Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

November 30, 2017

Project No. 11805.001

To: Bridge Development Partners
1334 Parkview Avenue, Suite 310
Manhattan Beach, California 90266

Attention: Mr. Tom Ashcraft

Subject: Geotechnical Exploration and Infiltration Testing for the Proposed Commercial Development, North and South of Vineyard Avenue and West of Maple Avenue, Rialto Area of Unincorporated San Bernardino County, California

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has conducted geotechnical exploration and infiltration testing for the proposed development at Vineyard Avenue on the west of side of Maple Avenue, in the Rialto area of unincorporated San Bernardino County, California. The site is bounded on the north by single-family residences, east by Maple Avenue, south by a vacant field, and west by both industrial properties and a vacant field. The purpose of our study has been to review the geotechnical conditions at the site and to identify significant geotechnical constraints to site development based on existing data. In addition to reports, maps, and aerial photographs available in our in-house library, we have reviewed the Conceptual Site Plan Scheme 8 prepared by Herdman Architecture and Design, not dated, and the Geotechnical Investigation Report by CHJ, Inc., dated April 19, 2004, provided to us by you, and comment on aspects of these references. We have also conducted infiltration testing for use in design of infiltration facilities for the proposed development at the proposed locations provided to us by you.

Our work has included the following:

- We reviewed previous geotechnical reports as well as geologic reports and maps relevant to the site and available from our in-house library. We also reviewed historic aerial photographs of the site dating back to 1938.

- Visited the site to observe existing surface conditions.

- Coordinated with Underground Service Alert (USA) prior to excavating borings so that utility companies could mark public utilities onsite.

- Conducted well permeameter tests within three borings (LB-1 through LB-3) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. The well permeameter tests were conducted based on the USBR 7300-89 method and in general accordance with San Bernardino County guidelines. The tests were conducted at depths ranging from approximately 6 to 10 feet (bgs) to estimate the infiltration rate for use of the proposed infiltration facilities. We used water from on-site faucets to provide water for the tests.

- Evaluated the collected data.

- Prepared this report to present the results of our geotechnical review and infiltration testing.

<u>Site Conditions and Proposed Development</u>

Based on our correspondence and the documents provided to us by you, the site of proposed development at Vineyard Avenue and west of Maple Avenue will consist of an approximately 392,500-square-foot commercial building, drainage, utility, hardscape, parking, and associated improvements in the Rialto area of unincorporated San Bernardino County, California.

Review of historic aerial photographs dating show in 1938 the southern portion of the site being used as an orchard with the northern portion being a vacant dirt lot. By 1959, the orchards had been removed and Vineyard Avenue had been constructed as what appears to be a dirt road traversing east-west across the center of the site, with the rest of the property being undeveloped. In 1980, aerial photographs show a single-family ranch-style residence in the southeast portion of the site that is still present today. Aerial photographs from 2005 show stockpiles in the central portion of the site just north of

Vineyard Avenue, which remain present today. The remainder of the site appears to have been a vacant dirt lot since at least 1959.

The rest of the parcel is bounded by Maple Avenue to the east, single-family homes to the north, a vacant field to the south, and both industrial properties and vacant fields to the west (see Site Location Map, Figure 1). The soil exposed at the surface is generally sand, gravel and cobbles. Vegetation generally consists of grasses; and shrubs and trees on the residential property. The site generally slopes to the southeast with approximately 25 feet of elevation difference.

Based on discussions and conceptual site plans from you, we understand infiltration of storm water will be required for the development and that the location of these facilities are to be located primarily in the southeast area of the site.

Previous Geotechnical Reports

CHJ Inc. (2004) conducted a geotechnical investigation of the site with the exception of the southeastern quadrant where the current residence is. The investigation included the excavation, logging, and sampling of six exploratory trenches. CHJ Inc. concluded that the site was geotechnically feasible to develop provided the recommendations presented in their report were implemented.

Earth Units

The site is mapped as being underlain with young alluvial fan deposits from the late Holocene (Morton et al., 2006). These alluvial valley deposits are described as unconsolidated to slightly consolidated coarse-grained sand to bouldery alluvial-fan deposits of the Lytle Creek fan. CHJ encountered boulders up to 24 inches in diameter within their test pits. The onsite soils are typically dense to very dense.

Based on our limited subsurface exploration, we encountered alluvial soil deposits consisting of gravelly sand and cobbles. CHJ Inc.'s report described the subsurface soils found in their test pits as dense to very dense gravelly sand with cobbles and boulders to their maximum depth explored. CHJ Inc. also encountered up to 2 feet of artificial fill in two of their test pits (Test Pit No. 5 and 6) located in the central and southwestern areas of the site.

The native subsurface soils encountered in our excavations consisted mainly of sand, gravel, and cobbles to their maximum depth explored. These excavations were located on the southern edge of existing residential property at the locations of the proposed infiltration facilities (Figure 2, Test Location Map). These excavations were primarily for use in evaluating the subsurface soils for infiltration and cover a very limited area of the site.

Laboratory Testing

Results of lab testing done by CHJ Inc. indicated on-site soils to be mildly corrosive to ferrous metals, and PH values of the soils were found to be alkaline. We conducted corrosivity lab testing on samples from our borings and the results suggested mildly to moderately corrosive soils.

Infiltration Testing

We conducted infiltration testing in the areas of the proposed infiltration facilities for the proposed development. Our infiltration depths ranged from approximately 6 to 10 feet below the existing ground surface, and were based on the anticipated depth of the facilities, as well as on evaluation of the suitability of the soil encountered during drilling.

Three well permeameter tests (LB-1 through LB-3) were conducted to estimate the infiltration rate at specific locations of the site. The well permeameter tests were conducted inside the borings with test water levels ranging from 2.5 to 6.0 feet below ground surface for LB-1, 4.2 to 10.0 feet for LB-2, and 4.9 to 10.0 feet for LB-3.

Well permeameter tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. This is a clean-water, small-scale test, and as such, correction factors need to be applied. The test consists of excavating a boring to the depth of the test (or deeper if it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand is placed in the boring bottom to support temporary perforated well casing pipe and a float valve. In addition, coarse sand is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float valve, lowered into the boring inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a relatively constant water head in the boring. The test was conducted based on the USBR 7300-89 test method.

Small-scale infiltration test rates were measured at the 3 well permeameter locations (LB-1 through LB-3). At location LB-1, the small-scale infiltration test rate was estimated to be 2.7 inches per hour, and was tested within sandy gravel alluvial soils. At location LB-2, the small-scale infiltration test rate was estimated to be 8.0 inches per hour, and was tested within sandy gravel alluvial soils. At location LB-3, the small-scale infiltration test rate was estimated to be 10.0 inches per hour, and was tested within sandy gravel alluvial soils. These are raw values, before applying an appropriate factor of safety or correction factor. Based on these results, the onsite soils at the depths tested resulted are anticipated to have high infiltration rates. Design rates, correction factors, and other infiltration facility recommendations are discussed below.

Groundwater

Using the California Department of Water Resources Water Data Library (2017), a well located approximately ½ mile to the east (#341412N1174003W001) showed depth to groundwater in 2011 to be on the order of 394 feet which. We found the most current depth to groundwater to be on the order of 420 feet taken from the same well in September of 2017. Shallow groundwater is not anticipated.

Seismic Hazards

The proposed development is not within a currently designated State established Earthquake Fault Zone for active surface faulting, and San Bernardino County (2010) has not identified any faults or fault zones through the site. No known active faults have been mapped onsite nor trending toward the site. The nearest known active faults are San Jacinto-San Bernardino Fault, located about 0.7 mile to the northeast, Cucamonga Fault, located about 3.4 miles to the northwest, and the San Andreas Fault, located about 6.7 miles to the northeast. However, as with the majority of southern California, the site is expected to be prone to strong seismic shaking.

San Bernardino County (2010) has this area mapped outside of any liquefaction or landslide hazard areas.

Seismic Design Parameters

We have provided seismic design parameters based on the UBC Seismic Map. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the current 2016 CBC. The CBC seismic design parameters listed in Table 1 below should be considered for the seismic analysis of the subject site.

Table 1 - 2016 CBC Seismic Design Parameters

Description (2016 CBC reference)	Design Value
Site Longitude (decimal degrees)	-117.4065
Site Latitude (decimal degrees)	34.1410
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, $\mathbf{S_s}$ (Figure 1613.3.1(1))	1.946
Mapped Spectral Response Acceleration at 1s Period, $\mathbf{S_1}$ (Figure 1613.3.1(2))	0.867
Short Period Site Coefficient at 0.2s Period, $\mathbf{F_a}$ (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, $\mathbf{F_v}$ (Table 1613.3.3(2)	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{MS}}$ (Eq. 16-37)	1.946
Adjusted Spectral Response Acceleration at 1s Period, $\mathbf{S_{M1}}$ (Eq. 16-38)	1.300
Design Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{DS}}$ (Eq. 16-39)	1.297
Design Spectral Response Acceleration at 1s Period, $\mathbf{S_{D1}}$ (Eq. 16-40)	0.867

Conclusions and Recommendations

Based on our review of published reports and maps, review of the conceptual site plan, and review of the data presented in CHJ Inc.'s geotechnical report, development of the site is feasible from a geotechnical viewpoint. Liquefaction and seismic settlement are not considered constraints to the project.

Specific recommendations for construction of the development of the site were provided by CHJ Inc. (2004). Those recommendations should be implemented during construction of the site. Additionally, seismic design parameters should be updated to be in accordance with the 2016 California Building Code.

Additional laboratory testing and geotechnical review of the development should be conducted as the project proceeds.

Infiltration Recommendations

Infiltration Rate:

For onsite undisturbed alluvial soils that are granular with a low fines content, we recommend an unfactored (small-scale) incremental infiltration rate of 4 inches per hour. These measured rates are applicable at the specific locations and depths tested. Infiltration rates are anticipated to vary significantly at various depths. It should be confirmed during infiltration facility excavation that the excavations penetrate into undisturbed granular soils.

The incremental infiltration rate is defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface. We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with San Bernardino County guidelines, since monitoring of actual facility performance has shown that actual infiltration rates are lower than for small-scale tests. The small-scale infiltration rate should be divided by a correction factor of at least 2 for buried chambers and at least 2.5 for open basins, but the correction/safety factor may be higher based on project-specific aspects.

If dry wells are considered, we suggest that they be planned with clusters of dry wells per general location based on the presumed-conservative infiltration rate. After the first dry well is constructed in each general location, it should be tested for infiltration. If the tested infiltration rates are sufficient to reduce the number of dry wells at that location, some or all of the remaining planned dry wells may be omitted, as appropriate, based on review of the test data. Due to the very granular nature of the soil at this site, we anticipate that significant caving may be encountered during drilling of dry wells. In addition, boulders will be encountered.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values will be reduced over time if silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

Additional Review and Evaluation:
Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review infiltration plans, including locations and depths of proposed facilities. Further testing may be required depending on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations:
The periodic flow of water carrying sediments in the basin or chamber, plus the introduction of wind-blown sediments and sediments from erosion of the basin side walls, can eventually cause the bottom of the basin or chamber to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within storm water, especially during construction of the project and prior to achieving a mature landscape on site. As it is typically very difficult to remove silt from buried infiltration facilities, we recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the buried infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these. Such features should be brought to Leighton's attention as they are identified.

Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process.

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overfilling to not be a concern to the facility or nearby improvements.

For buried chambers that allow interior standing water, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the cambers.

Additional Design Considerations (Particularly for Open Basins):
If open basins are planned, the soils that will be exposed at the bottom of the basin are critical to the basin's success.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber, these conditions would be of less concern.

Recreation areas should not be constructed within basin bottoms or below the spillway level.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install infiltration trenches or dry wells in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/dry well depth, would be developed based on conditions observed. Such a contingency should be anticipated for open basins.

Construction Considerations:

We recommend that Leighton evaluate the infiltration facility excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty or clayey soils are exposed.

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations:

The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

We appreciate the opportunity to provide our services for this review. If you have any questions, please contact this office at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Philip A. Buchiarelli, CEG 1715
Principal Geologist

Jason D. Hertzberg, GE 2711
Principal Engineer

BER/MM/JDH/PB/rsm

Attachments: References
Figure 1 - Site Location Map
Figure 2 - Test Location Map
Boring Logs and Infiltration Test Summary
Lab Results
Seismic Parameters

Distribution: (1) electronic copy

REFERENCES

California Building Standards Commission, 2016, 2016 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on 2015 International Building Code, Effective January 1, 2017.

California Department of Water Resources (CDWR), 2017, Water Data Library, www.ca.gov/waterdatalibrary

California Department of Conservation, 1995, Earthquake Fault Zones, Devore Quadrangle, Revised Official Map Effective: June 1, 1995.

CHJ Inc., 2004, Geotechnical Investigation, Proposed 11.7 Acre Residential Development Vineyard Avenue West of Maple Avenue, Rialto, California, Job No. 04267-3, dated April 19, 2004.

Morton, D.M., Miller, F.K., 2006, Geologic Map of the San Bernardino and Santa Ana 30'X60' Quadrangles, California: U.S. Geological Survey, Open File Report 2006-1217, scale 1:100,000.

NETR, 2017, Historic Aerials by Netronline, www.historicaerials.com

San Bernardino County, 2010, San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays, FH21 C Devore.

Project: 11805.001	Eng/Geol: JDH/PB
Scale: 1 " = 2,000 '	Date: November 2017

Base Map: Bing Maps 2017
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

SITE LOCATION MAP

Geotechnical Due Diligence Review
Proposed Commercial Development
Vineyard Avenue West of Maple Avenue
Unincorporated San Bernardino County, California

Figure 1

Leighton

Map Saved as P:\Drafting\11805\001\Maps\11805-001_F01_SLM_2017-10-24.mxd on 10/24/2017 4:08:18 PM

Project: 11805.001	Eng/Geol: JDH/PB
Scale: 1 " = 200 '	Date: November 2017

Base Map: Google Earth, 2017
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

TEST LOCATION MAP

Geotechnical Due Diligence Review
Proposed Commercial Development
Vineyard Avenue West of Maple Avenue
Unincorporated San Bernardino County, California

Figure 2

Leighton

Map Saved as P:\Drafting\11805\001\Maps\11805-001_F02_TLM_2017-10-24.mxd on 10/24/2017 4:09:59 PM

GEOTECHNICAL BORING LOG LB-1

Project No.	11805.001	**Date Drilled**	9-27-17
Project	Bridge Development Rialto	**Logged By**	B. Rodriguez
Drilling Co.	2R	**Hole Diameter**	10"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1517'
Location	see Figure 2, Test Location Map	**Sampled By**	B. Rodriguez

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								@surface: sand, gravel, & cobbles **Quaternary Alluvium (Qal)** note: gravel and cobbles in spoils	
1515										
				R-1	32 50/6				@2.5' fractured cobble, 2.5-inch diameter, very dense note: gravel and cobbles in spoils	
	5			R-2	30 24 28				@5' NO RECOVERY, dense note: gravel and cobbles in spoils	
1510										
	10			R-3	50/4				@10' NO RECOVERY, very dense note: gravel and cobbles in spoils	
1505										
	15			R-4	50/4				@15' NO RECOVERY, very dense note: sand and gravel in spoils	
1500										
	20			R-5	50/5				@20' fractured cobble, 2.5-inch diameter, very dense	
1495										
	25			S-6	21 50/5			SW	@25' SANDY GRAVEL (GW), very dense, light brown, moist, coarse sand, angular, nonplastic	
1490									**Total depth 26.5 feet** **No groundwater encountered when drilling** **Backfilled with soil cuttings on 9/29/17**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-2

Project No.	11805.001	**Date Drilled**	9-27-17
Project	Bridge Development Rialto	**Logged By**	B. Rodriguez
Drilling Co.	2R	**Hole Diameter**	10"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1517'
Location	see Figure 2, Test Location Map	**Sampled By**	B. Rodriguez

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								@surface: gravel, sand, and cobbles **Quaternary Alluvium (Qal)**	
1515										
	5									
1510										
				R-1	50/3	117	1	GW	@8.5' SAND (SW) with silt and gravel, very dense, light brown, dry, coarse sand, subangular, nonplastic, trace fines, 2.5-inch fractured rock in shoe	SA, M
	10								**Total depth 10 feet** **No groundwater encountered when drilling** **Backfilled with soil cuttings on 9/29/17**	
1505										
	15									
1500										
	20									
1495										
	25									
1490										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-3

Project No.	11805.001	**Date Drilled**	9-27-17
Project	Bridge Development Rialto	**Logged By**	B. Rodriguez
Drilling Co.	2R	**Hole Diameter**	10"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	1517'
Location	see Figure 2, Test Location Map	**Sampled By**	B. Rodriguez

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1					@surface: gravel, sand, and cobbles **Quaternary Alluvium (Qal)**	MD, CR
1515										
	5									
1510										
				R-1	21 42 38	122	2	GW	@8.5' SANDY GRAVEL (GW), very dense, light brown, moist, coarse sand, subrounded, nonplastic, 1.5-inch average gravel	M
	10									
1505									**Total depth 10 feet** **No groundwater encountered when drilling** **Backfilled with soil cuttings on 9/29/17**	
	15									
1500										
	20									
1495										
	25									
1490										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: Bridge Development Rialto 11805.001

Exploration #/Location	LB-1
Depth Boring drilled to (ft)	26.5
Tested by	BER
USCS Soil Type in test zone	GW
Weather (start to finish)	sunny
Liquid Used/pH	Faucet Water
Measured boring diameter	10 in.
Approx Depth to GW below GS	100 ft
Well Prep	Drilled to 26.5 feet, caved to 6 feet, placed perforated 4-inch pipe with 1-inch pilot tube pipe

5 Well Radius, "r"

Initial estimated Depth to Water Surface (in.): 32.712
Average depth of water in well, "h" (in.): 39.288
approx. h/r: 7.9
Tu (Fig. 8): 97.3 ft
Tu>3h?: yes, OK

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	6. ft		72	
Pilot Tube stickup (+ is above ground)		3. in.	3	
Depth to top of float assembly from top of pilot tube		13. in	13	10 Depth below GS (in.)
Float Assembly ID	C			
Float assembly Extension length (in.)		30		

Flow Meter:

Meter ID: 28
Meter Unit: Gallons
0.05 gallons/pulse

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube): ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3): from supply	Vol Change (in.^3): from Δh	Vol Change (in.^3): Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date: 9/29/2017	Start time: 8:25	Gallons																			
9/29/17	8:25	1240.9		4.92					0	56.0	16.0										
9/29/17	8:40	1248.6		3.59				15	15	40.1	31.9	15.96	24	1779	-501	1278	85	5111	0.9	1.07	**5.67**
9/29/17	9:03	1258.8		2.95				23	38	32.4	39.6	7.68	36	2356	-241	2115	92	5518	0.9	0.93	**4.23**
9/29/17	9:17	1263.9		2.85				14	52	31.2	40.8	1.2	40	1178	-38	1140	81	4888	0.9	0.82	**3.36**
9/29/17	9:33	1269.3		2.79				16	68	30.5	41.5	0.72	41	1247	-23	1225	77	4593	0.9	0.75	**3.09**
9/29/17	9:49	1273.8		2.8				16	84	30.6	41.4	-0.12	41	1040	4	1043	65	3912	0.9	0.64	**2.61**
9/29/17	10:06	1278.7		2.85				17	101	31.2	40.8	-0.6	41	1132	19	1151	68	4061	0.9	0.69	**2.73**
9/29/17	10:40	1288.1		2.9				34	135	31.8	40.2	-0.6	41	2171	19	2190	64	3865	0.9	0.67	**2.64**
9/29/17	11:41	1306.8		2.9				61	196	31.8	40.2	0	40	4320	0	4320	71	4249	0.9	0.73	**2.92**
9/29/17	12:54	1329.5		2.95				73	269	32.4	39.6	-0.6	40	5244	19	5263	72	4325	0.9	0.77	**2.99**

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: Bridge Development Rialto 11805.001

Exploration #/Location	LB-2
Depth Boring drilled to (ft):	10
Tested by:	BER
USCS Soil Type in test zone:	GW
Weather (start to finish)	sunny
Liquid Used/pH:	Faucet Water
Measured boring diameter	10 in. — 5 Well Radius, "r"
Approx Depth to GW below GS	100 ft
Well Prep:	drilled to 10 feet, place perforated 3" pipe (with filter fabric) within auger and pulled out

Initial estimated Depth to Water Surface (in.):	58.772
Average depth of water in well, "h" (in.):	61.228
approx. h/r:	12.2
Tu (Fig. 8):	95.1 ft
Tu>3h?:	yes, OK

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	10. ft		120	
Pilot Tube stickup (+ is above ground)		4.5 in.	4.5	
Depth to top of float assembly from top of pilot tube			0	-4.5 Depth below GS (in.)
Float Assembly ID				
Float assembly Extension length (in.)				

Flow Meter:

Meter ID: 50

Meter Unit: Gallons

0.05 gallons/pulse

Field Data — **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube): ft	Depth to WL in Boring: in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3): from supply	Vol Change (in.^3): from Δh	Vol Change (in.^3): Total	Flow (in^3/min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 9/29/2017	Start time: 8:44	Gallons																			
9/29/17	8:44			5.1					0	56.7	63.3										
9/29/17	8:57			3.9				13	13	42.3	77.7	14.4	71	0	-452	-452	-35	-2086	0.9	-0.12	**-0.84**
9/29/17	9:08			3.69				11	24	39.8	80.2	2.52	79	0	-79	-79	-7	-431	0.9	-0.02	**-0.16**
9/29/17	9:19			3.38				11	35	36.1	83.9	3.72	82	0	-117	-117	-11	-637	0.9	-0.03	**-0.22**
9/29/17	9:37			5.92				18	53	66.5	53.5	-30.48	69	0	956	956	53	3188	0.9	0.39	**1.31**
9/29/17	9:51			6.11				14	67	68.8	51.2	-2.28	52	0	72	72	5	307	0.9	0.04	**0.16**
9/29/17	9:58			6.2				7	74	69.9	50.1	-1.08	51	0	34	34	5	290	0.9	0.04	**0.16**
9/29/17	10:03	1813.6		4.08				5	79	44.5	75.5	25.44	63	0	-798	-798	-160	-9580	0.9	-0.57	**-4.30**
9/29/17	10:08	1830.8		3.4				5	84	36.3	83.7	8.16	80	3973	-256	3717	743	44606	0.9	2.36	**15.94**
9/29/17	10:17	1848.8		4.55				9	93	50.1	69.9	-13.8	77	4158	433	4591	510	30607	0.9	2.29	**11.33**
9/29/17	10:34	1880.5		4.38			2m57s/5gal	17	110	48.1	71.9	2.04	71	7323	-64	7259	427	25619	0.9	1.75	**10.24**
9/29/17	11:04	1938.9		4.3				30	140	47.1	72.9	0.96	72	13490	-30	13460	449	26921	0.9	1.81	**10.54**
9/29/17	11:24	1959.2		6.3			6m16s/5gal	20	160	71.1	48.9	-24	61	4689	753	5442	272	16327	0.9	2.28	**7.56**
9/29/17	12:01	2012.9		4.55				37	197	50.1	69.9	21	59	12405	-659	11746	317	19047	0.9	1.29	**9.03**
9/29/17	12:20	2031.8		6.35				19	216	71.7	48.3	-21.6	59	4366	678	5044	265	15928	0.9	2.25	**7.59**
9/29/17	12:52	2073.5		5.55				32	248	62.1	57.9	9.6	53	9633	-301	9331	292	17496	0.9	1.64	**9.23**

template updated: 3/7/16

Results of Well Permeameter, from USBR 7300-89 Method.

Leighton

Project: Bridge Development Rialto 11805.001

Exploration #/Location	LB-3
Depth Boring drilled to (ft):	10
Tested by	BER
USCS Soil Type in test zone	GW
Weather (start to finish)	sunny
Liquid Used/pH:	Faucet Water
Measured boring diameter	10 in. — 5 Well Radius, "r"
Approx Depth to GW below GS	100 ft
Well Prep	drilled to 10 feet, place perforated 3" pipe (with filter fabric) within auger and pulled out

Initial estimated Depth to Water Surface (in.):	92.743
Average depth of water in well, "h" (in.):	27.257
approx. h/r:	5.5
Tu (Fig. 8):	92.3 ft
Tu>3h?:	yes, OK

	ft	in.	Total (in.)	
Depth to Bot of well (or top of soil over Bentonite)	10. ft		120	
Pilot Tube stickup (+ is above ground)		7. in.	7	
Depth to top of float assembly from top of pilot tube			0	-7 Depth below GS (in.)
Float Assembly ID				
Float assembly Extension length (in.)				

Flow Meter:

Meter ID

Meter Unit Gallons

0.05 gallons/pulse

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (cu-ft or gal)	Interval Pulse Count	Depth to WL in Boring (measured from top of pilot tube) ft	in.	Water Temp (deg F)	Comments	Δt (min)	Total Elapsed Time (min.)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:	Gallons																			
9/29/2017	8:42																				
9/29/17	8:42			5.2					0	55.4	64.6										
9/29/17	9:00			4.55				18	18	47.6	72.4	7.8	69	0	-245	-245	-14	-816	0.9	-0.05	**-0.34**
9/29/17	9:11			3.73				11	29	37.8	82.2	9.84	77	0	-309	-309	-28	-1684	0.9	-0.09	**-0.62**
9/29/17	9:21			3.13				10	39	30.6	89.4	7.2	86	0	-226	-226	-23	-1356	0.9	-0.06	**-0.45**
9/29/17	9:35			4				14	53	41.0	79.0	-10.44	84	0	328	328	23	1404	0.9	0.09	**0.48**
9/29/17	9:51			3.9				16	69	39.8	80.2	1.2	80	0	-38	-38	-2	-141	0.9	-0.01	**-0.05**
9/29/17	10:04			5.03				13	82	53.4	66.6	-13.56	73	0	426	426	33	1964	0.9	0.16	**0.76**
9/29/17	10:18			5.05				14	96	53.6	66.4	-0.24	67	0	8	8	1	32	0.9	0.00	**0.01**
9/29/17	10:28			5.5			2m43s/5gal	10	106	59.0	61.0	-5.4	64	0	169	169	17	1017	0.9	0.09	**0.45**
9/29/17	10:22			5.5				-6	100	59.0	61.0	0	61	0	0	0	0	0	0.9	0.00	**0.00**
9/29/17	11:35			5.5			2m37s/5gal	73	173	59.0	61.0	0	61	0	0	0	0	0	0.9	0.00	**0.00**
9/29/17	12:22			5.5			2m32s/5gal	47	220	59.0	61.0	0	61	0	0	0	0	0	0.9	0.00	**0.00**
9/29/17	12:32			5.7			turned off	10	230	61.4	58.6	-2.4	60	0	75	75	8	452	0.9	0.04	**0.21**
9/29/17	12:35			8.75				3	233	98.0	22.0	-36.6	40	0	1149	1149	383	22970	0.9	13.23	**15.75**
9/29/17	12:36			9.33				1	234	105.0	15.0	-6.96	19	0	218	218	218	13104	0.9	10.66	**18.29**
9/29/17	12:37			9.49				1	235	106.9	13.1	-1.92	14	0	60	60	60	3615	0.9	3.24	**6.40**
9/29/17	12:38			9.6				1	236	108.2	11.8	-1.32	12	0	41	41	41	2485	0.9	2.54	**4.87**
9/29/17	12:39			9.69				1	237	109.3	10.7	-1.08	11	0	34	34	34	2033	0.9	2.34	**4.34**
9/29/17	12:42			9.87				3	240	111.4	8.6	-2.16	10	0	68	68	23	1356	0.9	2.18	**3.28**
9/29/17	12:44			9.96				2	242	112.5	7.5	-1.08	8	0	34	34	17	1017	0.9	1.86	**2.84**
9/29/17	12:46			10.01				2	244	113.1	6.9	-0.6	7	0	19	19	9	565	0.9	1.11	**1.71**
9/29/17	12:51			10.15				5	249	114.8	5.2	-1.68	6	0	53	53	11	633	0.9	1.89	**2.17**

Open Pit Percolation to Infiltration Calculation Sheet

Based on San Bernardino County WQMP Appendix D, dated May 19, 2011

Project Name: Bridge Rialto Due Dilligence
Project No.: 11805.001
Prepared by: B. Rodriguez
Date Prepared: 11/3/2017

Test Hole ID:	LB-3	
Test Hole Width:	9	inches
Test Hole Length:	9	inches
Test Hole Depth:	120	inches
Equivalent Radius	5.0777	inches

Start Time	Stop Time	Δt-Time Interval (min.)	D_0-Initial Depth (in.)	Initial Measure from fixed point (in.)	Final Measure from fixed	Initial Relative Depth Increase	Final Relative Depth	D_f-Initial Depth (in.)	ΔD-Change in Water Level (in.)	H_0 (in.)	H_f (in.)	ΔH (in.)	H_{avg} (in.)	I_t (in./hr)	Total Surface Area (ft^2)	Water Volume Change (ft^3)	Water Volume Change (gallons)	Percolation Rate ($gal/ft^2/day$)	Water Volume Change (in^3)	Total Surface Area (in^2)	I_t (in./hr)
12:32	12:35	3	108.4	68.4	105.0	0.0	36.6	145.0	36.6	11.6	-25.0	36.6	-6.7	13.5	30.6	1.72	12.83	201.6	2964.6	4401.0	13.47
12:35	12:36	1	145.0	105.0	112.0	36.6	43.6	151.9	7.0	-25.0	-31.9	7.0	-28.5	7.7	30.6	0.33	2.44	115.0	563.76	4401.0	7.69
12:36	12:37	1	151.9	112.0	113.9	43.6	45.5	153.8	1.9	-31.9	-33.8	1.9	-32.9	2.1	30.6	0.09	0.66	31.1	152.28	4401.0	2.08
12:37	12:38	1	153.8	113.9	115.2	45.5	46.8	155.1	1.3	-33.8	-35.1	1.3	-34.5	1.4	30.6	0.06	0.46	21.5	105.3	4401.0	1.44
12:38	12:39	1	155.1	115.2	116.3	46.8	47.9	156.2	1.1	-35.1	-36.2	1.1	-35.7	1.2	30.6	0.05	0.38	17.8	87.48	4401.0	1.19
12:39	12:42	3	156.2	116.3	118.4	47.9	50.0	158.3	2.1	-36.2	-38.3	2.1	-37.3	0.8	30.6	0.10	0.75	11.8	173.34	4401.0	0.79
12:42	12:44	2	158.3	118.4	119.5	50.0	51.1	159.5	1.1	-38.3	-39.5	1.1	-38.9	0.6	30.6	0.05	0.39	9.3	90.72	4401.0	0.62
12:44	12:46	2	159.5	119.5	120.1	51.1	51.7	160.1	0.6	-39.5	-40.1	0.6	-39.8	0.3	30.6	0.03	0.22	5.1	50.22	4401.0	0.34
12:46	12:51	5	160.1	120.1	121.8	51.7	53.4	161.8	1.7	-40.1	-41.8	1.7	-40.9	0.4	30.6	0.08	0.60	5.6	137.7	4401.0	0.38
						-68.4	-68.4	0.0	0.0	120.0	120.0	0.0	120.0	######	30.6	0.00	0.00	#DIV/0!	0	4401.0	#DIV/0!
						-68.4	-68.4	0.0	0.0	120.0	120.0	0.0	120.0	######	30.6	0.00	0.00	#DIV/0!	0	4401.0	#DIV/0!

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name:	Bridge Development/Rialto	Tested By:	R. Manning Date: 10/16/17
Project No.:	11805.001	Checked By:	J. Ward Date: 10/20/17
Boring No.:	LB-2	Depth (feet):	8.5
Sample No.:	R-1		
Soil Identification:	Light yellowish brown silty sand with gravel (SM)g		

		Moisture Content of Total Air - Dry Soil	
Container No.:	50	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	369.6	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	62.5	Wt. of Container No._____ (g)	1.0
Dry Wt. of Soil (g)	307.1	Moisture Content (%)	0.0

After Wet Sieve	Container No.	50
	Wt. of Dry Soil + Container (g)	330.3
	Wt. of Container (g)	62.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	267.8

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0		
3/4"	19.0	0.0	100.0
1/2"	12.5	39.6	87.1
3/8"	9.5	64.5	79.0
#4	4.75	97.8	68.2
#8	2.36	125.5	59.1
#16	1.18	151.4	50.7
#30	0.600	178.4	41.9
#50	0.300	209.8	31.7
#100	0.150	241.6	21.3
#200	0.075	266.9	13.1
PAN			

GRAVEL: **32 %**

SAND: **55 %**

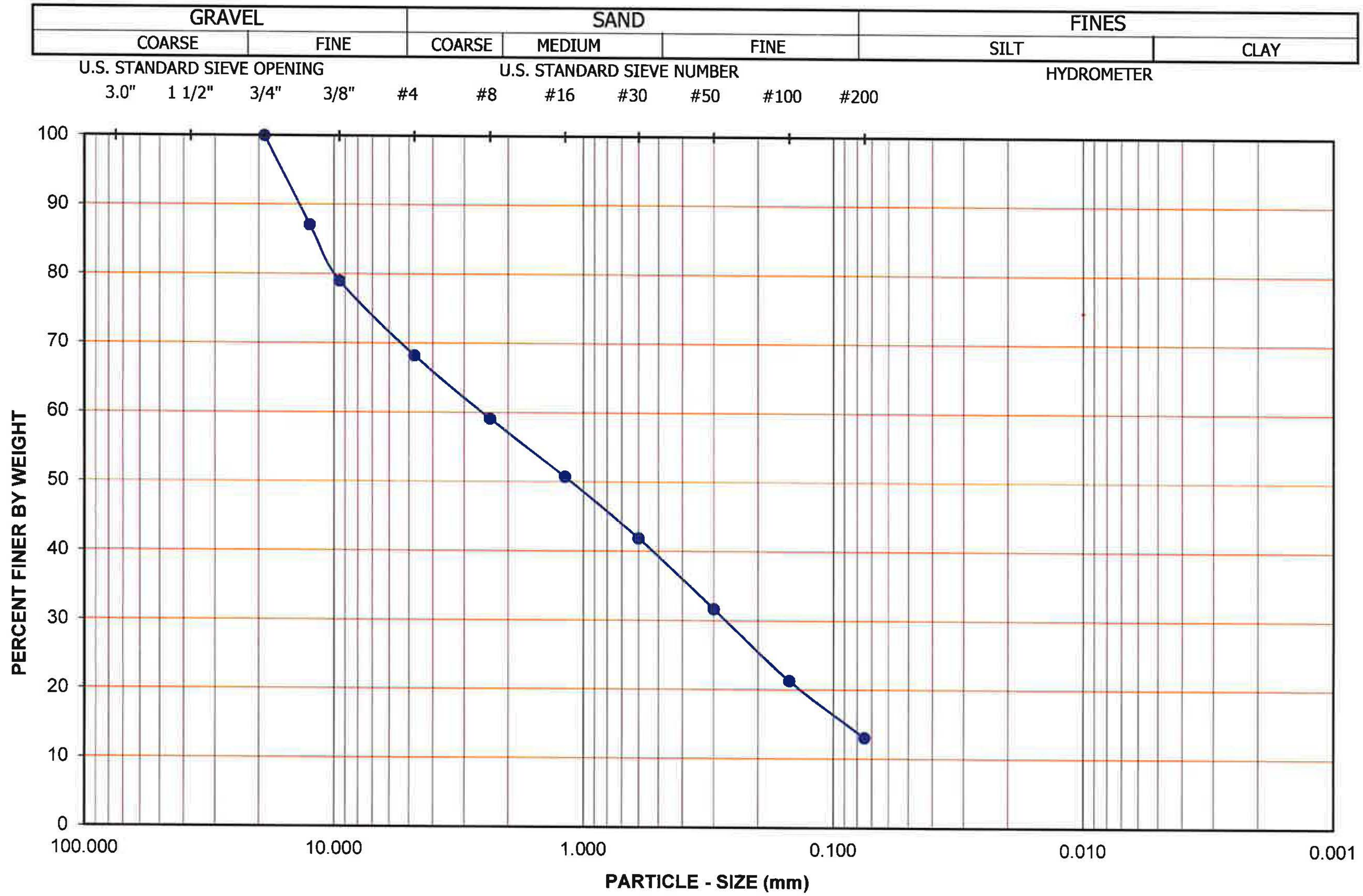
FINES: **13 %**

GROUP SYMBOL: **(SM)g**

$Cu = D60/D10 =$ ______

$Cc = (D30)^2/(D60*D10) =$ ______

Remarks:



Project Name: Bridge Development/Rialto

Project No.: 11805.001

Boring No.: LB-2 Sample No.: R-1

Depth (feet): 8.5 Soil Type : (SM)g

Soil Identification: Light yellowish brown silty sand with gravel (SM)g

GR:SA:FI : (%) 32 : 55 : 13

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Oct-17

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Leighton

Project Name:	Bridge Development/Rialto	Tested By:	R. Manning	Date:	10/09/17
Project No.:	11805.001	Input By:	J. Ward	Date:	10/20/17
Boring No.:	LB-3	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Dark olive gray poorly-graded sand with silt and gravel (SP-SM)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize material

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	19.7
#3/8	
#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft^3) 0.07450

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	7348	7587	7620			
Weight of Mold (g)	2660	2660	2660			
Net Weight of Soil (g)	4688	4927	4960			
Wet Weight of Soil + Cont. (g)	989.3	1045.6	1174.2			
Dry Weight of Soil + Cont. (g)	960.2	991.1	1087.0			
Weight of Container (g)	77.6	76.2	78.2			
Moisture Content (%)	3.30	5.96	8.64			
Wet Density (pcf)	138.7	145.8	146.8			
Dry Density (pcf)	134.3	137.6	135.1			

Maximum Dry Density (pcf)	**137.5**	**Optimum Moisture Content (%)**	**6.0**
Corrected Dry Density (pcf)	**142.5**	**Corrected Moisture Content (%)**	**5.0**

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

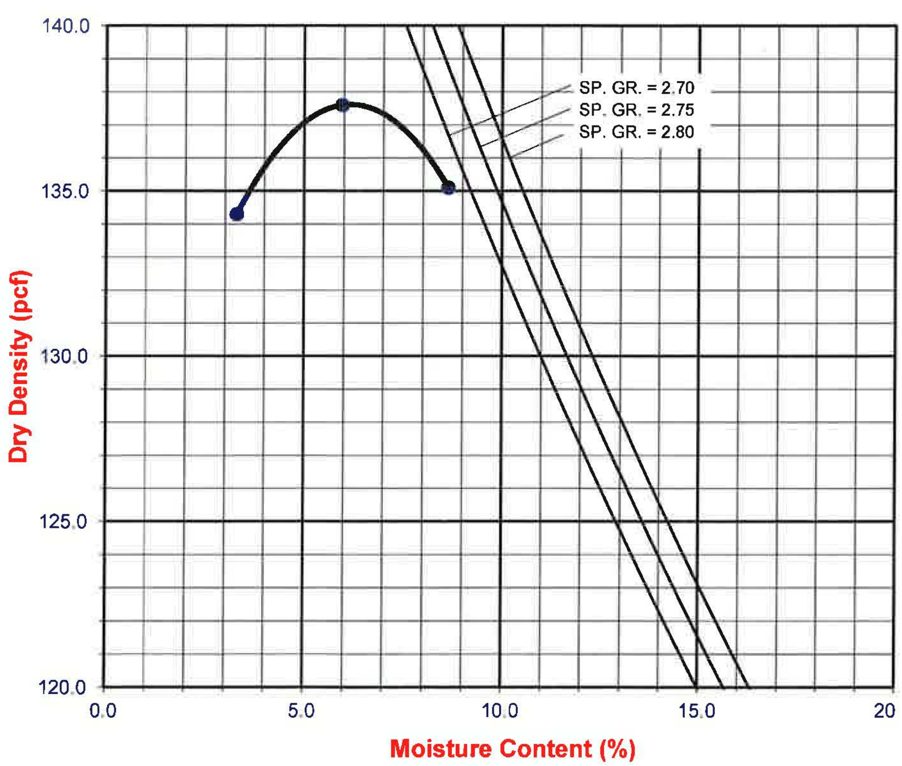
☒ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Bridge Development/Rialto	Tested By :	GB/ACS	Date:	10/10/17
Project No. :	11805.001	Data Input By:	J. Ward	Date:	10/20/17

Boring No.	LB-3			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Dark olive gray (SP-SM)g			
Wet Weight of Soil + Container (g)	132.98			
Dry Weight of Soil + Container (g)	132.77			
Weight of Container (g)	59.16			
Moisture Content (%)	0.29			
Weight of Soaked Soil (g)	100.22			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	92			
Crucible No.	16			
Furnace Temperature (°C)	860			
Time In / Time Out	11:20/12:05			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.0932			
Wt. of Crucible (g)	25.0915			
Wt. of Residue (g) (A)	0.0017			
PPM of Sulfate (A) x 41150	69.95			
PPM of Sulfate, Dry Weight Basis	**70**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	1.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	320			
PPM of Chloride, Dry Wt. Basis	**321**			

pH TEST, DOT California Test 643

pH Value	**7.16**			
Temperature °C	20.6			

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Bridge Development/Rialto	Tested By :	A. Santos Date: 10/10/17
Project No. :	11805.001	Data Input By:	J. Ward Date: 10/20/17
Boring No.:	LB-3	Depth (ft.) :	0-5
Sample No. :	B-1		

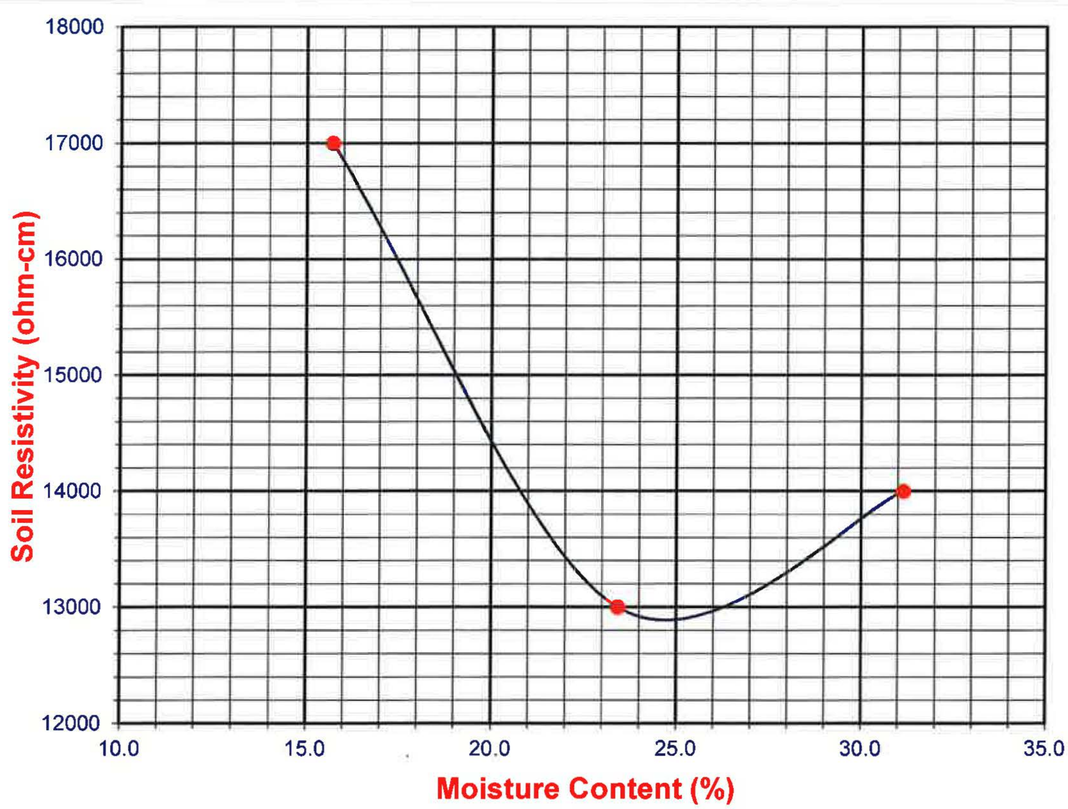
Soil Identification:* Dark olive gray (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.71	17000	17000
2	30	23.43	13000	13000
3	40	31.14	14000	14000
4				
5				

Moisture Content (%) (MCi)	0.29
Wet Wt. of Soil + Cont. (g)	132.98
Dry Wt. of Soil + Cont. (g)	132.77
Wt. of Container (g)	59.16
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
12900	**24.7**	**70**	**321**	**7.16**	**20.6**



USGS Design Maps Summary Report

User–Specified Input

Report Title Bridge Rialto
Wed November 1, 2017 16:50:15 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.14102°N, 117.40649°W

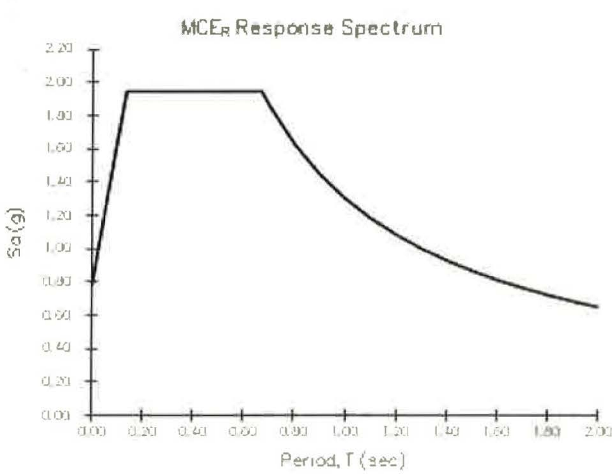
Site Soil Classification Site Class D – "Stiff Soil"

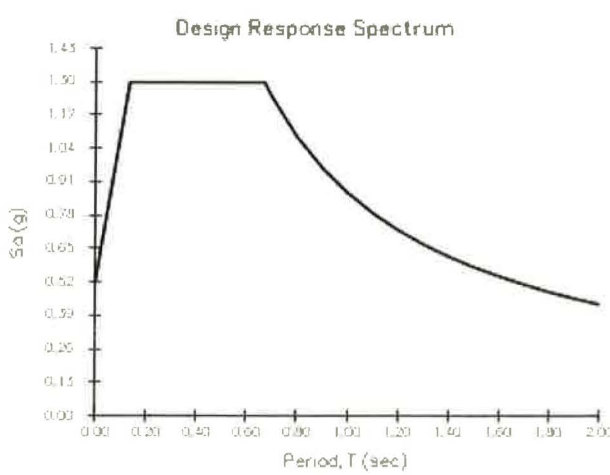
Risk Category I/II/III

USGS–Provided Output

$S_S =$	1.946 g	$S_{MS} =$	1.946 g	$S_{DS} =$	1.297 g
$S_1 =$	0.867 g	$S_{M1} =$	1.300 g	$S_{D1} =$	0.867 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.14102°N, 117.40649°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.946$ g

From Figure 22-2 [2] $S_1 = 0.867$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_s = 1.946 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.867 g, F_v = 1.500

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 1.946 = 1.946$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.500 \times 0.867 = 1.300$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

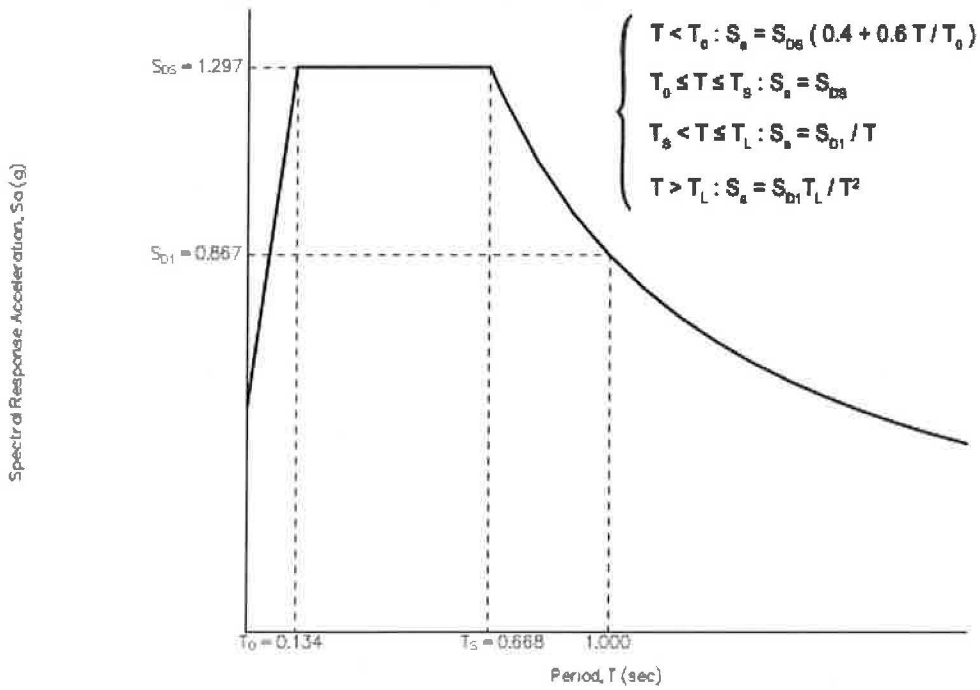
Equation (11.4-3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.946 = 1.297$ g

Equation (11.4-4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.300 = 0.867$ g

Section 11.4.5 — Design Response Spectrum

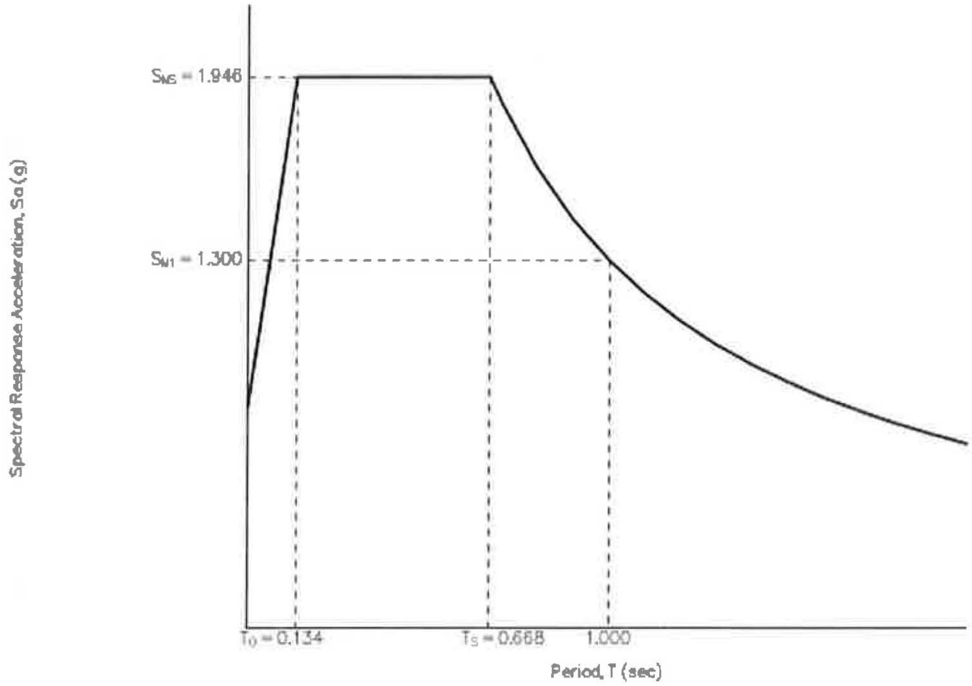
From Figure 22-12 [3] $T_L = 12$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.757

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.757 = 0.757$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.757 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.048

From Figure 22-18 [6] C_{R1} = 1.005

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 1.297 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.867 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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*    E Q F A U L T      *
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*    Version 3.00       *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 11805.001

DATE: 11-01-2017

JOB NAME: Bridge Rialto

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 34.1410
SITE LONGITUDE: 117.4065

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

TEST.OUT

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN JACINTO-SAN BERNARDINO	0.7(1.1)	6.7	0.533	X
CUCAMONGA	3.4(5.4)	7.0	0.529	X
SAN ANDREAS - San Bernardino	6.7(10.8)	7.3	0.336	IX
SAN ANDREAS - Southern	6.7(10.8)	7.4	0.345	IX
CLEGHORN	10.1(16.2)	6.5	0.192	VIII
SAN JACINTO-SAN JACINTO VALLEY	12.9(20.8)	6.9	0.190	VIII
NORTH FRONTAL FAULT ZONE (West)	13.5(21.7)	7.0	0.247	IX
SAN ANDREAS - Mojave	13.7(22.0)	7.1	0.200	VIII
SAN ANDREAS - 1857 Rupture	13.7(22.0)	7.8	0.267	IX
SAN JOSE	16.3(26.3)	6.5	0.158	VIII
SIERRA MADRE	18.3(29.5)	7.0	0.188	VIII
CHINO-CENTRAL AVE. (Elsinore)	20.1(32.4)	6.7	0.144	VIII
WHITTIER	23.8(38.3)	6.8	0.100	VII
ELSINORE-GLEN IVY	23.8(38.3)	6.8	0.100	VII
CLAMSHELL-SAWPIT	26.0(41.9)	6.5	0.094	VII
ELYSIAN PARK THRUST	29.1(46.9)	6.7	0.095	VII
HELENDALE - S. LOCKHARDT	33.5(53.9)	7.1	0.083	VII
RAYMOND	34.3(55.2)	6.5	0.067	VI
ELSINORE-TEMECULA	34.5(55.6)	6.8	0.064	VI
NORTH FRONTAL FAULT ZONE (East)	35.5(57.1)	6.7	0.074	VII
SAN JACINTO-ANZA	39.4(63.4)	7.2	0.073	VII
PINTO MOUNTAIN	39.5(63.6)	7.0	0.063	VI
VERDUGO	39.6(63.7)	6.7	0.065	VI
COMPTON THRUST	40.7(65.5)	6.8	0.067	VI
LENWOOD-LOCKHART-OLD WOMAN SPRGS	45.9(73.8)	7.3	0.065	VI
NEWPORT-INGLEWOOD (L.A.Basin)	47.0(75.6)	6.9	0.047	VI
HOLLYWOOD	47.1(75.8)	6.4	0.040	V
NEWPORT-INGLEWOOD (Offshore)	47.8(77.0)	6.9	0.046	VI
JOHNSON VALLEY (Northern)	49.5(79.6)	6.7	0.037	V
SAN GABRIEL	51.4(82.8)	7.0	0.045	VI
SIERRA MADRE (San Fernando)	51.6(83.1)	6.7	0.045	VI
LANDERS	54.6(87.9)	7.3	0.052	VI
SAN ANDREAS - Coachella	55.7(89.6)	7.1	0.044	VI
PALOS VERDES	56.2(90.4)	7.1	0.043	VI
EMERSON So. - COPPER MTN.	56.5(91.0)	6.9	0.037	V
BURNT MTN.	57.3(92.2)	6.4	0.024	IV
ELSINORE-JULIAN	57.4(92.3)	7.1	0.042	VI
SANTA MONICA	57.5(92.5)	6.6	0.036	V
GRAVEL HILLS - HARPER LAKE	57.6(92.7)	6.9	0.036	V
EUREKA PEAK	57.9(93.2)	6.4	0.023	IV

TEST.OUT

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
NORTHRIDGE (E. Oak Ridge)	58.1(93.5)	6.9	0.045	VI
CALICO - HIDALGO	62.2(100.1)	7.1	0.038	V
SANTA SUSANA	63.5(102.2)	6.6	0.031	V
MALIBU COAST	64.7(104.2)	6.7	0.033	V
BLACKWATER	64.9(104.5)	6.9	0.030	V
CORONADO BANK	67.2(108.2)	7.4	0.043	VI
PISGAH-BULLION MTN.-MESQUITE LK	67.9(109.3)	7.1	0.033	V
HOLSER	68.0(109.5)	6.5	0.026	V
SAN JACINTO-COYOTE CREEK	69.8(112.3)	6.8	0.025	V
ROSE CANYON	69.8(112.4)	6.9	0.027	V
SAN ANDREAS - Carrizo	73.9(118.9)	7.2	0.032	V
ANACAPA-DUME	74.4(119.8)	7.3	0.044	VI
OAK RIDGE (Onshore)	77.2(124.3)	6.9	0.030	V
SAN CAYETANO	80.2(129.0)	6.8	0.026	V
SIMI-SANTA ROSA	80.3(129.2)	6.7	0.024	V
EARTHQUAKE VALLEY	81.3(130.9)	6.5	0.015	IV
GARLOCK (West)	83.4(134.2)	7.1	0.025	V
GARLOCK (East)	86.6(139.3)	7.3	0.028	V
SANTA YNEZ (East)	90.6(145.8)	7.0	0.020	IV
SAN JACINTO - BORREGO	95.1(153.0)	6.6	0.013	III
PLEITO THRUST	96.5(155.3)	7.2	0.028	V
WHITE WOLF	98.8(159.0)	7.2	0.027	V
So. SIERRA NEVADA	100.0(160.9)	7.1	0.025	V

-END OF SEARCH- 63 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JACINTO-SAN BERNARDINO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.7 MILES (1.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5335 g