Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED BIKE PATH PROJECT

BEACH CITIES HEALTH DISTRICT
514 NORTH PROSPECT AVENUE, SUITE 3
REDONDO BEACH, CALIFORNIA 90277

CONVERSE PROJECT NO. 20-31-273-01

Prepared For:

PAUL MURDOCH ARCHITECTS
Mr. Paul Murdoch, AIA, LEED AP
6310 San Vicente Boulevard
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Presented By:

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September 25, 2020

Converse Consultants
Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

September 25, 2020

Mr. Paul Murdoch, AIA, LEED AP
President
Paul Murdoch Architects
6310 San Vicente Boulevard
Los Angeles, California 90048

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Proposed Bike Path Project
Beach Cities Health District
514 North Prospect Avenue, Suite 3
Redondo Beach, California 90277
Converse Project No. 20-31-273-01

Dear Mr. Murdoch:

Converse Consultants (Converse) has prepared this report to provide geotechnical recommendations for the proposed Beach Cities Health District Bike Path in Redondo Beach and Torrance, California. This report has been prepared in accordance with our proposal dated August 11, 2020.

Based on the results of our study, it is our opinion that the proposed project is feasible from a geotechnical standpoint provided that the findings, conclusions and recommendations presented in this report are incorporated in the preparation of the final design plans, specifications and construction of the project.

We appreciate this opportunity to be of service to Beach Cities Health District. If you have any questions regarding this report, please contact us at (626) 930-1200.

Sincerely,

CONVERSE CONSULTANTS

Sivathasan

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President / Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

SKS /MBS/BA:jjl

PROFESSIONAL CERTIFICATION

This report has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

Babak Abbasi, PhD, EIT
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1.0 INTRODUCTION

This report contains the results of our geotechnical investigation, laboratory testing, and recommendations for the Proposed Bike Path Project.

The purpose of the study was to evaluate the subsurface soil conditions and to collect representative soil samples and provide geotechnical recommendations and design recommendations for the design and construction of the proposed project.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Site Description

The Beach Cities Health District is planning a new Bike Path Project along the eastern side of their site. The proposed Bike Path Project is located along the eastern slope area along Diamond Lane in the City of Redondo Beach and Towers Street and Flagler Lane Street in City of Torrance. The proposed Bike Path Project runs along the common boundary line between the two cities of Redondo Beach and Torrance.

The proposed Bike Path Project improvements will extend approximately 1180 feet from North Prospect Avenue on the south end, along Diamond Street, Tower Street cul-de-sac and Flagler Lane to Beryl Street on the north end. The bike path improvements will be made along the west side of Diamond Street, along the existing 8-foot-wide mid-slope sidewalk to the Towers Street cul-de-sac, and then along Flagler Lane. The project site is shown on Drawing No. 1, *Site Plan and Approximate Location of Borings*.

2.2 Project Description

The bike path will be approximately 14 feet wide along Diamond Street and the mid-slope sidewalk. Retaining walls up to 6 feet high are planned along the existing slopes that range in height from approximately 9 to 38 feet in height. The existing slope gradients are reported to range from approximately 2:1 (H:V) to 1.5:1 (H:V) slope gradients. The central portion of the existing sidewalk slope descends to residential homes along Tomlee Avenue and Towers Street. The slope area is reported to be underlain with older sand dune (eolian) deposits.

3.0 SCOPE OF WORK

The scope of Converse's investigation included the tasks described in the following sections.

SITE PLAN AND APPROXIMATE LOCATION OF BORINGS

BH-1 HAND AUGER BOREHOLES

0 100 200

SCALE IN FEET
SCALE: 1"=200'

Converse Consultants

PMA-Beach Cities Health District Bike Path
Diamond Lane, Redondo Beach, CA 90277 and
Flager Lane & Towers Street, Torrance, CA 90503

Project No.	Drawing No.
20-31-273-01	1

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3.1 Site Reconnaissance

Our field exploration included a site reconnaissance by a member of the Converse staff on August 18, 2020. The purpose of the site reconnaissance was to observe surface conditions and to stake/mark the boring locations in the field so that hand auger boring access to all the locations is available.

3.2 Subsurface Exploration

Three (3) exploratory borings (BH-1, BH-2, and BH-3) were advanced within the project site on August 24, 2020. All borings were drilled using a 4-inch diameter hand auger. The boring BH-1 was drilled to an explored depth of 12 feet below the existing ground surface (bgs), the boring BH-2 was drilled to an explored depth of 9 feet bgs, and the boring BH-3 was drilled to an explored depth of 7 bgs. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. California Modified Sampler (ring samples), and bulk soil samples were obtained for laboratory testing. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*. The approximate locations of the exploratory borings are shown in Drawing No. 1, *Site Plan and Approximate Location of Borings.*

3.3 Laboratory Testing

Representative soil samples obtained during the subsurface explorations were tested in our laboratory to evaluate their engineering properties. Laboratory testing included the following.

- In situ moisture contents and dry densities (ASTM D2216)
- Soil corrosivity tests (Caltrans 643, 422, 417, and 532)
- Resistance R-Value (ASTM D2844)
- Grain-Size Analysis (ASTM D6913)
- Laboratory maximum density (ASTM D1557)
- Direct shear (ASTM D3080)
- Swell/Collapse (ASTM D4546)

3.4 Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated with respect to the planned construction. This report was prepared to provide the findings, conclusions and recommendations developed during our study and evaluation.

4.0 SUBSURFACE CONDITIONS

The various elements of the subsurface condition are presented below.

4.1 Subsurface Profile

The earth materials encountered during our investigation consist of alluvium to a maximum depth of 12 feet bgs. Deeper artificial fill may exist at the site. Fill material was not encountered at the boring locations. The alluvial soil deposits mainly consisted of sand, with few gravel, and trace of silt.

4.2 Subsurface Variations

Based on results of the subsurface exploration and our experience, variations in the continuity and nature of subsurface conditions should be anticipated. Due to uncertainties involved in the nature and depositional characteristics of the earth materials, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If, during construction, subsurface conditions different from those presented in this report are encountered, this office should be notified immediately so that recommendations can be revised and modified as needed.

4.3 Excavatability

Based on our exploratory borings, the soil to the maximum depth explored is expected to be excavatable with conventional heavy-duty earthmoving equipment, such as excavators, bulldozers, and front loaders.

5.0 SUBSURFACE CONDITIONS

5.1 CBC Seismic Design Parameters

General seismic parameters based on the 2019 California Building Code and ASCE 7-16 with Supplement 1 are calculated using the ATC hazard, *Seismic Design by location* website application and the site coordinates (33.8529 degrees North Latitude, 118.3780 degrees West Longitude). The seismic parameters are presented below.

Table No. 1, CBC Seismic Design Parameters

Seismic Parameter	Value
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.876 g
Mapped 1-second Spectral Response Acceleration, S_1	0.673 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v^*	1.7

Seismic Parameter	Value
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.876 g
MCE 1-second period Spectral Response Acceleration, S_{M1}*	1.144 g
Design Spectral Response Acceleration for short period, S_{DS}	1.251 g
Design Spectral Response Acceleration for 1-second period, S_{D1}*	0.763 g

*ASCE 7-16 section 21.3, for the site-specific ground motion these values are used: F_v=2.5, S_{M1}=1.683, and S_{D1}=1.122, See Table No. 2

5.2 Site-Specific Response Spectra

A site-specific response spectrum was developed for the project for a Maximum Considered Earthquake (MCE), defined as a horizontal peak ground acceleration that has a 2 percent probability of being exceeded in 50 years (return period of approximately 2,475 years).

In accordance with ASCE 7-16, Section 21.2 the site-specific response spectra can be taken as the lesser of the probabilistic maximum rotated component of MCE ground motion and the 84th percentile of deterministic maximum rotated component of MCE ground motion response spectra. The design response spectra can be taken as 2/3 of site-specific MCE response spectra but should not be lower than 80 percent of CBC general response spectra. The risk coefficient C_R has been incorporated at each spectral response period for which the acceleration was computed in accordance with ASCE 7-16, Section 21.2.1.1.

The 2019 CBC mapped acceleration parameters are provided in the following table. These parameters were determined using the *ATC hazard by location Seismic Design Maps* website application, and in accordance with ASCE 7-16 Sections 11.4, 11.6, 11.8, 21.2, and 21.3.

Table No. 2, 2019 CBC Mapped Acceleration Parameters

Site Class	D	Seismic Design Category	D
S_s	1.876	**C_{RS}**	0.892
S_1	0.673	**C_{R1}**	0.891
F_a	1	**0.08 F_v/F_a**	0.200
F_v	2.5	**0.4 F_v/F_a**	1.000
S_{MS}	1.876	**T_0**	0.179
S_{M1}	1.683	**T_S**	0.897
S_{DS}	1.251	**T_L**	8
S_{D1}	1.122		

A site-specific response analysis, using faults within 200 kilometers of the sites, was developed using the computer program EZ-FRISK Version 8.06 (Fugro, 2019).

The weighted mean maximum-rotated horizontal spectral acceleration values were computed by multiplying the weighted mean geometric spectral values derived from four next-generation attenuation (NGA) West 2 ground motion attenuation models by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) with the scale factors provided in ASCE 7-16 Section 21.2. An average shear wave velocity at upper 30 meters of soil profile (V_{s30}) of 270 meters per second, depth to bedrock of with a shear wave velocity 1,000 meters per second at 150 meters below grade, and depth of bedrock where the shear wave velocity is 2,500 meters per second at 2,500 meters below grade were selected for EZ-Frisk Analysis.

The probabilistic response spectrum results and peak ground acceleration for each attenuation relationship are presented in the following table.

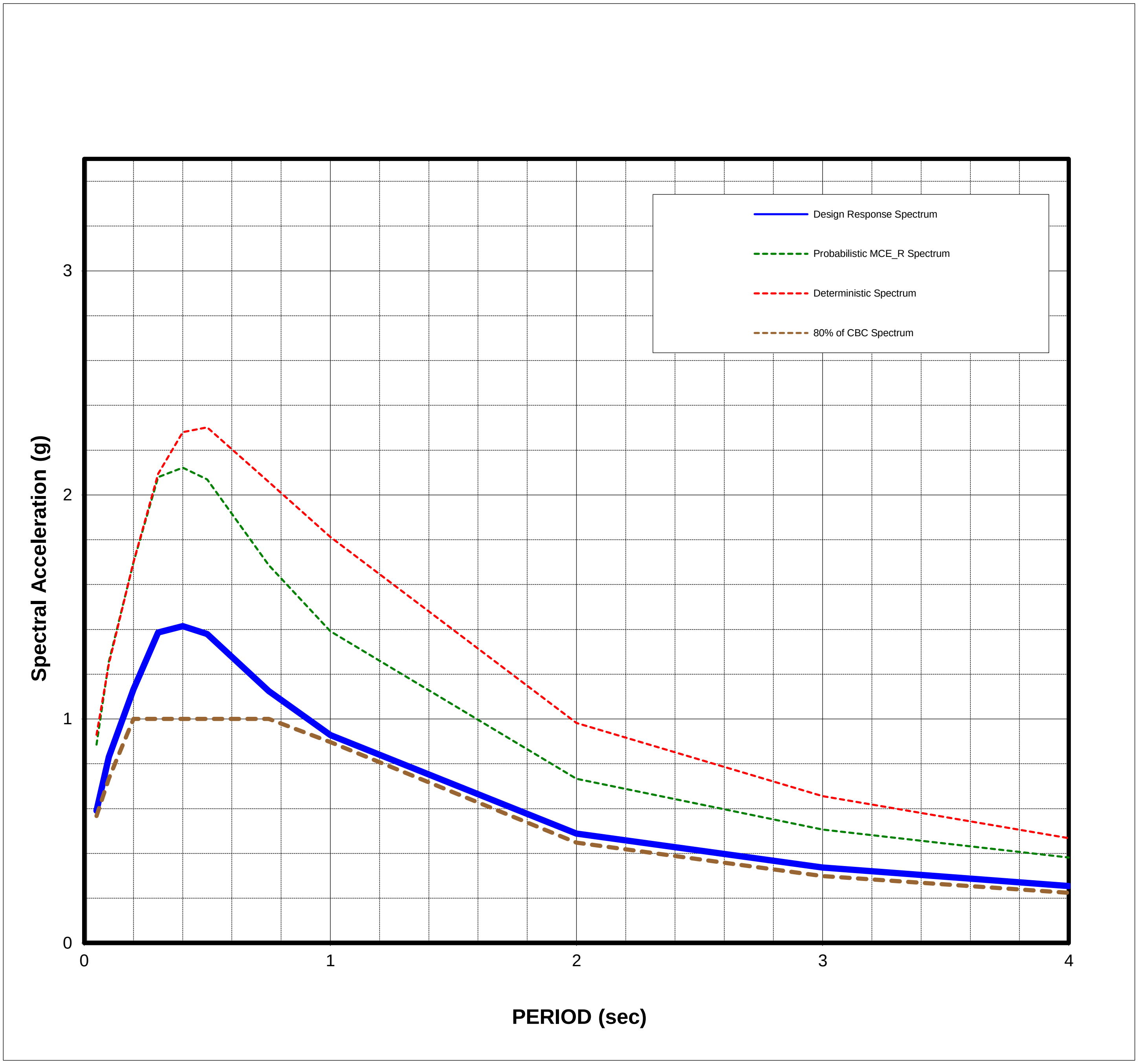
Table No. 3, Probabilistic Response Spectrum Data

Attenuation Relationship	Probabilistic Mean	Abrahamson et al. (2014)	Boore et al. (2014)	Campbell-Bozorgnia (2014)	Chiou-Youngs (2014)
Peak Ground Acceleration (g)	0.783	0.775	0.876	0.636	0.827
Spectral Period (sec)	**2% in 50yr Probabilistic Spectral Acceleration (g)**				
0.05	0.903	0.786	1.056	0.804	0.957
0.10	1.280	1.088	1.590	1.132	1.273
0.20	1.732	1.837	1.916	1.287	1.814
0.30	2.073	2.254	2.049	1.614	2.223
0.40	2.069	2.249	1.931	1.755	2.234
0.50	1.975	2.041	1.867	1.746	2.168
0.75	1.529	1.447	1.423	1.514	1.729
1.00	1.202	1.119	1.100	1.288	1.299
2.00	0.610	0.579	0.522	0.774	0.541
3.00	0.406	0.378	0.345	0.559	0.311
4.00	0.296	0.291	0.260	0.405	0.198
5.00	0.213	0.223	0.195	0.288	0.120

Deterministic response spectra parameters were determined using PEER spread sheet and presented in Table No. 5. Following fault parameters were used to calculate the spectrum.

- Palos Verdes Connected Fault, Mw–7.7, R_{RUP}–4.0 km, R_{JB}–4.0 km, Rx–4.0 km and dip angle are 90 degree. The Palos Verdes Connected is a strike-slip Fault with length of approximately 280 km. The fault line extends into the Pacific Ocean makes shore somewhere near the southwest point of the Redondo Beach-Torrance border.

Applicable response spectra data are presented in the table below and on Drawing No. 2, *Site Specific Design Response Spectrum.* These curves correspond to response values



Note: Calculated using EZFRISK program Risk Engineering, version 8.06

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

Proposed Bike Path Project

Los Angeles

For : Beach Cities Health District

Project Number:

20-31-273-01

Converse Consultants

Drawing No.

2

obtained from above attenuation relations for horizontal elastic single-degree-of-freedom systems with equivalent viscous damping of 5 percent of critical damping.

Table No. 4, Probabilistic MCE_R Spectral Acceleration (g)

Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g) Geometric Mean	Risk Coefficient C_R	Scale Factors for MCE_R	Probabilistic MCE_R Spectral Acceleration (g)
0.05	0.903	0.892	1.100	0.886
0.10	1.280	0.892	1.100	1.256
0.20	1.732	0.892	1.100	1.699
0.30	2.073	0.892	1.125	2.080
0.40	2.069	0.892	1.150	2.122
0.50	1.975	0.892	1.175	2.069
0.75	1.529	0.891	1.238	1.686
1.00	1.202	0.891	1.300	1.392
2.00	0.610	0.891	1.350	0.733
3.00	0.406	0.891	1.400	0.506
4.00	0.296	0.891	1.450	0.382
5.00	0.213	0.891	1.500	0.285

Table No. 5, Site-Specific Response Spectrum Data

Period (sec)	84th Percentile Deterministic Response Spectrum, (g) Geometric Mean	Scale Factors for MCE_R	84th Percentile Deterministic MCE Response Spectrum, (g)	Site Specific MCE_R Spectral Acceleration (g)	80% CBC Design Response Spectrum	Site Specific Design Spectral Acceleration (g)
0.05	0.845	1.100	0.930	0.886	0.568	0.591
0.10	1.133	1.100	1.246	1.246	0.735	0.831
0.20	1.546	1.100	1.700	1.699	1.001	1.133
0.30	1.861	1.125	2.094	2.080	1.001	1.387
0.40	1.983	1.150	2.280	2.122	1.001	1.415
0.50	1.959	1.175	2.302	2.069	1.001	1.379
0.75	1.663	1.238	2.058	1.686	1.001	1.124
1.00	1.394	1.300	1.812	1.392	0.897	0.928
2.00	0.727	1.350	0.982	0.733	0.449	0.489
3.00	0.469	1.400	0.656	0.506	0.299	0.338
4.00	0.323	1.450	0.468	0.382	0.224	0.255
5.00	0.236	1.500	0.353	0.285	0.179	0.190

The site-specific design response parameters are provided in the following table. These parameters were determined from Design Response Spectra presented in table above and following guidelines of ASCE Section 21.4.

Table No. 6, Site-Specific Seismic Design Parameters

Parameter	Value (5% Damping)	Lower Limit, 80% of CBC Design Spectra
Site-Specific 0.2-second period Spectral Response Acceleration, S_{MS}	1.910	1.501
Site-Specific1-second period Spectral Response Acceleration, S_{M1}	1.527	0.915
Site-Specific Design Spectral Response Acceleration for short period S_{DS}	1.273	1.001
Site-Specific Design Spectral Response Acceleration for 1-second period, S_{D1}	1.018	0.897

6.0 RETAINING WALL RECOMMENDATIONS

6.1 Shallow Foundations

6.1.1 Vertical Capacity

The proposed retaining wall can be supported by conventional shallow footings. We recommend footings be founded at least 18 inches below lowest adjacent final grade entirely into compacted fill or into native soil. A minimum footing width of 18 inches for continuous footings. The allowable bearing value for footings with above minimum sizes founded on compacted fill and competent native soils may be designed for a net bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loads. The net allowable bearing pressure can be increased by 250 psf for each additional foot of footing depth and by 250 psf for each additional foot of footing width up to a maximum value of 3,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

6.1.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with normal dead load forces. An allowable passive earth pressure of 200 psf per foot of depth up to a maximum of 2,000 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

6.1.3 Settlement

The static settlement of retaining walls supported on continuous footings founded on compacted fill and native soil will depend on the actual footing dimensions and the imposed vertical loads. Most of the footing settlement at the project site is expected to occur immediately after the application of the load. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30-foot span.

6.1.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

6.2 Lateral Earth Pressure

The proposed retaining wall is anticipated to be up to 6 feet in height. The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following fluid pressures are recommended for vertical walls with no hydrostatic pressure, no surcharge, and level backfill.

Table No. 7, Lateral Earth Pressures for Retaining Wall Design

Wall Type	Equivalent Fluid Pressure (pcf)
	Level Backfill
Cantilever Wall (Active pressure)	35 (Triangular Distribution)
Restrained Wall (At-rest pressure)	55 (Triangular Distribution)

The recommended lateral pressures assume that the walls are fully back-drained with granular, free-draining, non-expansive soil materials to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by free draining, uniformly graded, ¾ -inch washed, permeable aggregate material, and wrapped in filter fabric (Mirafi 140N or equivalent) and should extend to about 2 feet below the finished grade. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, Schedule 40 PVC or rigid ABS (SDR-35), or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. The subdrain should be connected to a suitable outlet point, surface drain or sump pump. Subterranean walls should be waterproofed to prevent moisture migration and moisture problems.

In addition, walls with inclined backfill should be designed based on section 373.1 (Equivalent Fluid Pressure, EFP) of the BOE Structural Design Manuel. Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively.

Cantilever retaining walls greater than 6 feet, as measured from the surface, should be designed to resist additional earth pressure caused by seismic ground shaking. A dynamic earth pressure of 20H (psf), based on an inverted triangular distribution, can be used for design of wall.

6.3 Sidewalk

Slabs-on-grade should have a minimum thickness of four (4) inches for support of normal pedestrian and bike live loads. Minimum reinforcement for slabs-on-grade should be No. 4 reinforcing bars, spaced at 18 inches on-center each way. The thickness and reinforcement of more heavily loaded slabs will be dependent upon the anticipated loads and should be designed by a structural engineer. A static modulus of subgrade reaction equal to 125 pounds per square inch per inch may be used in structural design of concrete slabs-on-grade.

It is critical that the exposed subgrade soils should not be allowed to desiccate prior to the slab pour. Care should be taken during concrete placement to avoid slab curling. Slabs should be designed and constructed as promulgated by the ACI and Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

6.4 Soil Corrosivity Evaluation

Converse retained the Environmental Geotechnology Laboratory, Inc., located in Arcadia, California, to test one (1) sample taken in the general area of the proposed structures. The tests included minimum resistivity, pH, soluble sulfates, and chloride content, with the results summarized on the following table:

Table No. 8, Soil Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) % by Weight	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-1	0-5.0	6.41	145	0.007	8,100

In accordance with the Caltrans Corrosive Guidelines (2012), the pH value and chloride content of the sample tested is in the “non-corrosive” range. However, the resistivity is in the “corrosive” range to ferrous metals.

Soluble sulfate concentrations tested for this project are less than 0.20 in the soil. Mitigation measures to protect concrete in contact with the soils should be anticipated.

Type I or II Portland Cement may be used for the construction of the foundations and slabs.

In general, conventional corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

The test results presented herein are considered preliminary. If advanced corrosivity study is desired by the design team, a corrosion engineer can be consulted for appropriate mitigation procedures and construction design.

6.5 Flexible Pavement

The flexible pavement structural section design recommendations were performed in accordance with the method contained in the *CALTRANS Highway Design Manual*, Chapter 630, without the factor of safety. No specific traffic study was performed to determine the Traffic Index (TI) for the proposed project; therefore a wide range of TI values were evaluated.

Due to various earth materials encountered at the site, flexible pavement structural section recommendations are prepared for both subgrade soils. We recommend that the project structural engineer consider the traffic loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 9, Flexible Pavement Structural Sections

Design R-value	Design TI	Asphalt Concrete (AC) Over Aggregate Base (AB) Structural Sections		Full AC Structural Section
		AC (inches)	AB (inches)	AC (inches)
58	4	***3.0***	***3.0***	***4.0***
	5	***3.0***	***3.0***	***4.0***
	6	***3.0***	***3.0***	***5.0***
	7	***3.0***	***4.5***	***6.5***
	8	***3.5***	***5.5***	***7.5***
	9	***4.0***	***6.5***	***8.5***
	10	***5.0***	***7.0***	***9.5***

Base material shall conform to requirements for Crushed Miscellaneous Base (CMB) or equivalent and should be placed in accordance with the requirements of the Standard Specifications for Public Works Construction (SSPWC, latest Edition). Asphaltic materials should conform to Section 203-1, "Paving Asphalt," of the Standard Specifications for Public Works Construction (SSPWC, latest Edition) and should be placed in accordance with Section 302-5, "Asphalt Concrete Pavement," of the SSPWC, 2012 edition.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

6.6 Rigid Pavement

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, Portland Cement Concrete Pavement (PCCP) for Light, Medium and Heavy Traffic Rigid Pavement. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 10, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
58	5.0	6.0
	6.0	6.5
	7.0	6.5
	8.0	7.0
	9.0	7.5
	10	7.5

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,750 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are 3.0 inches to 1.0 inch, respectively.

Transverse contraction joints should not be spaced more than 10 feet and should be cut to a depth of 1/4 the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

Prior to placement of concrete, at least the upper 12.0 inches of subgrade soils below rigid pavement sections should be compacted to at least ninety-five percent (95%) relative compaction as defined by the ASTM D 1557 standard test method.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into pavement base and/or subgrade.

6.7 Site Drainage

Adequate positive drainage should be provided away from the improvements to prevent ponding and to reduce percolation of water into backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the retaining wall with a minimum 5% slope gradient for at least 10 feet measured perpendicular to the face of the wall. Impervious surfaces within 10 feet of the foundation shall have a minimum 2 percent slope away from the retaining wall per 2019 CBC.

Planters and landscaped areas adjacent to the wall should be designed to minimize water infiltration into the subgrade soils.

7.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

7.1 General Evaluation

Based on our field exploration, laboratory testing, and analyses of subsurface conditions at the site, remedial grading will be required to prepare the sites for support of the proposed retaining wall that are constructed with conventional shallow footings. To reduce differential settlement, variations in the soil type, degree of compaction, and thickness of the compacted fill, the thickness of compacted fill placed underneath the footings should be kept uniform.

Site grading recommendations provided below are based on our experience with similar projects in the area and our evaluation of this investigation.

The site soils can be excavated utilizing conventional heavy-duty earth-moving equipment. The excavated site soils, free of vegetation, shrub and debris, may be placed as compacted fill in structural areas after proper processing. Rocks larger than three (3) inches in the largest dimension should not be placed as fill.

On-site clayey soils and with an expansion index exceeding 20 should not be re-used for compaction within 2 feet below the proposed foundations. Soils containing organic materials should not be used as structural fill. The extent of removal should be determined by the geotechnical representative based on soil observation during grading.

7.2 Over-Excavation

Prior to the start of construction of retaining wall, all loose soil, fill and soils disturbed during demolition if any should be removed to firm acceptable native material or compacted fill. In order to provide uniform support for the retaining wall foundation, the minimum depth of over-excavation should be 3 feet below the ground surface or 1 foot below bottom of proposed shallow foundations, whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils or fill soils are exposed on the excavation bottom. Over-excavation should extend at least one (1) foot laterally beyond the limits of footings or as limited by the existing structures or slope. Excavation activities should not disturb existing utilities, buildings, and remaining structures. These structures should be protected in place.

Existing soils exposed below proposed project areas should be scarified at least 12 inches, moisture conditioned as needed within three percent of optimum moisture content for granular soils and at approximately three percent above the optimum moisture for fine-grained soils, and compacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557) to produce a firm and unyielding surface.

Over-excavation should not undermine adjacent off-site improvements. If loose, yielding soil conditions are encountered, the following options can be considered:

a. Over-excavate until reach firm bottom.
b. Scarify or over-excavate additional 18 inches deep, and then place at least 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base should be compacted to 95% relative compaction.

The actual depth of removal should be based on recommendations and observation made during grading. Therefore, some variations in the depth and lateral extent of over-excavation should be anticipated. Over-excavation and re-compaction of upper alluvium and sedimentary bedrock is recommended for site grading to provide a relative uniform bearing material below proposed retaining wall footing.

7.3 Backfill Placement

Following observation of the excavation bottom, subgrade soil surfaces should be scarified to a depth of at least six inches. The scarified soil should be moisture-conditioned within three (3) percent of optimum moisture for granular soils and to approximate three (3) percent above the optimum moisture for fine-grained soil. Scarified soil shall be compacted to a minimum 90 percent of the laboratory maximum dry density as determined by the ASTM Standard D1557 test method.

Any import fill should be tested and approved by Project Geotechnical Consultant. The import fill should have an expansion potential less than 20. The imported materials should be thoroughly mixed, and moisture conditioned within three (3) percent above the optimum moisture. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D1557 test method.

Where the fill is not within the areas specified above or is not to support any structures, excavated site soils, free of deleterious materials and rock particles larger than three inches in the largest dimension, should be suitable for placement as compacted fill. The site materials should be thoroughly mixed, and moisture conditioned to approximate three percent above the optimum moisture, and then compacted to at least 90 percent of relative compaction.

7.4 Subgrade Preparation

Final subgrade soils for proposed improvements and pavement should be uniform and non-yielding. To obtain a uniform subgrade, soils should be well mixed and uniformly compacted. The subgrade soils should be moisture conditioned before placing concrete.

7.5 Excavatability

Based on our field exploration, the earth materials at the site should be excavatable with conventional heavy-duty earth moving and trenching equipment. Some gravel should be expected during excavation.

7.6 Pipe Backfill Recommendations

Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material.

7.6.1 Pipe Subgrade Preparation

The pipe subgrade should be level, firm, uniform, free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than two (2) inches in the largest dimension, if any, should be removed from the trench bottom and replaced with compacted materials.

During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

7.6.2 Pipe Bedding

The bedding zone is defined as that portion of the pipe trench from four inches below the pipe invert to one foot above the top of pipe, in accordance with Section 306-1.2.1 of the *Standard Specifications for Public Works Construction* (SSPWC) and Los Angeles County Department of Public Works Standard Plans, 3080-0, Case 3, Pipe Bedding in Trenches. On-site soils, in the upper soil profile, consisted primarily of sand and may not be suited for use as bedding material. Sandy soil materials with a Sand Equivalent of not less than 30 are acceptable for bedding material.

7.7 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface.

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site. No more than 30 percent of the backfill volume should be larger than ¾ inch in the largest dimension.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within three (3) percent of optimum moisture content and then placed in horizontal layers if the expansion index is less than or equal to 30. Should the expansion index be greater than 30, backfill materials shall be brought to approximately 3 percent above optimum moisture content. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened, or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by Converse during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

7.7.1 Select Imported Fill Materials for Trench Zone Backfill

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3 inches in the largest dimension
- Contain less than 30 percent by weight retained on ¾-inch sieve
- Contain at least 15 percent fines (passing #200 sieve)
- Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 7.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required. For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

8.2 Temporary Excavations

Based on the sandy materials encountered in the exploratory borings, sloped temporary excavations (if necessary) may be constructed according to the slope ratios presented in Table No. 9, *Slope Ratios for Temporary Excavations*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand may have to be constructed at a flatter gradient than presented in the following table:

Table No. 11, Slope Ratios for Temporary Excavations

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	vertical
4 – 8	1:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within 5 feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of 6 feet of stockpiled soils placed at least 5 feet from the trench edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

8.3 Slot Cut Recommendations

Temporary excavations during possible improvements should not extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of the existing utility lines or structures. The remedial grading excavations should not cause loss of bearing and/or lateral support for adjacent utilities or structures.

If remedial grading excavations extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of adjacent off-site utility lines or structure foundations, shoring or slot cutting shall be employed. “A-B-C” slot cuts exposing native sandy soils may be excavated with maximum 8 feet wide sections to prevent the existing utility lines or off-site structures from becoming unstable. Backfill should be accomplished in the shortest period of time possible and in alternating sections.

The ABC slot cutting method for retaining walls could be a possible option as an alternative to shoring for excavation less than 8 feet or with cohesive soils. In general, for structures it is not recommended for slot cutting if the height of excavation exceeds more than 8 feet or into sandy soils and with surcharging load.

8.4 Shoring Design

Temporary shoring will be required for the recommended excavation due to space limitations and property line boundaries and because of nearby existing structures or facilities and traffic loading. Temporary shoring may consist of the use of conventional soldier piles and lagging. Shoring should ultimately be designed by a qualified structural engineer considering the recommendations below in their final design and others which are applicable. Existing structures adjacent to excavation should be monitored for distress or excessive vibration during excavation.

8.4.1 Cantilevered Shoring

Cantilevered shoring systems may include soldier piles with lagging to maintain temporary support of vertical wall excavations. Shoring design must consider the support of adjacent underground utilities and/or structures and should consider the effects of shoring deflection on supported improvements. Due to sandy nature of on-site soils, some caving during the drilling of soldier-pile borings should be anticipated. A soldier pile system will require continuous lagging to control caving and sloughing in the excavation between soldier piles.

Temporary cantilevered shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 40 pounds per cubic foot (pcf) for non-surcharged condition. This pressure is valid only for shoring retaining level ground.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, existing structures, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the excavation. Surcharge pressures from the existing structures should be added to the above earth pressures for surcharges within a horizontal distance less than or equal to the wall height. Surcharge coefficients of 50% of any uniform vertical surcharge should be added as a horizontal earth pressure for shoring design. All shoring should be designed and installed in accordance with state and federal safety regulations.

The minimum embedment depth for piles is ten (10) feet from the lowest adjacent grade into firm alluvium, below the bottom of the excavation. Vertical skin friction against soldier piles may be taken as 250 psf. Fixity may be assumed at two (2) feet below the excavation into firm native alluvium or bedrock. For the design of soldier piles spaced at least 3.0 diameters on-center, the passive resistance of the soils adjacent to the piles may be assumed to be 250 psf per foot of embedment depth. Soldier pile members placed in drilled holes should be properly backfilled with a sand/cement slurry or lean concrete in order to develop the required passive resistance.

Caving soils should be anticipated between the piles. To limit local sloughing, caving soils can be supported by continuous lagging or guniting. The lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed for a nominal pressure of up to a maximum of 400 psf. All lumber to be left in the ground should be treated in accordance with Section 204-2 of the "Standard Specifications for Public Works Construction" (Latest Edition).

8.4.2 Tie-Back Shoring

A tie-back soldier-pile shoring system may be used to maintain temporary support of deep vertical walled excavations. Braced or tied-back shoring, retaining a level ground surface, should be designed for a uniform pressure of 20H psf, where H is the height of the retained cut in feet.

Surcharge pressures should be added to this earth pressure for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A surcharge coefficient of 50 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for braced shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation.

8.4.3 Tie-Backs

For design of tie-back shoring, it should be assumed that the potential wedge of failure is determined by a plane at 30 degrees from the vertical, through the bottom of the excavation. Tie-back anchors may be installed at angles of 15 to 40 degrees below a horizontal plane. Soil friction values, for estimating the allowable capacity of drilled friction anchors, may be computed using the following equation:

$q = 40\text{H}$; $q \leq 500$ *pounds-per-square-foot (psf)*

where:

H = average depth of anchor below ground surface
q = anchor surface area resistance, in psf (excluding tip),

Only the frictional resistance developed beyond the assumed failure plane should be included in the tie-back design for resisting lateral loads. After shoring/tie-back is no longer needed to support the excavation, stress should be carefully released and shoring system including tieback may be able to be left in place.

All shoring and tie-back should be designed by experienced California licensed Civil Engineer and installed by experienced contractors. Shoring/tie-back design should also be reviewed by a geotechnical consultant to verify the soil parameters used in the design are in conformance with geotechnical report.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by a competent person employed by the contractor. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

It is recommended that Converse review plans and specifications for proposed shoring and that a Converse representative observes the installation of shoring. A licensed surveyor should be retained to establish monuments on shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project Structural (shoring) Engineer and Converse for review and evaluation. Adjacent building elements should be photo-documented prior to construction.

8.5 Geotechnical Services During Construction

This report has been prepared to aid in the evaluation of the existing roadway pavement with respect to the planned pavement reconstruction/rehabilitation project. It is recommended that this office be provided an opportunity to review final design drawings and specifications to determine if the recommendations of this report have been properly implemented.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

9.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice within our profession at this time in this area. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field investigations and laboratory tests, combined with interpolation of soil conditions beyond the boring locations.

10.0 REFERENCES

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Appendix A

Field Exploration

APPENDIX A: FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the field exploration were determined. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used.

Three (3) exploratory borings (BH-1, BH-2, and BH-3) were advanced within the project site on August 24, 2020. All borings were drilled using a 4-inch diameter hand auger. The boring BH-1 was drilled to an explored depth of 12 feet below the existing ground surface (bgs), the boring BH-2 was drilled to an explored depth of 9 feet bgs, and the boring BH-3 was drilled to an explored depth of 7 bgs. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. California Modified Sampler (ring samples), and bulk soil samples were obtained for laboratory testing.

Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Figure No. A-1, *Soil Classification Chart*. The log of the exploratory boring is presented in Figures Nos. A-2, A-3, and A-4, *Log of Borings*.

Appendix B

Laboratory Testing Program

APPENDIX B: LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The following is a summary of the laboratory tests conducted for the investigation.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in-situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. Moisture Content and Dry Density Tests were performed in general accordance with the ASTM Standard D2216, and D7263 test method, respectively. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Soil Corrosivity

One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The pH and Saturated Resistivity Tests were performed in accordance with CT 643, the Soluble Chlorides Test is performed in accordance with CT 422, and the Soluble Sulfate Test is performed in accordance with CT 417. The corrosivity of soil is discussed according to Caltrans Corrosive Guidelines (2012) in section 6.3 of this report. The test results received from EGL are included in the following table:

Table No. B-1, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 643) Ohm-cm
BH-1	0-5	6.41	145	0.007	8,100

R-value

One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844 Standard. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table:

Table No. B-2, R-value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-3	0-5	Poorly graded sand with silt (SP-SM)	58

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected sample. Testing was performed in general accordance with the ASTM Standard D6913 test method. Grain-size distribution curves are shown in Figure No. B-1, *Grain Size Distribution Results*.

Maximum Dry Density Test

One (1) laboratory maximum dry density-moisture content relationship test was performed on a representative bulk sample of the upper 5 feet of soil material. The testing was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Figure No. B-2, *Moisture-Density Relationship Results*.

Direct Shear

Direct shear test was performed on one (1) undisturbed soil sample. Test was conducted in accordance with ASTM Standard D3080 laboratory procedure. Three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The sample was then sheared at a constant strain rate of 0.01 inch/minute. Shear deformation was recorded until a maximum of about 0.50-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density, see Figure No. B-3, *Direct Shear Test Results,* and the following table:

Table No. B-3, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1	5-6	Poorly graded sand with silt (SP-SM)	28	130

Swell/Collapse Test

Swell or Collapse test was performed on one (1) relatively undisturbed samples. Data obtained from this test was used to evaluate the settlement characteristics of the foundation soils under load. Tests were performed in general accordance with the ASTM Standard ASTM D4546. Preparation for this test involved trimming the sample and placing the 1-inch high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable equilibrium state. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. The sample was tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Figure No. B-4, *Swell/Collapse Test Results*.

Sample Storage

Soil samples presently stored in our laboratory will be discarded thirty days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.