, Rialto			Logged By: JMO		Trench No.: TP-3				
Project Number: 22041-01			Date: 3/29/2022		Engineering Properties:				
Equipment: Backhoe			Location:						
Geologic Attitudes	Unit	SOIL DESCRIPTION:			GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	@ 0' to T.D. - <u>Undocumented Artificial Fill:</u>			Afu	ML	B-1 @ 2-4'		
		@ 0 - 3' - Sandy SILT: yellowish brown, slightly moist gravel and asphalt chunks; rootlets; trace Clay							
	B	@ 3' - 6.5' - Silty SAND with Gravel: gray, dry; miscellaneous trash				SM			

GRAPHICAL REPRESENTATION BELOW:

Elevation: 970' MSL



Total Depth: 6.5'
Groundwater: N/A
Backfilled: 3/29/22

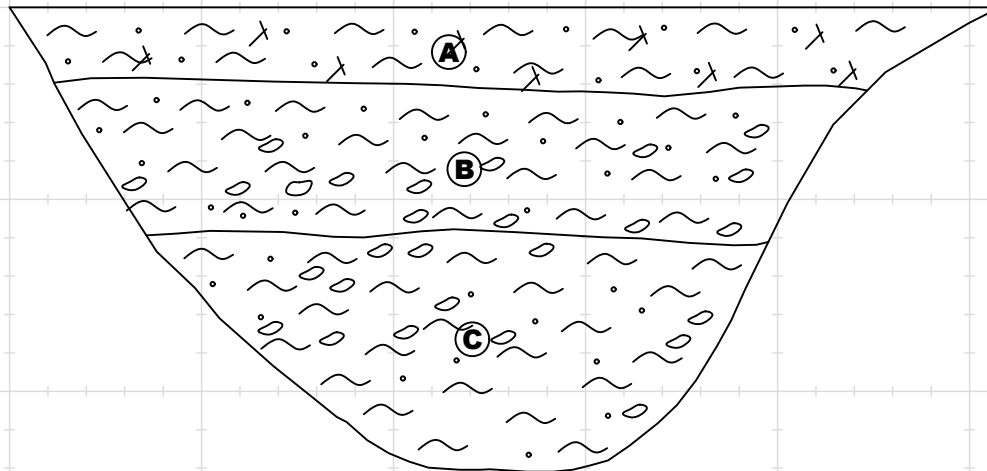
scale: 1in = 5 ft

Project Name: Willow Ave, Rialto			Logged By: JMO		Trench No.: TP-4				
Project Number: 22041-01			Date: 3/29/2022		Engineering Properties:				
Equipment: Backhoe			Location:						
Geologic Attitudes	Unit	SOIL DESCRIPTION:			GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
		@ 0' to T.D. - Undocumented Artificial Fill: A @ 0 - 2' - Sand SILT with Clay: red-brown to brown, slightly moist, many gravels; miscellaneous trash; rootlets B @ 2 - 6' - Sandy SILT with Clay: dark brown, slightly moist, stiff, many gravels, miscellaneous trash C @ 6 - 12' - Sandy SILT with Clay: dark gray to brown, slightly moist; miscellaneous trash			Afu	ML	B-1 @ 5-6' B-2 @ 10-12'		

GRAPHICAL REPRESENTATION BELOW:

Elevation: 972' MSL

Trend: E-W



Total Depth: 12'
Groundwater: N/A
Backfilled: 3/29/22

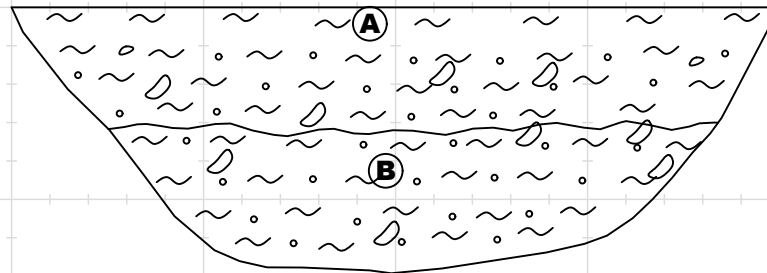
scale: 1in = 5 ft

Project Name: Willow Ave, Rialto			Logged By: JMO		Trench No.: TP-5				
Project Number: 22041-01			Date: 3/29/2022		Engineering Properties:				
Equipment: Backhoe			Location:						
Geologic Attitudes	Unit	SOIL DESCRIPTION:			GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	@ 0' to 3' - Undocumented Artificial Fill:			Afu		B-1 @ 0-2'		
	B	@ 0 - 3' - Sandy SILT with Gravels: light brown to yellowish brown, very dry; trace Clay				ML			
		@ 3' to T.D. - Old Alluvial-Fan Deposits:			Qof		B-2 @ 2-4'		
		@ 3 - 6.5' - Sandy SILT with Gravels: light brown to yellowish brown, very dry; caliche; trace Clay				ML			

GRAPHICAL REPRESENTATION BELOW:

Elevation: 972' MSL

Trend: N-S



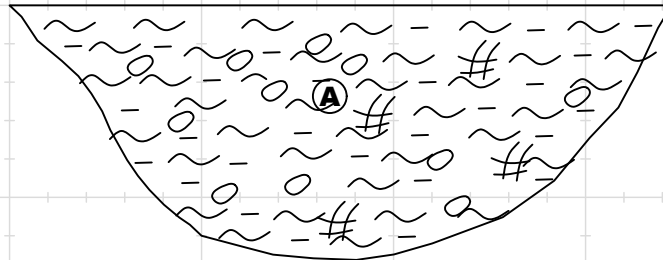
Total Depth: 6.5
Groundwater: N/A
Backfilled: 3/29/22

scale: 1 in = 5 ft

Project Name: Willow Ave, Rialto		Logged By: JMO	Trench No.: TP-6				
Project Number: 22041-01		Date: 3/29/2022	Engineering Properties:				
Equipment: Backhoe		Location:					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	@ 0' to T.D. - Undocumented Artificial Fill: @ 0 - 6.5' - Silty SAND with Gravel: dark brown, slightly moist; miscellaneous trash	Afu	SM	B-1 @ 4-6'		

GRAPHICAL REPRESENTATION BELOW:

Elevation: 971' MSL



Total Depth: 6.5'
Groundwater: N/A
Backfilled: 3/29/22

scale: 1in = 5 ft

Appendix C
Laboratory Test Results

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: Dedeaux - Rialto
 Project No.: 22041-01
 Boring No.: HS-3
 Sample No.: R-4
 Sample Description: Olive brown silty sand (SM)

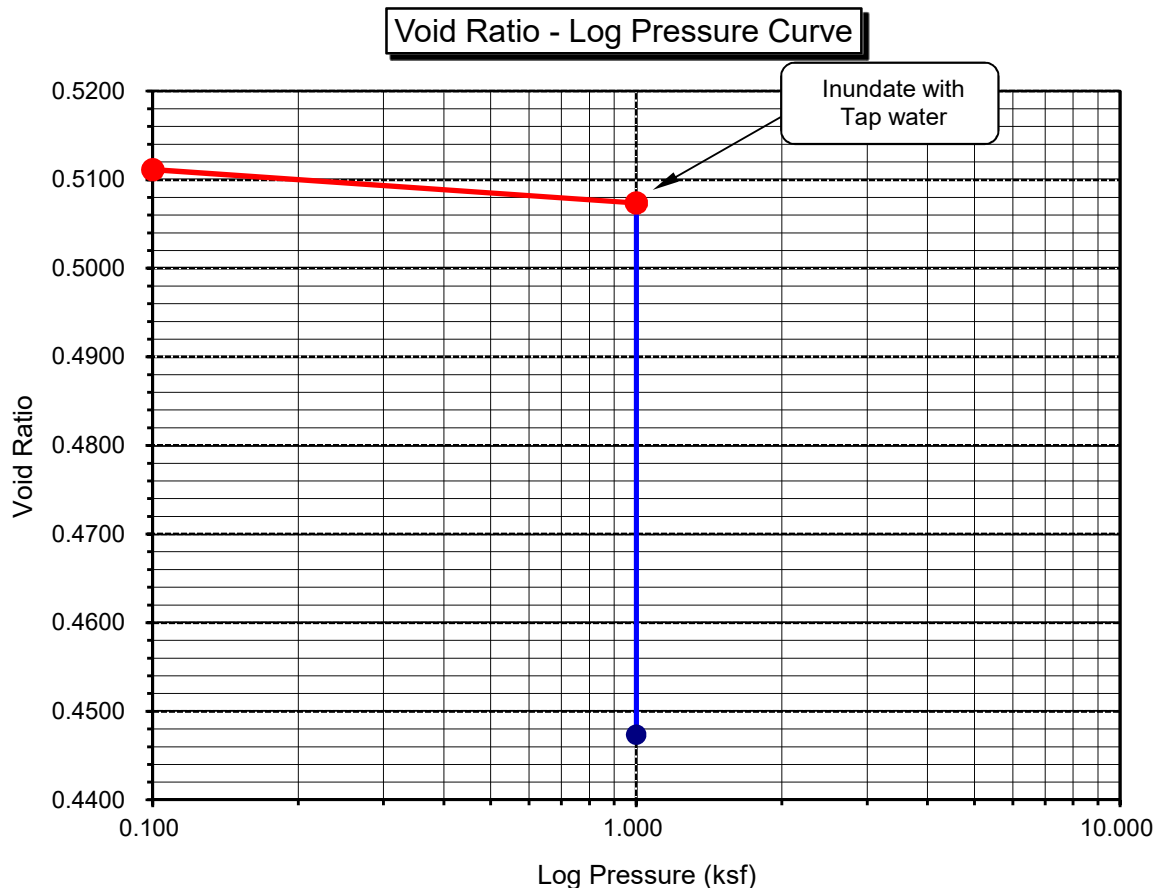
Tested By: G. Bathala Date: 04/13/22
 Checked By: J. Ward Date: 05/02/22
 Sample Type: Ring
 Depth (ft.): 10.0

Initial Dry Density (pcf):	111.5
Initial Moisture (%):	3.17
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2716
Diameter(in):	2.415

Final Dry Density (pcf):	116.5
Final Moisture (%) :	13.0
Initial Void Ratio:	0.5113
Specific Gravity(assumed):	2.70
Initial Saturation (%)	16.7

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2715	0.9999	0.00	-0.01	0.5111	-0.01
1.000	0.2661	0.9945	0.29	-0.55	0.5074	-0.26
H2O	0.2264	0.9548	0.29	-4.52	0.4474	-4.23

Percent Swell (+) / Settlement (-) After Inundation = -3.98



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: Dedeaux - Rialto
 Project No.: 22041-01
 Boring No.: HS-5
 Sample No.: R-2
 Sample Description: Yellowish brown silty sand (SM)

Tested By: G. Bathala Date: 04/13/22
 Checked By: J. Ward Date: 05/02/22
 Sample Type: Ring
 Depth (ft.): 7.5

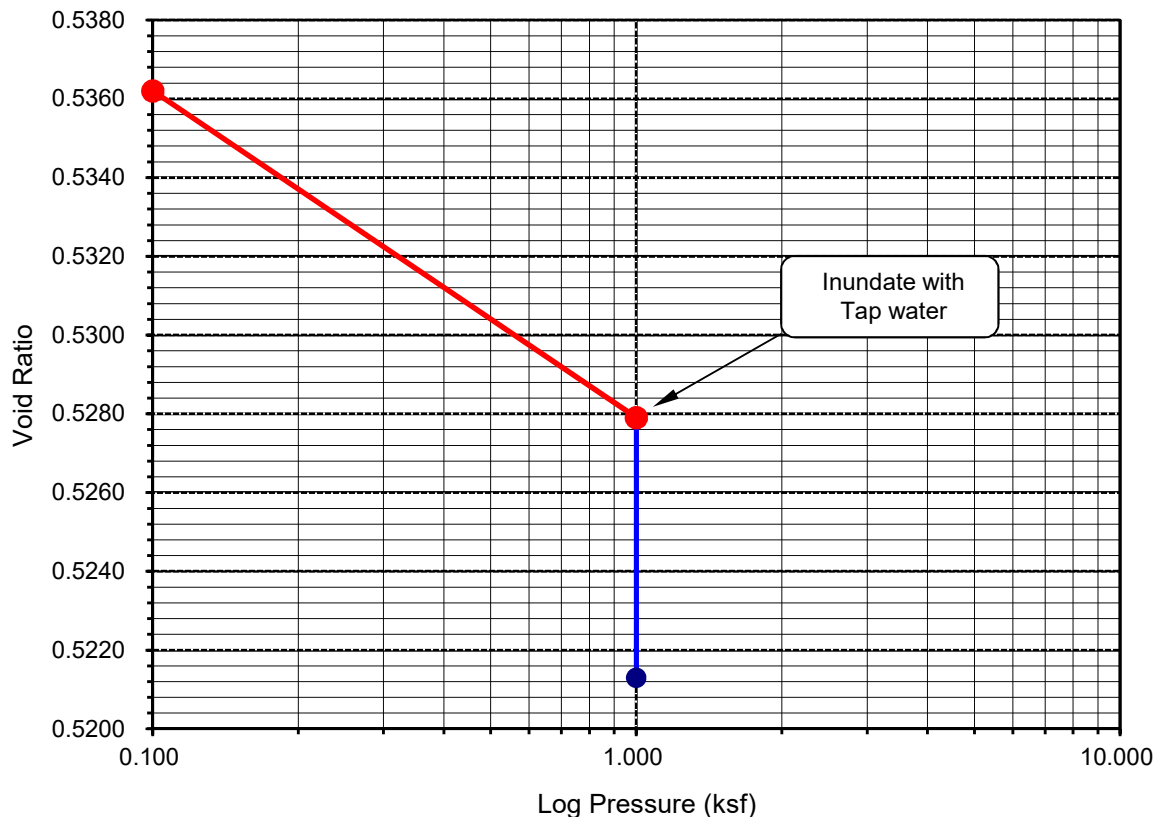
Initial Dry Density (pcf):	109.7
Initial Moisture (%):	3.56
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3628
Diameter(in):	2.415

Final Dry Density (pcf):	110.8
Final Moisture (%) :	14.8
Initial Void Ratio:	0.5362
Specific Gravity(assumed):	2.70
Initial Saturation (%)	17.9

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3628	1.0000	0.00	0.00	0.5362	0.00
1.000	0.3567	0.9939	0.07	-0.61	0.5279	-0.54
H2O	0.3524	0.9896	0.07	-1.04	0.5213	-0.97

Percent Swell (+) / Settlement (-) After Inundation = -0.43

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: Dedeaux - Rialto
 Project No.: 22041-01
 Boring No.: HS-6
 Sample No.: R-2
 Sample Description: Light olive brown poorly-graded sand with silt (SP-SM)

Tested By: G. Bathala Date: 04/13/22
 Checked By: J. Ward Date: 05/02/22
 Sample Type: Ring
 Depth (ft.): 7.5

Initial Dry Density (pcf):	112.6
Initial Moisture (%):	3.49
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2982
Diameter(in):	2.415

Final Dry Density (pcf):	113.4
Final Moisture (%) :	13.3
Initial Void Ratio:	0.4966
Specific Gravity(assumed):	2.70
Initial Saturation (%)	19.0

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2982	1.0000	0.00	0.00	0.4966	0.00
1.000	0.2922	0.9940	0.27	-0.60	0.4917	-0.33
H2O	0.2886	0.9904	0.27	-0.96	0.4863	-0.69

Percent Swell (+) / Settlement (-) After Inundation = -0.36

Void Ratio - Log Pressure Curve

