APPENDIX F

GEOTECHNICAL STUDY

GEOTECHNICAL STUDY REPORT
Proposed Senior Living Project
514 North Prospect Avenue
Redondo Beach, California

Converse Project No. 15-31-312-01

June 24, 2016

Prepared For:

Beach Cities Health District
514 North Prospect Avenue
Redondo Beach, California 90277

Prepared By:

Converse Consultants
717 South Myrtle Avenue
Monrovia, California 91016

June 24, 2016

Mr. Leslie Dickey
Beach Cities Health District
514 North Prospect Avenue
Redondo Beach, California 90277

Subject: **GEOTECHNICAL STUDY REPORT**
Proposed Senior Living Project
Redondo Beach, California
Converse Project No. 15-31-312-01

Dear Mr. Dickey:

Converse Consultants (Converse) has prepared this geotechnical study report to present the findings, conclusions and recommendations of our geologic and geotechnical study for the Proposed Senior Living Project for Beach Cities Health District in Redondo Beach, California. Our services were performed in accordance with our proposal dated December 10, 2015.

Based on our field exploration, laboratory testing, geologic evaluation, and geotechnical analysis, the site is suitable from a geotechnical standpoint for the proposed project, provided our conclusions and recommendations are implemented during design and construction.

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

CONVERSE CONSULTANTS

Sivathasan

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

REGISTERED PROFESSIONAL ENGINEER
KATHIRAVETPILLAI SIVATHASAN
No. 2708
Exp 6-30-18
GEOTECHNICAL
STATE OF CALIFORNIA

Dist: 4/Addressee

MM/MBS/SKS:jjl

PROFESSIONAL CERTIFICATION

This report for the Proposed Senior Living Project located at 514 North Prospect Avenue, in the City of Redondo Beach, California, has been prepared by the staff of Converse Consultants 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.

In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions contained in this report may not be valid unless these changes and additional relevant information are reviewed and the recommendations of this report are modified or verified in writing.

Mohammad Malim, EIT
Senior Staff Engineer

PROFESSIONAL GEOLOGIST
MARK B. SCHLUTER
No. 1415
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Mark B. Schluter, PG, CEG
Senior Engineering Geologist

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REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL
STATE OF CALIFORNIA

EXECUTIVE SUMMARY

The following is a summary of our geotechnical study, conclusions and recommendations, as presented in the body of this report, please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed project is located at 514 North Prospect Avenue in Redondo Beach, California. The proposed project will consist of development of multi-level buildings on two vacant properties at Diamond Street and Flagler Lane; and Beryl Street and Flagler Lane, in the City of Redondo Beach, California, for the possible development of a Senior Living Project.

- Twelve (12) exploratory borings (BH-1 through BH-10 and PT-1 and PT-2) were drilled within the project sites on March 31, 2016 and on April 4 and 5, 2016. The borings were advanced using a limited access track drill rig and a truck-mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 10 to 61.5 feet below the existing ground surface (bgs). Borings PT-1 and PT-2 were utilized for percolation tests prior to backfill.

- There are no known active faults projecting toward or extending across the proposed site. The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture.

- The site is not located within a potential liquefaction zone per the State of California Seismic Hazard Zones Map for the Los Angeles Quadrangle as shown in Drawing No. 5, *Seismic Hazard Zone Map*. Based on the results of our subsurface exploration, including the absence of shallow groundwater, relatively dense soils with high blow counts and our experience on similar projects it is concluded that the subject site is not considered susceptible to liquefaction.

- Groundwater was not encountered in our exploratory borings to a maximum depth of 61.5 feet. Groundwater is not anticipated during construction and will not need to be considered in design.

- The on-site soil has a “Very Low” expansive potential and mitigation for expansive soils is not anticipated.

- In general, the pH value, chloride content, and water soluble sulfates of the site soils are in the non-corrosive range. The saturated resistivity of samples taken are in the non-corrosive range to ferrous metals.

- Variable thickness undocumented fill soils were encountered in the borings. The undocumented fill is not considered suitable for any slab or foundation support.

- The earth materials at the site should be excavatable with conventional heavy-duty earth moving and trenching equipment. The on-site soil materials contain about 5 to 10 percent gravel up to 3 inches in maximum dimension. Larger gravels, cobbles and possible boulders may exist at the site. Earthwork should be performed with suitable equipment for gravelly materials.

- Shallow footings or deep foundations are considered suitable for structure support provided the recommendations in this report are incorporated into the project plans, specifications, and are followed during site construction.

Results of our investigation indicate that the site is suitable from a geotechnical standpoint for the proposed development, provided that the recommendations contained in this report are incorporated into the design and construction of the project.

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APPENDICES

1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the proposed Senior Living Project located at 514 North Prospect Avenue, in the City of Redondo Beach, California, as shown on Drawing No. 1, *Site Location Map*.

The purpose of this work was to evaluate the subsurface soil conditions, perform geotechnical analyses, provide design and construction recommendations for the two (2) proposed building sites, and provide design recommendations for the design of storm water infiltration systems, including current standard of practice seismic and geotechnical engineering interpretations.

We have used site plans provided to us by your office as a reference for this project. The current site plan is included in this report as Drawing No. 2, *Site Plan and Boring Location Map*.

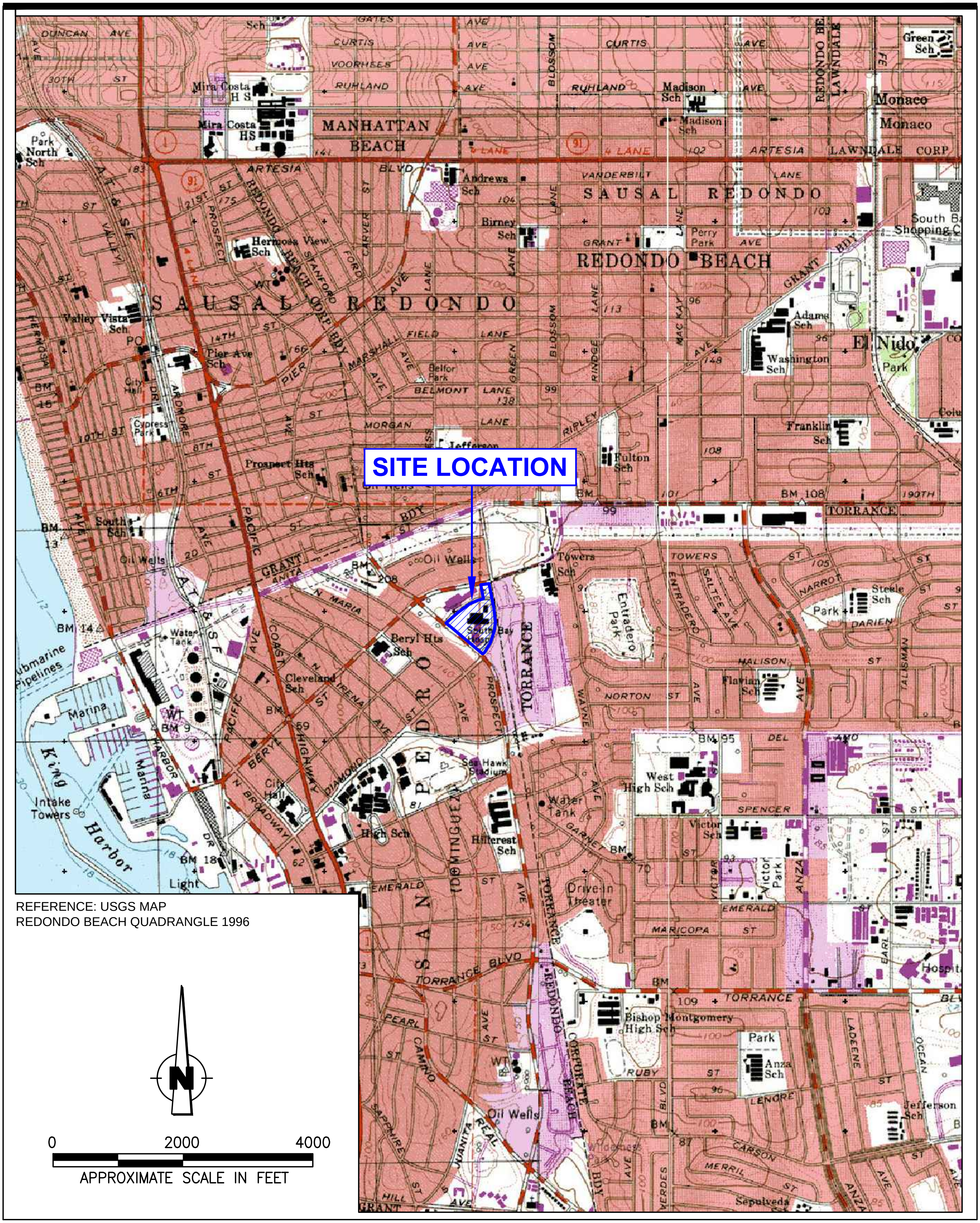
This report is written for the project described herein and is intended for use solely by Beach Cities Health District and its design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

2.0 SITE AND PROJECT DESCRIPTION

The proposed project site is located at 514 North Prospect Avenue in Redondo Beach, California. The site dimensions are approximately 140 feet east-west by 150 feet north-south at the Beryl/Flagler lot (northern portion of the site), and an approximately 75 feet east-west by 500 feet north-south at the Diamond/Flagler lot located along the existing eastern slope of the site. Both portions of the site are currently undeveloped.

The Beryl/Flagler lot portion of the site is relatively flat, with surface elevations ranging from approximately 135 to 145 feet relative to mean-sea-level (MSL), with surface gradients toward the northeast. The Diamond/Flagler parcel consists of an existing slope with an approximate 2:1 (horizontal: vertical) gradient with surface elevations ranging from approximately 130 to 155 feet relative to mean-sea-level (MSL), sloping toward the east on the eastern portion of the site. The sites are bounded by Beryl Street to the north, by a shopping center to the northwest, by Prospect Avenue to the Southwest, by Diamond Street to the southeast, and by Flagler Lane to the east. The site coordinates are: North latitude: 33.8537 degrees, West longitude: 118.3786 degrees.

The proposed project will consist of development of a multi-level building(s) on two vacant properties at Diamond Street and Flagler Lane and Beryl Street and Flagler



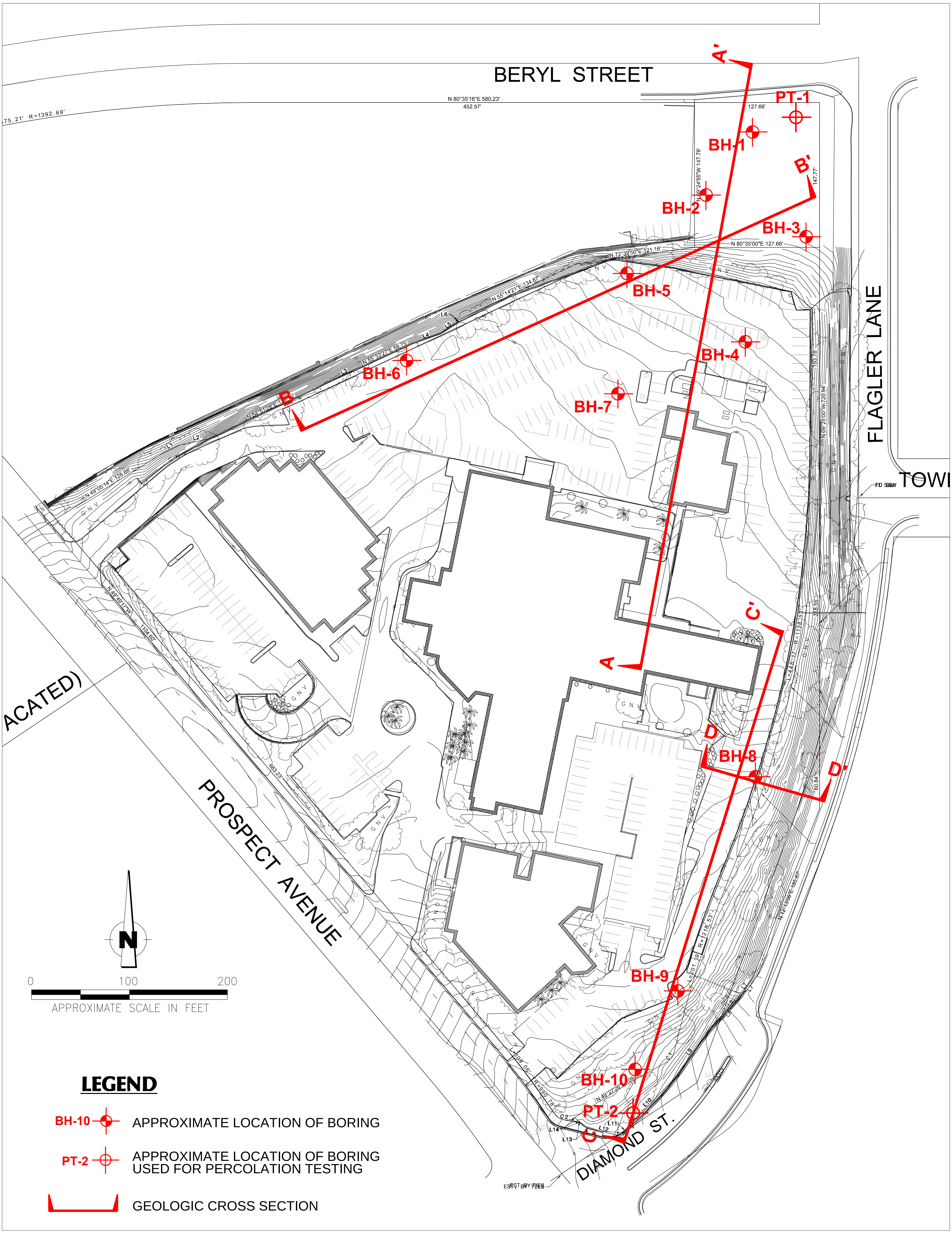
REFERENCE: USGS MAP
REDONDO BEACH QUADRANGLE 1996

SITE LOCATION MAP

Converse Consultants

SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	1



SITE PLAN AND BORING LOCATION MAP

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SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	2

Lane in the City of Redondo Beach, California for the possible development of a Senior Living Project. The structural loads are not known at this time but are anticipated to be moderate. The structures are planned to be founded on shallow foundations or concrete mat foundations.

3.0 SCOPE OF WORK

The scope of our work included a site reconnaissance, subsurface exploration with soil sampling, landscape soil sampling and testing, percolation testing, laboratory testing, engineering analysis, and preparation of this report.

3.1 Site Reconnaissance

During the site reconnaissance on March 14, 2016, the surface conditions were noted and the locations of the borings were determined so that drill rig access to all the locations was available. The borings were located using existing boundary features as a guide and should be considered accurate only to the degree implied by the method used. Underground Service Alert (USA) of Southern California was notified of our proposed drilling locations at least 48 hours prior to initiation of the subsurface field work.

3.2 Subsurface Exploration and Percolation Testing

Twelve (12) exploratory borings (BH-1 through BH-10 and PT-1 and PT-2) were drilled within the project sites on March 31, 2016 and on April 4 and 5, 2016. The borings were advanced using a limited access track drill rig and a truck-mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 10 to 61.5 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

California Modified Sampler (Ring samples), Standard Penetration Test samples, and bulk soil samples were obtained for laboratory testing. Standard Penetration Tests (SPTs) were performed in selected borings at selected intervals using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The bore holes were backfilled and compacted with soil cuttings by reverse spinning of the auger following the completion of drilling and patched with asphalt where necessary to match existing conditions.

Borings PT-1 and PT-2 were utilized for percolation tests prior to backfill. Percolation test procedures and test results are further discussed in report section 7.0, *Percolation Testing* and Appendix C.

The approximate locations of the exploratory borings are shown in Drawing No. 2, *Site Plan and Boring Location Map*. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

3.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in the classification and to evaluate relevant engineering properties. The tests performed included:

- In Situ Moisture Contents and Dry Densities (ASTM Standard D2216)
- Grain Size Distribution (ASTM Standard C136)
- Fines Content/Passing No. 200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D4318)
- Maximum Dry Density and Optimum-Moisture Content Relationship (ASTM Standard D1557)
- Direct Shear (ASTM Standard D3080)
- Residual Direct Shear (ASTM Standard D6467)
- Consolidation (ASTM Standard D2435)
- Expansion Index (ASTM Standard D4829)
- R-value (ASTM Standard D2844)
- Soil Corrosivity Tests (Caltrans 643, 422, 417, and 532)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*. For *in-situ* moisture and density data, see the Logs of Borings in Appendix A, *Field Exploration*.

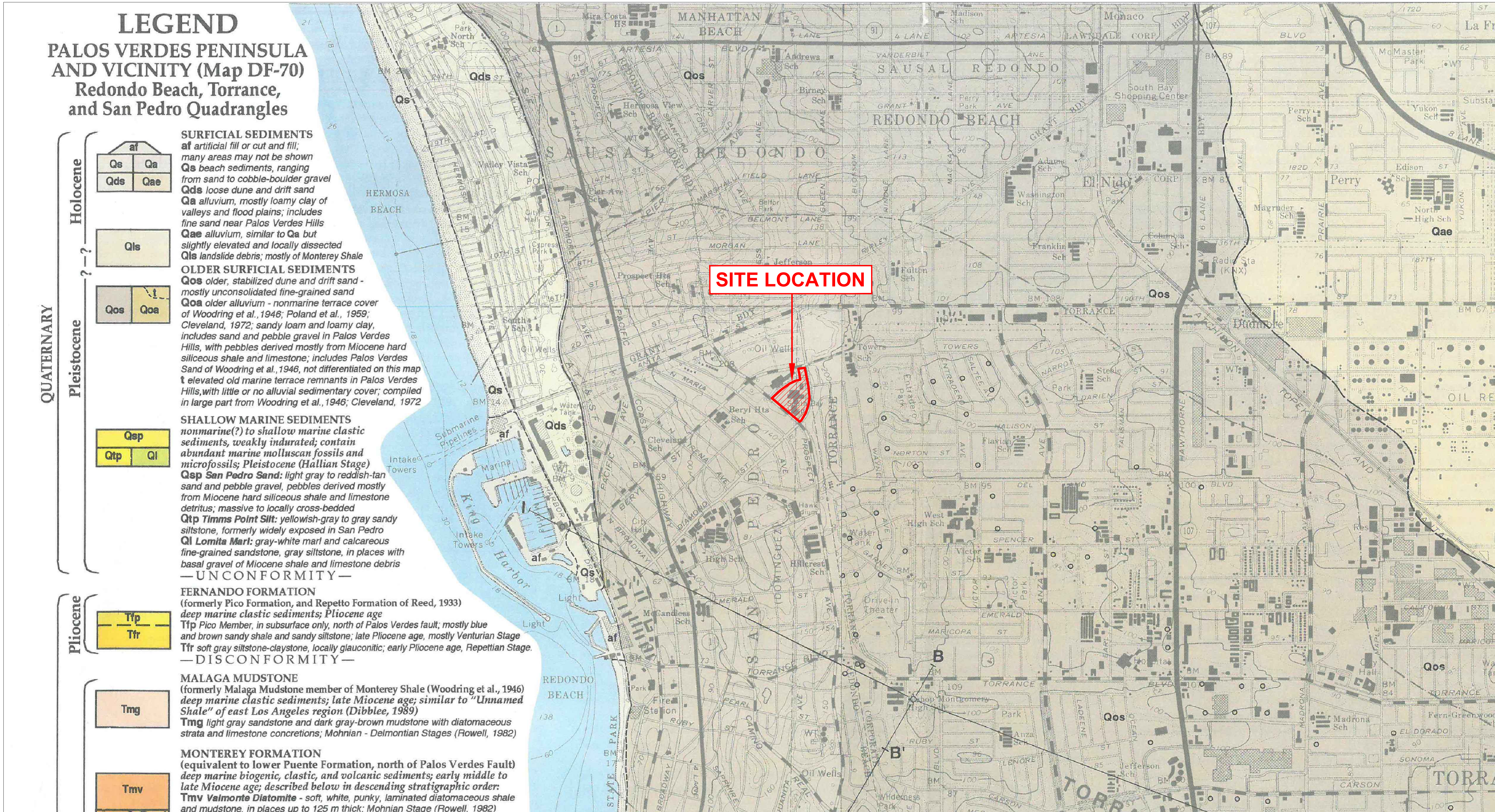
3.4 Engineering Analyses and Report

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

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geology

The project site is located in the western portion of the Redondo Beach 7.5-minute quadrangle, as shown on Drawing No. 3, *Regional Geologic Map*. The site is located on a northwest-trending coastal plain, locally known as the Torrance Plain. This plain consists of dense silty sand and sand deposits of older, stabilized dune and drift sands covered with moderately dense silty sand and sandy clay of younger alluvial deposits.



REFERENCE: GEOLOGIC MAP OF THE
PALOS VERDES PENINSULA AND VICINITY REDONDO BEACH,
TORRANCE, AND SAN PEDRO QUADRANGLES

N

0 2000 4000
APPROXIMATE SCALE IN FEET

REGIONAL GEOLOGIC MAP

Converse Consultants

SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
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The project site is underlain by deep alluvial deposits that have gradually filled the Los Angeles basin and coastal plains.

4.2 Subsurface Profile of Subject Site

Based on our data obtained from our field exploration, the subsurface conditions generally consist of existing fill soils placed during previous site grading operations and natural alluvial soils, as encountered in the borings drilled to the maximum depth explored of 61.5 feet below the ground surface (bgs). The observed fill soils consist primarily of silty sand and clayey sand. The depth of the fill ranges from approximately three (3) to thirteen (13) feet. The alluvial sediments consist predominately of older dune and drift sand encountered to a maximum drilled depth of approximate 61.5 feet below ground surface. Based on our experience on nearby projects, larger size gravels and cobbles should be anticipated during excavations. A review of the regional geology of the site shows that much of the alluvial soils on the site are stabilized dune and drift sand as shown on Drawing No. 3, *Regional Geologic map.*

Subsurface geologic conditions beneath the subject site are depicted on Drawings No. 4a, *Geologic Cross-Section A-A';* No. 4b, *Geologic Cross-Section B-B';* No. 4c, *Geologic Cross-Section C-C'; and* No. 4d, *Geologic Cross-Section D-D'*. The geologic cross-sections show the interpreted extent and limits of the different types of subsurface materials encountered during our study. For additional information on the subsurface conditions, see the Logs of Boring Data in Appendix A, *Field Exploration*.

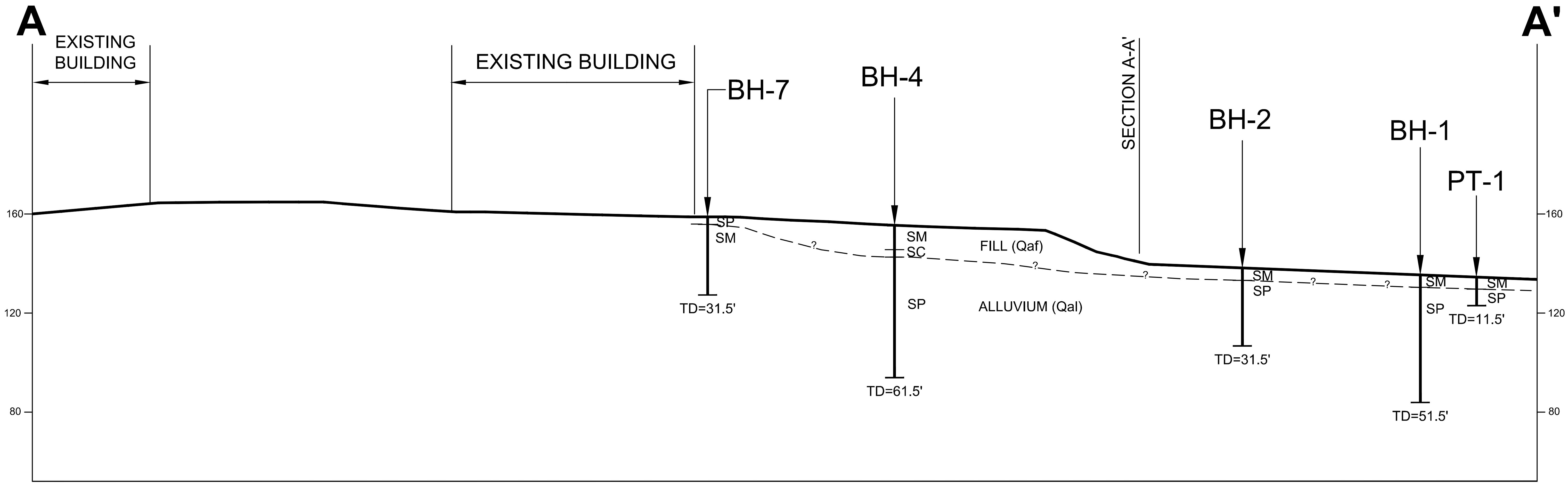
4.3 Groundwater

Groundwater was not encountered in our exploratory borings to a maximum depth of 61.5 feet. In accordance with the Seismic Hazard Zone Report for the Redondo Beach Quadrangle (CDMG, 1998), the historically highest groundwater level is reportedly at depths of greater than 50 feet. Groundwater is not anticipated during construction and will not need to be considered in design.

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present at various depths due to local conditions or during rainy seasons. Groundwater conditions below any given site vary depending on numerous factors including seasonal rainfall, local irrigation, and groundwater pumping, among other factors.

4.4 Subsurface Variations

Based on results of the subsurface exploration and our experience with the subject area, some variations in the continuity and nature of subsurface conditions within the project site are anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in



PROFILE
HORIZONTAL: 1"=40'
VERTICAL: 1"=40'

0 40

APPROXIMATE SCALE IN FEET

GEOLOGIC CROSS SECTION A-A′

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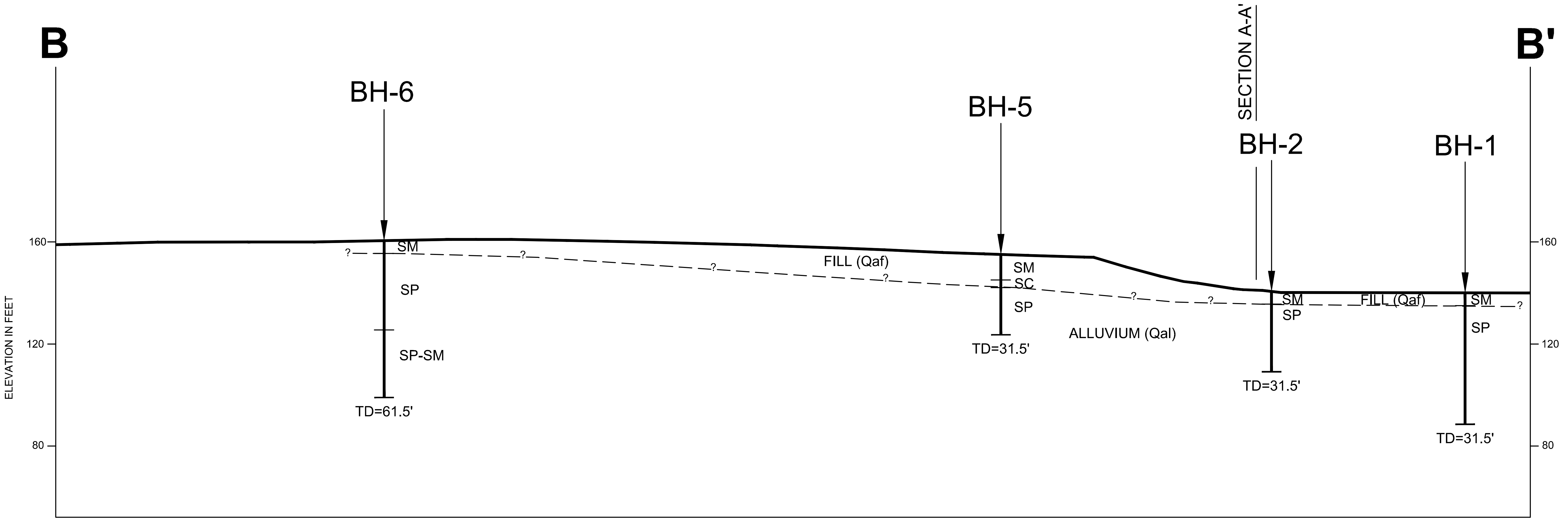
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	4a



PROFILE
HORIZONTAL: 1"=40'
VERTICAL: 1"=40'

0 40
APPROXIMATE SCALE IN FEET

GEOLOGIC CROSS SECTION B-B′

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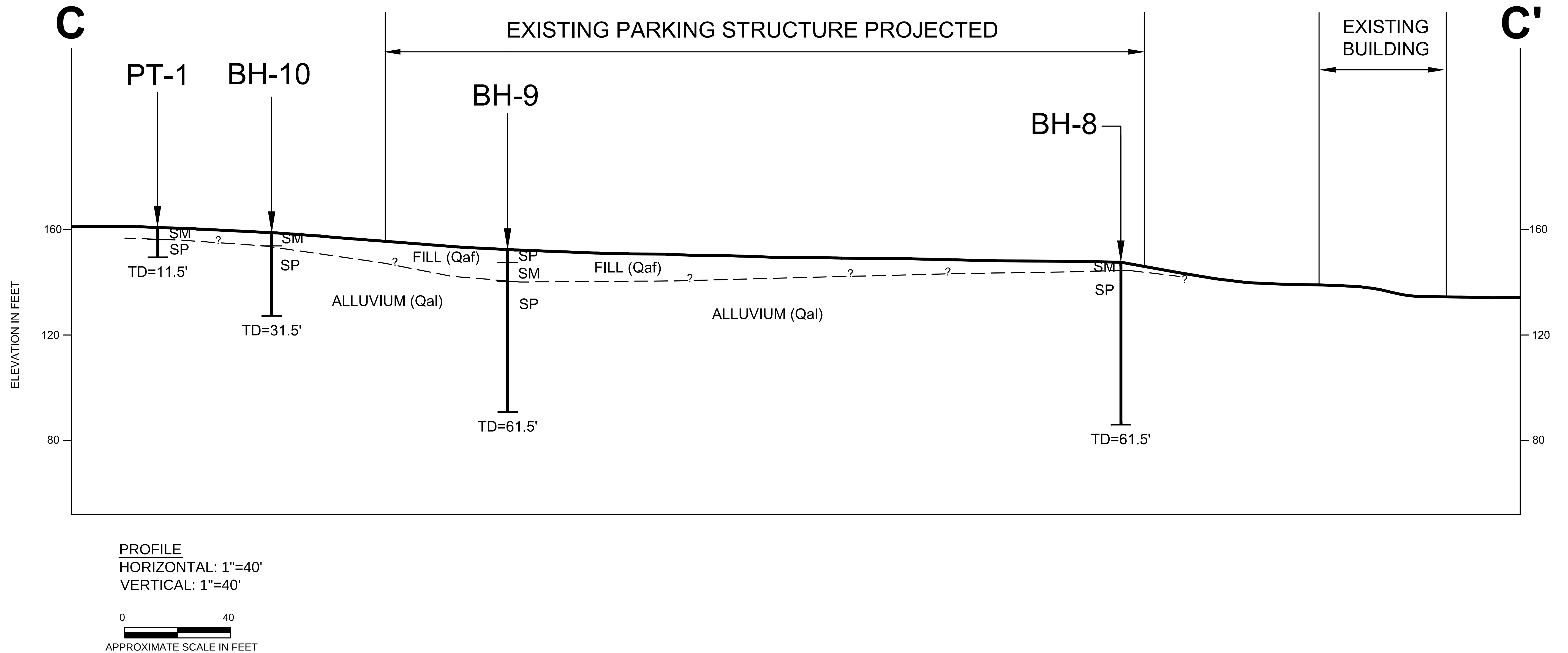
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	4b

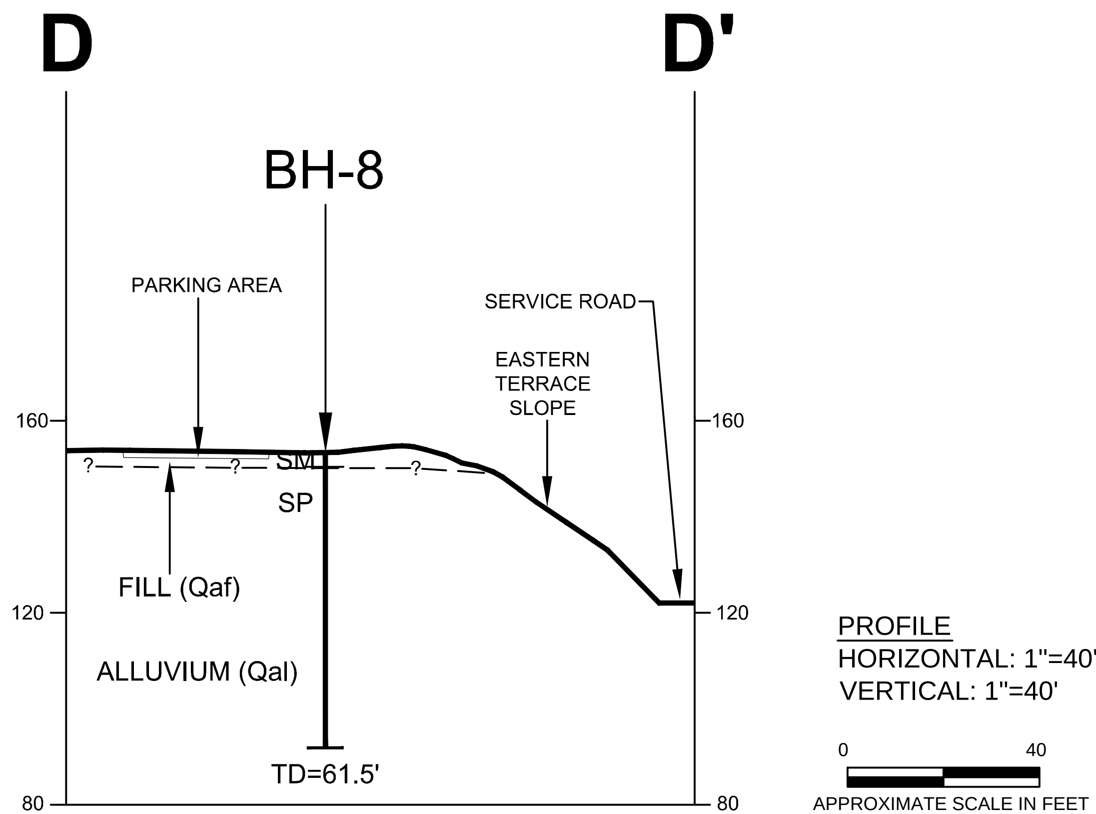


GEOLOGIC CROSS SECTION C-C'

Converse Consultants

SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	4c



GEOLOGIC CROSS SECTION D-D'

Converse Consultants

SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	4d

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 modified, if necessary.

5.0 FAULTING AND SEISMIC HAZARDS

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, liquefaction and seismically induced settlement, lateral spreading, landslides, earthquake induced flooding, tsunamis and seiches, and volcanic eruption hazard.

Results of a site-specific evaluation for each type of possible seismic hazards are discussed in the following sections.

5.1 Fault Surface Rupture and Active Faults

The project site is not located within a currently designated State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zones) for surface fault rupture (Special Studies Zone, Los Angeles Quadrangle, 1977). No surface faults are known to project through or towards the site. The closest known fault to the project site with a mappable surface expression is the Newport Inglewood Fault, mapped approximately 6.3 miles northeast of the project site.

Newport Inglewood Fault

The Newport Inglewood fault zone is located at approximately 6.3 miles northeast of the project site. The Newport Inglewood fault system is about 66 km long on shore and extends northwest from Huntington Beach through Long Beach to Culver City and Cheviot Hills. The Newport Inglewood fault continues offshore to the southeast of Huntington Beach and makes landfall in La Jolla as the Rose Canyon fault. The Newport Inglewood fault is characterized by a series of uplifts and anticlines including Newport Mesa, Huntington Beach Mesa, Bolsa Chica Mesa, Alamitos Heights and Landing Hill, Signal Hill and Reservoir Hill, Dominguez Hills and Baldwin Hills.

Several earthquakes have occurred along the fault zone including the March 10, 1933 "Long Beach" earthquake of M_W 6.4, with its epicenter off Newport Beach, and smaller earthquakes at Inglewood on June 20, 1920 (M 4.9), Gardena on November 14, 1941 (M 5.4). These earthquakes show evidence of right-lateral strike slip focal mechanisms.

The Newport Inglewood fault is considered to be active and considered capable of producing a maximum moment magnitude (Mw) 7.1 earthquake. The slip rate is considered to be about 1.0 mm/year but may range up to 2 to 3 mm/year along isolated segments (Cao et al., 2003).

Seismic hazard fault models for the Los Angeles Basin and vicinity will continue to be refined as new information and technology develops and becomes available through time.

5.2 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

The site is not located within a potential liquefaction zone per the State of California Seismic Hazard Zones Map for the Redondo Beach Quadrangle as shown in Drawing No. 5, *Seismic Hazard Zone Map*. Based on the results of our subsurface exploration, including the absence of shallow groundwater, relatively dense soils with high blow counts and our experience on similar projects it is concluded that the subject site is not considered susceptible to liquefaction. We anticipate total seismically-induced settlement to be on the scale of 0.50 inches and differential settlement to be less than 0.25 inches over a distance of 30 feet.

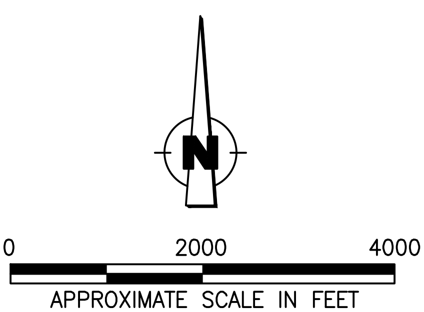
5.3 Lateral Spreading

Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography at the project site and in the immediate vicinity of the site is relatively flat, with no significant nearby slopes or embankments. Under these circumstances, the potential for lateral spreading at the subject site is considered negligible.

5.4 Seismically-Induced Slope Instability

Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The project site is not located within an area of earthquake-induced landslide as shown on Drawing No. 5, *Seismic Hazard Zone Map*. The project site is underlain by dense alluvial deposits on an older terrace slope. No evidence of

REFERENCE: STATE OF CALIFORNIA SEISMIC HAZARD ZONES REDONDO BEACH QUADRANGLE 1999



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

SEISMIC HAZARD ZONE MAP

Converse Consultants	SENIOR LIVING PROJECT 514 NORTH PROSPECT AVENUE REDONDO BEACH, CALIFORNIA FOR: BEACH CITIES HEALTH DISTRICT	Project No. 15-31-312-01	Drawing No. 5

landslides was observed on descending hillside slopes below the site. The potential for seismically induced landslides to affect the proposed site is considered to be very low.

5.5 Earthquake-Induced Flooding

Review of the FEMA Flood Insurance Rate Maps (FIRM), Los Angeles County Map Number 06037C1907F, dated September 26, 2008, indicates that the site is located within an area designated as Zone X, described as an area outside a 0.2% annual flood chance. Since the site is not located within a flood plain subject to a 1.0% or greater chance of flooding in any year, the site is not located within a flood hazard area as defined by the CBC.

5.6 Tsunami and Seiches

Tsunami Inundation Map for Emergency Planning for the Redondo Beach Quadrangle, the site is not located within a mapped Tsunami Inundation Area as shown on Drawing No. 6, *Tsunami Inundation Map*. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on site location away from lakes and reservoirs, seiches do not pose a hazard.

5.7 Volcanic Eruption Hazard

There are no known volcanoes near the site. According to Jennings (1994), the nearest potential hazards from future volcanic eruptions is the Amboy Crater-Lavic Lake area located in the Mojave Desert more than 120 miles east/northeast of the site. Volcanic eruption hazards are not present.

6.0 SEISMIC ANALYSIS

6.1 CBC Seismic Design Parameters

Seismic parameters based on the 2013 California Building Code are calculated using the United States Geological Survey *U.S. Seismic Design Maps* website application and the site coordinates (33.8537 degrees North Latitude, 118.3786 degrees West Longitude). The seismic parameters are presented below.

REFERENCE: STATE OF CALIFORNIA, COUNTY OF LOS ANGELES,
REDONDO BEACH QUADRANGLE,
CALIFORNIA EMERGENCY MANAGEMENT AGENCY,
CALIFORNIA GEOLOGICAL SURVEY, USC

N

NOT TO SCALE

TSUNAMI INUNDATION MAP

Converse Consultants	SENIOR LIVING PROJECT 514 NORTH PROSPECT AVENUE REDONDO BEACH, CALIFORNIA FOR: BEACH CITIES HEALTH DISTRICT	Project No. 15-31-312-01	Drawing No. 6

Table No. 1, CBC Seismic Design Parameters

Seismic Parameters	2013 CBC
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.613 g
Mapped 1-second Spectral Response Acceleration, S_1	0.612 g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.613 g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.918 g
Design Spectral Response Acceleration for short period, S_{DS}	1.075 g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.612 g
Seismic Design Category	D

7.0 PERCOLATION TESTING RESULTS

Percolation testing was performed utilizing exploratory borings PT-1 and PT-2 on March 21 and April 5, 2016. The tests were performed using the falling head test method in accordance with Los Angeles County "Low Impact Development Best Management Practice Guideline for Design, Investigation, and Reporting". The results of the percolation tests are tabulated below and presented in Appendix C, *Percolation Testing*.

Table No. 2, Percolation Testing Results

Boring No.	Depth of Boring* (feet)	Predominant Soil Types (USCS)	Average Percolation Rate (inches/hour)	Lowest Percolation Rate (inches/hour)
PT-1	10	Sand (SP)	13.24	4.18
PT-2	10	Silty Sand (SM) over Sand (SP)	4.84	3.08

*Approximate

In accordance with County of Los Angeles requirements, the minimum percolation rate for design of infiltration system for storm water management is 0.3 inch per hour. Therefore, the soils at the site are suitable for infiltration system. The project Civil Engineer shall review the raw data of percolation test presented in *Appendix C* to determine specific soil layers and percolation rates for design of the proposed infiltration system. Infiltration system should be properly maintained periodically to minimize sedimentation in the infiltration system. A proposed infiltration system must comply with the following setbacks in accordance with Los Angeles County guideline.

Table No. 3, Infiltration Facility Setback Requirements per Los Angeles County

Setback from	Distance
Property lines and public right of way	5 feet
Any foundation	15 feet or within 1:1 plane drawn up from the bottom of foundation, whichever greater
Face of any slope	H/2, 5 feet minimum (H is height of slope)
Water wells used for drinking water	100 feet

8.0 GEOTECHNICAL EVALUATIONS AND CONCLUSIONS

Based on the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned site development, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans, specifications, and are followed during site construction. The following is a summary of the major geologic and geotechnical factors to be considered for the planned project:

- There are no known active faults projecting toward or extending across the proposed site. The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture.

- The site is not located within a mapped Seismic Hazard Zone for liquefaction. Based on the results of our subsurface exploration, including the absence of shallow groundwater, relatively dense soils with high blow counts and our experience on similar projects it is concluded that the subject site is not considered susceptible to liquefaction.

- Groundwater was not encountered in our exploratory borings to a maximum depth of 61.5 feet. Groundwater is not anticipated during construction and will not need to be considered in design.

- Variable thickness undocumented fill soils were encountered in the borings. The undocumented fill is not considered suitable for any slab or foundation support.

- The on-site soil has a “Very Low” expansive potential and mitigation for expansive soils is not anticipated.

- Site soils have “negligible” concentrations of water soluble sulfates.

- In general, the pH value, chloride content, and saturated resistivity of the site soils are in the non-corrosive range. The saturated resistivity of samples taken are in the non-corrosive range to ferrous metals.

- The earth materials at the site should be excavatable with conventional heavy-duty earth moving and trenching equipment. The on-site materials contain about 5 to 10 percent gravel up to 3 inches in maximum dimension. Larger gravels, cobbles and possible boulders may exist at the site. Earthwork should be performed with suitable equipment for gravelly materials.

- Shallow footings or deep foundations are considered suitable for structure support provided the recommendations in this report are incorporated into the project plans, specifications, and are followed during site construction. Foundation recommendations are based on the different subsurface soils anticipated beneath planned structure.

9.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

9.1 General Evaluation

Based on our field exploration, laboratory testing, and analyses of subsurface conditions at the site, remedial grading is required to prepare the site for support of the proposed structures. The subject site has slopes. It is anticipated that the site preparation may include both cut and fill. 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. It is our understanding that site preparation will require removal of existing structures with their foundations and other existing underground manmade structures and utilities.

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. Rocks larger than one (1) inch should not be placed within the upper 12 inches of subgrade soils. 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

9.2 Over-Excavation/Removal

For new structural improvements, prior to the start of construction, all loose soil, undocumented fill and soil disturbed during demolition should be removed to firm acceptable native material or compacted fill.

Due to the undocumented fill encountered at the site, we recommend the future planned building site be over-excavated to a depth of 5 feet below the existing grade, 3 feet below bottom of footings or depth of undocumented fill, whichever is deeper. Over-excavation should extend at least five (5) feet laterally beyond the limits of perimeter footings where feasible.

Over-excavation for retaining walls, if any, should be two (2) feet below bottom of footings and should extend three (3) feet laterally beyond the retaining wall area. The upper 24 inches of site soils should be removed in areas of sidewalks and surface parking. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, deeper removal will be required until firm native soils are encountered. The over-excavation should extend two (2) feet laterally beyond the sidewalk and surface parking areas. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, deeper removal will be required until firm native soils are encountered.

Excavation activities should not disturb adjacent utilities or undermine any adjacent buildings and structures to remain. Existing utilities should be removed and adequately capped at the project boundary line, or salvaged/rerouted as designed.

The actual depth of removal should be based on recommendations and observations made during grading. Therefore, some variations in the depth and lateral extent of over-excavation recommended in this report should be anticipated.

Site grading may result in transition lines between cut and/or fill conditions. This transition line would require special grading considerations. Detailed site grading recommendations are provided in the following sections. In order to provide a relative uniform bearing material below shallow foundations, over-excavation and re-compaction of a minimum of 3 feet below the bottom of foundations and slabs-on-grades are recommended.

9.3 Structural Fill

The approved bottom of the excavations should be scarified to a depth of at least six (6) inches. The scarified soils should be moisture conditioned to near-optimum moisture content and compacted to at least 90 percent of the laboratory maximum dry density to produce a firm and unyielding surface.

All structural fill should be placed on competent, scarified and compacted native materials as determined by a geotechnical engineer and in accordance with the specifications presented in this section. Excavated site soils, free of deleterious materials and rock fragments larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any import fill should be tested and approved by Converse. The import fill should have an expansion potential less than 20.

Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned when necessary, within three (3) percent of the optimum moisture for granular soils and at approximately three (3) percent above the optimum moisture for fine-grained soils. 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. The amount of processing required for proper moisture conditioning at the site will depend on the seasonal variations in the *in-situ* moisture conditions, the depth of cut, the equipment, and the processing method.

All exposed subgrade soil surfaces should be observed by a Converse representative prior to placement of fill, base materials or slabs. The exposed subgrade should be scarified at least 6 inches, moisture conditioned as needed to within 2 percent above optimum moisture content, and compacted to 90 percent relative compaction. The upper 12 inches of subgrade below new pavement should be compacted to 95 percent relative compaction.

If loose, yielding soil conditions are encountered at the excavation bottom, the following options can be considered:

a. Over-excavate until firm bottom is reached.
b. Over-excavate additional 18 inches deep below subgrade, 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.
c. Over-excavate additional 18 inches deep below subgrade, then place a layer of geogrid (i.e. Mirafi HP570, X600 or equivalent), and place 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base should be compacted to 95% relative compaction. An additional layer of geogrid may be needed on top of base depending on the actual site conditions.

Fill exceeding five (5) feet in height shall not be placed on native slopes that are steeper than 5:1 horizontal:vertical (H:V). Where native slopes are steeper than 5:1 H:V, and the height of the fill is greater than five (5) feet, the fill shall be keyed and benched into competent materials. The height and width of the benches shall be at least two (2) feet.

9.4 Excavatability

Based on our field exploration, the earth materials at the site should be excavatable with conventional heavy-duty earth moving and trenching equipment. The onsite materials contain about 5 to 10 percent gravels up to 3 inches in maximum dimension. Larger gravels, cobbles and possible boulders may exist at the site. Earthwork should be performed with suitable equipment for gravelly materials.

9.5 Expansive Soil

Based on our laboratory testing results, the on-site clayey earth materials are considered to be very “Very Low” expansion potential. Mitigation for expansive soils is not considered necessary. The on-site soil materials will be mixed during the grading and the expansion potential might change. Therefore, the expansion potential of site soils should be verified after the grading as slabs, foundations and pavement placed directly on expansive subgrade soil will likely crack over time.

The recommendations contained in this report are based on the anticipated expansion soil conditions. Any proposed import fill should have an expansion index less than 20, and should be evaluated and approved by Converse prior to import to the site.

9.6 Pipeline Backfill Recommendations

Any soft and/or unsuitable material encountered at the pipe invert should be removed and replaced with an adequate bedding material. 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. 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 Latest Edition of the *Standard Specifications for Public Works Construction* (SSPWC).

9.7 Trench Zone Backfill

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 two (2) 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 2 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.

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 less than 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.

9.8 Shrinkage and Subsidence

The shrinkage and/or bulkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. For preliminary estimation, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The approximate shrinkage factor for the upper ten (10) feet of alluvial soils is estimated to range from ten (10) to twenty (20) percent.
- Subsidence would depend on the construction methods including type of equipment utilized. For estimation purposes, ground subsidence may be taken as 0.20 feet.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

9.9 Subgrade Preparation

Final subgrade soils for structures and streets should be uniform and non-yielding. To obtain a uniform subgrade, soils should be well mixed and uniformly compacted. The subgrade soils should be non-expansive and well-drained. The near-surface site soils should be free draining. We recommend that at least the upper two (2) inches of subgrade soils underneath the slab-on-grade should be comprised of well-drained granular soils such as sands, gravel or crushed aggregate satisfying the following criteria:

- Maximum size ≤ 1.5 inches
- Percent passing U.S. #200 sieve ≤ 12 percent
- Sand equivalent ≥ 30

The subgrade soils should be moisture conditioned before placing concrete.

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the earthwork and site grading recommendations provided in this report will be followed. The proposed buildings may be supported by shallow continuous and isolated square footings.

10.0 DESIGN RECOMMENDATIONS

10.1 Shallow Foundations

10.1.1 Vertical Capacity

Continuous and square footings should be founded at least 24 inches below lowest adjacent final grade on the recommended earth materials. A minimum footing width of 24 inches is recommended for continuous and square footings. The net allowable dead plus live load bearing value for isolated square and continuous footings is 3,000 psf. The net allowable bearing pressure can be increased by 350 psf for each additional foot of excavation depth and width up to a maximum value of 5,000 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.

10.1.2 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 350 psf per foot of depth up to a maximum of 3,000 psf may be used for footings poured against properly compacted fill or undisturbed stiff natural soils. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

10.1.3 Settlement

The static settlement of structures supported on continuous and/or spread footings founded on compacted fill 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.

10.1.4 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.

10.2 Pier Foundations

The planned development on the eastern slope of the site and cantilever light poles, can be supported on piers (caissons) provided the following recommendations incorporated into design and construction. The piers can be connected to a grade beam system determined by the project structural engineer to control the deflections of structure under the design tolerance.

10.2.1 Vertical Capacity

Piers should be at least 24-inch in diameter extending at least 8 feet below adjacent final grade on compacted fill or native alluvial soils. Piers can be designed for an allowable skin friction of 300 psf against the perimeter of pier for a minimum embedment of 8 feet below the adjacent grade or depth of fill, whichever is greater. Furthermore, sonotubes should be used for the depth of the installed piers equal to that of the depth of fill. The soil skin friction associated with the depth of installed sonotubes should be neglected in pier capacity calculations.

If end bearing capacity is to be considered for design, the bottom of pier should be cleaned out with appropriate equipment. The allowable end bearing capacity can be designed for 3,500 psf. However, the diameter of pier may need to be increased and temporary casing may be required to facilitate cleanout.

10.2.2 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 350 psf per foot of depth up to a maximum of 3,000 psf may be used for foundations poured against compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5. For ground surface restrained by concrete slab, the passive resistance may be calculated from the ground surface. For unrestrained ground condition, the passive resistance of the upper one (1) feet earth material should be neglected in design.

10.2.3 Settlement

The static settlement of structures supported on piers founded on native alluvium 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 0.5 inch.

10.2.4 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.

10.3 Modulus of Subgrade Reaction

For the subject project, design of the structures supported on compacted fill subgrade prepared in accordance with the recommendations provided in this report may be based on a soil modulus of subgrade reaction of (k_s) of 150 pounds per square inch per inch.

10.4 Lateral Earth Pressure

The proposed retaining walls are anticipated to be up to 17 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. 4, Lateral Earth Pressures for Retaining Wall Design

Backfill Slope (H:V)	Cantilever Wall Equivalent Fluid Pressure (pcf)	Restrained Wall Equivalent Fluid Pressure (psf)
Level	35 (triangular pressure distribution)	55 (triangular pressure distribution)

The recommended lateral pressures assume that the walls are fully back-drained 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 a minimum one (1) square feet per lineal feet of free draining, uniformly graded, ¾ -inch washed, crushed aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, rigid ABS (SDR-35) or PVC A-2000, 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 solid pipe outlets, with a maximum outlet spacing of 100 feet. Waterproofing membranes should be added to the subterranean wall levels for moisture sensitive areas to mitigate moisture migration through the walls.

In addition, walls with inclined backfill should be designed for an additional equivalent fluid pressure of one (1) pound per cubic foot for every two (2) degrees of slope inclination. 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. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Retaining walls taller than 6 feet should be designed to resist additional earth pressure caused by seismic ground shaking based on Section 1615A.1.6 of CBC 2010. A seismic earth pressure of 18H (psf), based on an inverted triangular distribution, can be used for design of wall.

10.5 Slabs-on-Grade

Slabs-on-grade should have a minimum thickness of four inches for support of nominal ground-floor live loads without hydrostatic uplift pressures. Minimum reinforcement for slabs-on-grade should be No. 3 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.

Slabs should be designed and constructed as promulgated by the American Concrete Institute (ACI) and the Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted. Care should be taken during concrete placement to avoid slab curling.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, slabs should be protected by at least a 10-mil-thick moisture barrier between the slab and compacted subgrade that meets the performance criteria of ASTM E 1745 Class A material. Polyethylene sheets should be overlapped a minimum of six inches, and should be taped or otherwise sealed.

10.6 Soil Corrosivity Evaluation

Based on our review of soil corrosivity test results (see Appendix B), the soluble sulfate concentration, pH, and chloride content are not in the corrosive range to concrete in accordance with the Caltrans Corrosive Guidelines (2012). The minimum saturated resistivity is not the corrosive range to ferrous metal. Protections of underground metal pipe are not anticipated. Since the soluble sulfate concentrations tested for this project are less than 2,000 ppm in the soil, mitigation measures to protect concrete in contact with the soils are not anticipated. Type I or II Portland Cement may be used for the construction of the foundations and slabs.

The test results presented herein are considered preliminary. Additional testing and evaluation of the as-graded soils is recommended. A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed.

10.7 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. 5, 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)
67	4	2.0	2.0	3.0
	5	2.5	3.0	4.0
	6	3.0	4.0	5.0
	7	4.0	4.5	6.5
	8	5.0	5.0	7.5
	9	6.0	5.5	8.5

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.

10.8 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. 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. 6, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
67	4.5	5.50
	5.0	5.75
	6.0	6.00
	7.0	6.25
	8.0	6.50
	9.0	6.75

The pavement sections presented in the table are based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,000 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 three (3) inches and one (1) inch, respectively.

Transverse contraction joints should not be spaced more than 15 feet and should be cut to a depth of ¼ 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.

All outside edges should conform to Section 201 of the Standard Specifications for Public Works Construction (SSPWC, latest edition), and should be constructed in accordance with Section 302-6 of the SSPWC. Pavement subgrade should be prepared in accordance with Section 9.7 of this report.

The PCCP materials should conform to Section 201 of the Specifications for Public Works Construction and should be constructed in accordance with Section 302-6 of the SSPWC.

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.

10.9 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the building 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 building foundation shall be sloped a minimum of 2 percent away from the building per 2013 CBC.

Non-erosive drainage control devices should be installed along the eastern slopes to prevent water from flowing over the slopes. Interceptor drains, terrace drains, down drains, catch basins, and other drainage devices should be kept clean of debris and maintained in good working order to provide adequate drainage for the slope areas.

Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the subgrade soils. Gutters and downspouts should be installed on the roof, and runoff should be directed to the storm drain through non-erosive devices. Lower level walkways and open patio areas may require special drainage provisions and sump pumps to provide suitable drainage.

11.0 CONSTRUCTION RECOMMENDATIONS

11.1 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 11.2, *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.

11.2 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 7, *Slope Ratios for Temporary Excavation*. 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. 7, Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	vertical
4 – 8	1:1
8 +	1.5:1

*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 retard 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 five (5) feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of six (6) feet of stockpiled soils placed at least five (5) feet from the trench edge.

For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

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.

If the excavation occurs near existing structures, special construction considerations would be required during excavation to protect these existing structures during construction. The proposed excavation should not cause loss of bearing and/or lateral supports of the existing structures.

11.3 Shoring Design

Temporary shoring may be required for the possible 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 a trench box (where feasible), or 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.

Drilled excavations for soldier piles, may require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation. Casing may be needed if granular earth material is located behind the existing retaining wall. Caving, raveling and sloughing of the alluvial sand materials may occur and should be anticipated during construction.

11.3.1 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 35 pounds per cubic foot (pcf) for non-surcharged condition. This pressure is valid only for shoring retaining level ground. This equivalent fluid pressure is valid only for shoring supporting level ground.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, 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 trench 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 for excavation up to 20 feet. Vertical skin friction against soldier piles for may be taken as 350 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 350 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 350 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).

11.3.2 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.

11.3.3 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. Tie-back installation and testing guidelines and procedures are presented in Appendix E, *Guide Specifications for Installation and Acceptance of Tie-back Anchors.* 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:

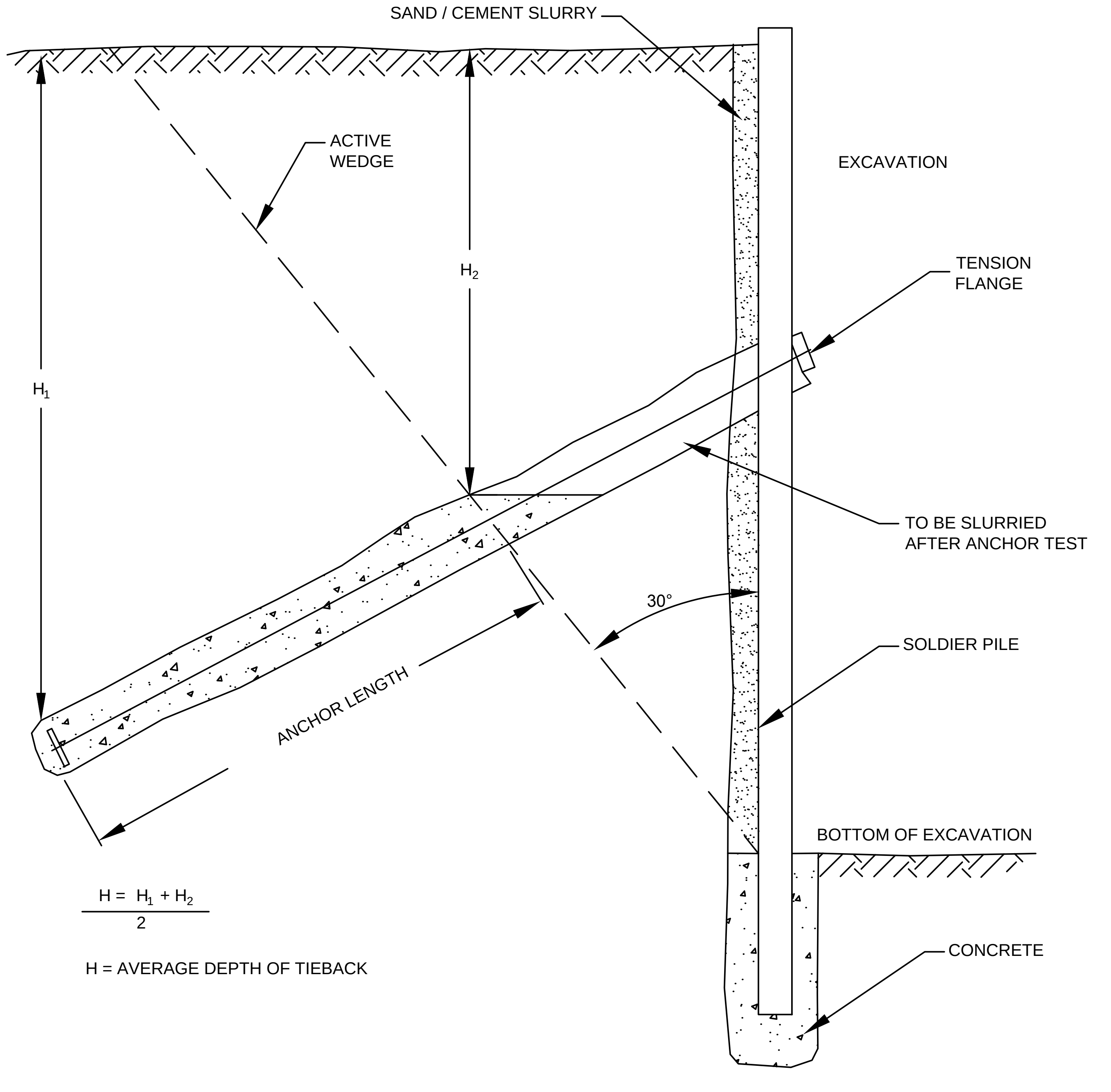
H = average depth of anchor below ground surface, shown on Drawing No. 7, *Schematic Tie-Back Design*

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.



$$H = \frac{H_1 + H_2}{2}$$

H = AVERAGE DEPTH OF TIEBACK

SCHEMATIC TIE-BACK DESIGN

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Drawing No.
7

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.

12.0 PLAN REVIEW AND CONSTRUCTION INSPECTION SERVICES

This report has been prepared to aid in evaluation of the site, to prepare site-grading recommendations, and to assist the civil/structural engineer in the design of the proposed developments. It is recommended that this office be provided the opportunity to provide final site grading and design recommendations once the final grading plan is available.

All site grading and earthwork should be completed under the observation and testing of a qualified geotechnical consultant to verify compliance with the recommendations set forth in this report. All ground surfaces should be examined and approved by the project geotechnical consultant prior to placing any fill and/or structure. All footing excavations should be observed prior to placement of steel and concrete to see that footings are founded on satisfactory soil and that excavations are free of loose, disturbed or deleterious materials.

13.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and site grading recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the final site grading and actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

14.0 REFERENCES

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APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance of the site and a subsurface exploration program consisting of drilling soil borings. During the site reconnaissance on March 14, 2016, the surface conditions were noted and the locations of the borings were determined. The borings were located using existing boundary features as a guide and should be considered accurate only to the degree implied by the method used.

Twelve (12) exploratory borings (BH-1 through BH-10 and PT-1 and PT-2) were drilled within the project sites on March 31, 2016 and on April 4 and 5, 2016. The borings were drilled using a limited access track drill rig and truck-mounted drill rig equipped with an 8-inch diameter hollow-stem auger for soil sampling. Soils were logged by a Converse engineer and classified in the field by visual examination in accordance with the Unified Soil Classification System. The field descriptions have been modified where appropriate to reflect the laboratory test results.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4-inches inside diameter and 3.0-inches outside diameter) lined with sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches, using an automatic hammer. Samples were retained in brass rings (2.4-inches inside diameter and 1.0-inch in height). The central portion of the sample was retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Blow counts for each sample interval are presented on the logs of borings. Bulk samples of typical soil types were also obtained.

Standard Penetration Tests (SPT) were also performed using a standard (1.4-inches inside diameter and 2.0-inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, failing 30 inches for each blow. The recorded blow counts for every six inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings in the “BLOWS" column. The standard penetration test was performed in accordance with the ASTM Standard D1586 test method. The soil retrieved from the spoon sampler was carefully sealed in waterproof plastic containers for shipment to the laboratory.

Borings PT-1 and PT-2 were utilized for percolation tests prior to backfill. Percolation test procedures and test results are further discussed in report section 7.0, *Percolation Testing* and Appendix C.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. 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 Drawing No. A-1, *Soil Classification Chart*. The logs of the exploratory boring are presented in Drawing Nos. A-2 through A-14, *Log of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

- STANDARD PENETRATION TEST
  Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE 2.42" I.D. sampler.
- DRIVE SAMPLE No recovery
- BULK SAMPLE
- GRAB SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

NOTE: 10-DCP BLOWS

LW=LIGHT WEIGHT
HW= HEAVY WEIGHT

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION	
Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h

STRENGTH	
Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

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Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	A-1

Project ID: 15-31-312-01.GPJ; Template: KEY

Log of Boring No. BH- 1

Dates Drilled: 3/21/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, with gravels up to 0.75" maximum dimension, brown.						ca,er,max
5		**ALLUVIUM (Qal):** **SAND (SP):** fine to medium-grained, with clay, trace silt, orangish brown.			15/17/22	6	114	ds
10					23/45/50(45")	6	112	
15		-fine to coarse-grained, trace silt, light brown			8/14/21			
20		-fine to medium-grained			18/30/47	2	100	
25		-fine-grained			9/22/30			
30					32/50(5")	2	102	

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Project Name
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FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01** Drawing No. **A-2a**

Log of Boring No. BH- 1

Dates Drilled: 3/21/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine to coarse-grained, trace silt, light brown.			12/19/23			
40		-brown			14/24/35	5	107	
45		-light brown			13/26/34			
50					34/50(4.5")	3	100	
		End of boring at 51.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 3-21-16.						

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Project Name
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Project No. **15-31-312-01** Drawing No. **A-2b**

Log of Boring No. BH- 2

Dates Drilled: 3/21/2016 | Logged by: MM | Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, few gravels up to 0.75" maximum dimension, brown.						ei
5		**ALLUVIUM (Qal):** **SAND (SP):** fine to medium-grained, few silt, brown.			14/19/23	6	113	c
10		-fine to medium-grained, with clay,			19/33/45	7	111	
15		-fine to coarse-grained, few silt, light brown			9/14/18			
20					18/36/50(5")	3	107	
25		-fine-grained			13/33/33			
30					33/47/50(3.5")	3	102	
		End of boring at 31.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 3-21-16.						

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Project No. 15-31-312-01 | Drawing No. A-3

Log of Boring No. BH- 3

Dates Drilled: 3/21/2016 | Logged by: MM | Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SAND (SP):** fine to medium-grained with silt, few gravels with 0.5" in maximum dimension, brown.						
					4/5/10	7	113	
5		-with debris large rock up to 4" maximum dimension, light brown			30/43/43	4	107	
		ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained, few clays, orangish brown.						
10					6/11/12			wa(fc=21%)
15		**SAND (SP):** fine to medium-grained, few clays, orangish brown.			22/44/50(4")	7	109	
20		-fine-grained, light brown			10/18/16			wa(fc=5%)
25					21/37/50(5")	2	104	
30					10/18/21			wa(fc=3%)

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REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **A-4a**

Log of Boring No. BH- 3

Dates Drilled: 3/21/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine-grained, trace silt, few clays, orangish brown.			21/31/46	2	102	
40					8/12/22			wa(fc=3%)
45					26/37/50(5")	2	101	
50					13/20/33			
		End of boring at 51.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 3-21-16.						

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SENIOR LIVING PROJECT
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FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **A-4b**

Log of Boring No. BH- 4

Dates Drilled: 4/4/2016 Logged by: WB Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**5" ASPHALT CONCRETE WITH NO BASE**						ca,er,max
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, brown to orangish brown.						
5			■		12/20/27	7	100	
10		**CLAYEY SAND (SC):** fine to medium-grained, dark brown.	■		2/5/10	6	109	c
		ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained with silt, with silt, brown to orangish brown.						
15			■		10/14/22	5	114	
20		-trace clay	☒		8/11/14			
25			■		37/50(5")	9	109	
30			☒		9/18/26			

Converse Consultants

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SENIOR LIVING PROJECT
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REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01** Drawing No. **A-5a**

Log of Boring No. BH- 4

Dates Drilled: 4/4/2016 Logged by: WB Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine to medium-grained with silt, with silt, brown to orangish brown.			23/50(5")	4	106	
40					13/25/37			
45					23/33/50(5")			
50		-fine-grained			16/22/23			
55					15/21/26			
60		-orangish brown to brown			13/19/25			
		End of boring at 61.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings and patched with asphalt on 4-4-16.						

Converse Consultants

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Project No. **15-31-312-01** Drawing No. **A-5b**

Log of Boring No. BH- 5

Dates Drilled: 4/4/2016 | Logged by: WB | Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blows/ft	Moisture (%)	Dry Unit Wt. (pcf)	Other
0	**4" ASPHALT CONCRETE WITH NO BASE**						ei
	FILL (Af): **SILTY SAND (SM):** fine to medium-grained, with gravel up to 1" in maximum dimension, brown to orangish brown.		X				
		X	X	20/32/44	3	113	
5		X		36/49/50(5")	6	130	
10	**CLAYEY SAND TO SANDY CLAY (SC-CL):** fine to medium-grained, dark brown.	X		9/12/14	6	114	
	ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained with silt, trace clay, light brown.						
15		X		7/10/12			
20	-brown to orangish/reddish brown	X		30/49/50(5")	6	118	
25	-light brown to orangish brown	X		10/18/21			
30	-fine-grained, light brown	X		29/35/50(5")	2	104	

End of boring at 31.5 feet.
Groundwater not encountered during drilling.
Borehole backfilled with soil cuttings and patched with asphalt on 4/4/16.

Converse Consultants

Project Name: SENIOR LIVING PROJECT, 514 NORTH PROSPECT AVENUE, REDONDO BEACH, CALIFORNIA, FOR: BEACH CITIES HEALTH DISTRICT

Project No.: 15-31-312-01 | Drawing No.: A-6

Log of Boring No. BH- 6

Dates Drilled: 4/4/2016 Logged by: WB Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT WITH NO BASE						r
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, brown.				4	110	
5		**ALLUVIUM (Qal):** **SAND (SP):** fine to medium-grained with silt, trace clay, brown to orangish brown.			7/12/16	7	110	ds
10					15/25/34			wa(fc=6%)
15					21/37/50	7	111	
20		-light brown			10/15/17			wa(fc=7%)
25		-fine-grained			15/30/50(5")	2	104	
30		-fine to medium-grained			12/16/24			wa(fc=9%)

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. 15-31-312-01

Drawing No. A-7a

Log of Boring No. BH- 6

Dates Drilled: 4/4/2016 | Logged by: WB | Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND WITH SILT (SP-SM): fine to medium-grained, trace clay, brown to orangish brown.			27/50(5")	3	104	
40					15/27/31			wa(fc=8%)
45					23/49/50(4")	2	111	
50					16/26/34			
55		-light brown to brown			25/45/47			
60					21/40/42			

End of boring at 61.5 feet.
Groundwater not encountered during drilling.
Borehole backfilled with soil cuttings and patched with asphalt on 4-4-16.

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **A-7b**

Log of Boring No. BH- 7

Dates Drilled: 4/4/2016 Logged by: WB Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT CONCRETE WITH NO BASE						ma(fc=6%)
		FILL (Af): **SAND (SP):** fine to medium-grained, trace silt, orangish brown to brown.						
		ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained, trace clay, orangish brown to brown.						
5					10/22/39	7	119	
10					15/25/45	7	110	
15					12/17/18			
20					27/50(5")	4	101	
25		-fine-grained, light brown			11/13/14			
30		-fine to medium-grained			25/37/50(5")	2	106	

Converse Consultants

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Project No. **15-31-312-01** Drawing No. **A-8a**

Log of Boring No. BH- 7

Dates Drilled: 4/4/2016 Logged by: WB Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine to medium-grained, trace clay, orangish brown to brown.	X		10/13/21			
		End of boring at 35.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings and patched with asphalt on 4-4-16.						

Converse Consultants

Project Name
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Project No. **15-31-312-01**

Drawing No. **A-8b**

Log of Boring No. BH- 8

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT CONCRETE WITH 4" BASE						r
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, brown.						
		ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained, few silt, brown.						
5					9/15/20	4	113	ds
10		-fine to coarse-grained, trace silt, brown			20/30/36	2	114	
15		-fine to medium-grained, light brown			6/9/13			wa(fc=6%)
20		-medium to coarse-grained, light brown to orangish brown			19/34/50	4	108	
25					8/13/16			wa(fc=4%)
30					11/21/34	2	95	

Converse Consultants

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REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. 15-31-312-01

Drawing No. A-9a

Project ID: 15-31-312-01.GPJ; Template: LOG

Log of Boring No. BH- 8

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine to medium-grained, few silt, brown.			10/15/18			wa(fc=4%)
40		-fine-grained, trace silt			27/50(5")	4	100	
45					14/20/28			wa(c=7%)
50					41/50(3")	5	100	
55					25/46/50(5")			
60					29/45/50(3")	5	103	
		End of boring at 61.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings and patched with asphalt on 4-5-16.						

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
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FOR: BEACH CITIES HEALTH DISTRICT

Project No. 15-31-312-01

Drawing No. A-9b

Log of Boring No. BH- 9

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT WITH 3" BASE						ei,ca,er
		FILL (Af): **SAND (SP):** fine to medium-grained, with silt, orangish brown.		X				
5		**SILTY SAND (SM):** fine to medium-grained, few gravels up to 0.75" in maximum dimension, dark brown.	X		40/50(5")	2	125	
10			X		11/13/16	3	118	
		ALLUVIUM (Qal): **SAND (SP):** fine to medium-grained, light brown.						
15			X		11/20/24	2	110	ds
20		-fine to coarse-grained, light brown/orangish brown	X		15/26/32	2	104	
25			X		14/19/30			
30			X		37/50(5")	6	105	

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Project No. **15-31-312-01**

Drawing No. **A-10a**

Log of Boring No. BH- 9

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SAND (SP): fine to medium-grained, light brown.			17/22/31			
40		-fine to coarse-grained			44/25(5")	2	107	
45					16/33/40			
50		-fine to medium-grained			24/50(5")	2	102	
55		-with silt, brown			14/20/28			
60		-light brown			33/50(5")	3	96	
		End of boring at 61.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings and patched with asphalt on 4-5-16.						

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SENIOR LIVING PROJECT
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Project No. 15-31-312-01

Drawing No. A-10b

Log of Boring No. BH-10

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" TOPSOIL						
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, with some organics (roots), brown.			11/27/29	2	114	
5		**SAND (SP):** fine to coarse-grained, with silt, few organics, brown.			11/18/29	3	111	
10		**ALLUVIUM (Qal):** **SAND (SP):** fine to coarse-grained, brown.			17/18/29	2	112	max
15		-light brown			10/11/12			
20		-orange brown			33/49/50(4")	8	113	
25					23/37/37			
30					34/50(4")	3	99	
		End of boring at 31.5 feet. Groundwater not encountered during drilling. Borehole backfilled with soil cuttings on 4-5-16.						

Converse Consultants

Project Name
SENIOR LIVING PROJECT
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REDONDO BEACH, CALIFORNIA
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Project No. **15-31-312-01** Drawing No. **A-11**

Log of Boring No. PT- 1

Dates Drilled: 3/21/2016 | Logged by: MM | Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" GRAVEL **FILL (Af):** **SILTY SAND (SM):** fine to coarse-grained, with silt, few gravels up to 0.75" in maximum dimension, brown.						ma (fc=16%)
					12/14/18	5	110	
5		**ALLUVIUM (Qal):** **SAND (SP):** medium to coarse-grained, light brown.			17/25/31	3	107	
10					14/23/28	2	107	
		End of boring at 11.5 feet. Groundwater not encountered during drilling. Borehole utilized for percolation testing. Borehole backfilled with gravel and soil cuttings on 3-21-16.						

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **A-11**

Log of Boring No. PT- 2

Dates Drilled: 4/5/2016 Logged by: MM Checked By: SKS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): N/A Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FT	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" TOPSOIL						
		FILL (Af): **SILTY SAND (SM):** fine to medium-grained, with some organics, brown.						
5		**ALLUVIUM (Qal):** **SAND (SP):** fine to medium-grained, few silt, brown.	■		5/15/21	2	109	
			■		35/38/39	2	110	
10		End of boring at 10 feet. Groundwater not encountered during drilling. Borehole utilized for percolation testing. Borehole backfilled with gravel and soil cuttings on 4-5-16.						

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 amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

B1.1 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. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

B1.2 Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on two (2) selected samples. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

B1.3 Percent Finer than Sieve No. 200

The percent finer than sieve No. 200 tests were performed on twelve (12) representative soil samples to aid in the classification of the on-site soils and to estimate other engineering parameters. Testing was performed in general accordance with the ASTM Standard D1140 test method. Test results are presented in the Logs of Borings in Appendix A, *Field Exploration*.

B1.4 Maximum Dry Density Test

Three (3) laboratory maximum dry density-moisture content relationship tests were performed on representative bulk samples. The tests were conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2, *Moisture-Density Relationship Results*.

B1.5 Direct Shear

Direct shear tests were performed on three (3) relatively undisturbed samples at soaked moisture conditions. For each test, 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 samples were 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.

Residual shear test was performed on one (1) relatively undisturbed sample at soaked moisture conditions. For each test, 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 samples were then sheared at a constant strain rate of 0.005 inch/minute. Shear deformation was recorded until a maximum of about 0.30-inch shear displacement was achieved. This process was repeated for 5 passes of shearing for each sample. Residual strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters.

For test data, including sample density and moisture content, see Drawing Nos. B-3a through B-3d, *Direct Shear Test Results,* and in the following table:

Table No. B-1, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1	5	Sand (SP)	30	60
BH-6	5	Sand (SP)	28	20
BH-8	5	Sand (SP)	28	40
BH-9	15**	Sand (SP)	30	40

*Residual Shear

B1.6 Consolidation Test

Consolidation tests were performed on two (2) selected samples. Data obtained from this test performed on a relatively undisturbed soil sample was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the one-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 state of equilibrium. 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 Drawing Nos. B-4a and B-4b, *Consolidation Test Results*.

B1.7 Expansion Index Test

Three (3) representative bulk samples were tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM D4829 Standard. Test results are presented in the following table:

Table No. B-2, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-1	0-5	Sand (SP)	1	Very Low
BH-5	0-5	Sand (SP)	0	Very Low
BH-9	0-5	Sand (SP)	1	Very Low

B1.8 R-value Test

Two (2) representative bulk soil samples were tested for resistance value (R-value) in accordance with State of California Standard Method 301-G. 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-3, R-value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-6	1-5	Sand (SP)	70
BH-8	0-5	Sand (SP)	67

B1.9 Soil Corrosivity

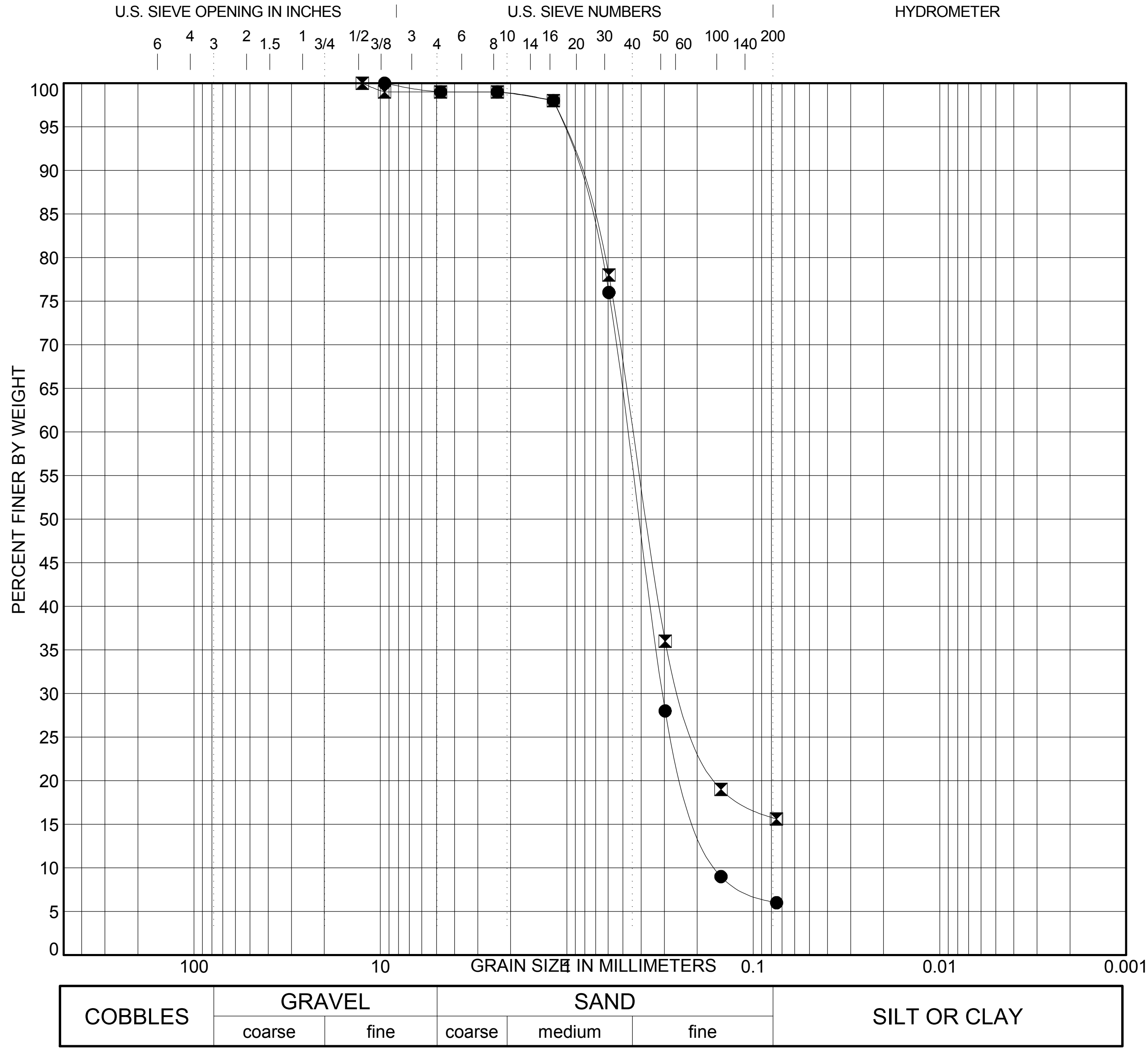
Three (3) representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The test results received from EGL are included in the following table:

Table No. B-4, 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	7.81	115	0.002	12,000
BH-4	1-5	7.79	145	0.005	6,600
BH-9	1-5	7.29	150	0.035	7,100

B1.10 Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.



	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH- 7**	**0-5**	**SAND (SP)**				**1.28**	**3.05**
⧗	**PT- 1**	**0-5**	**SILTY SAND (SM)**					

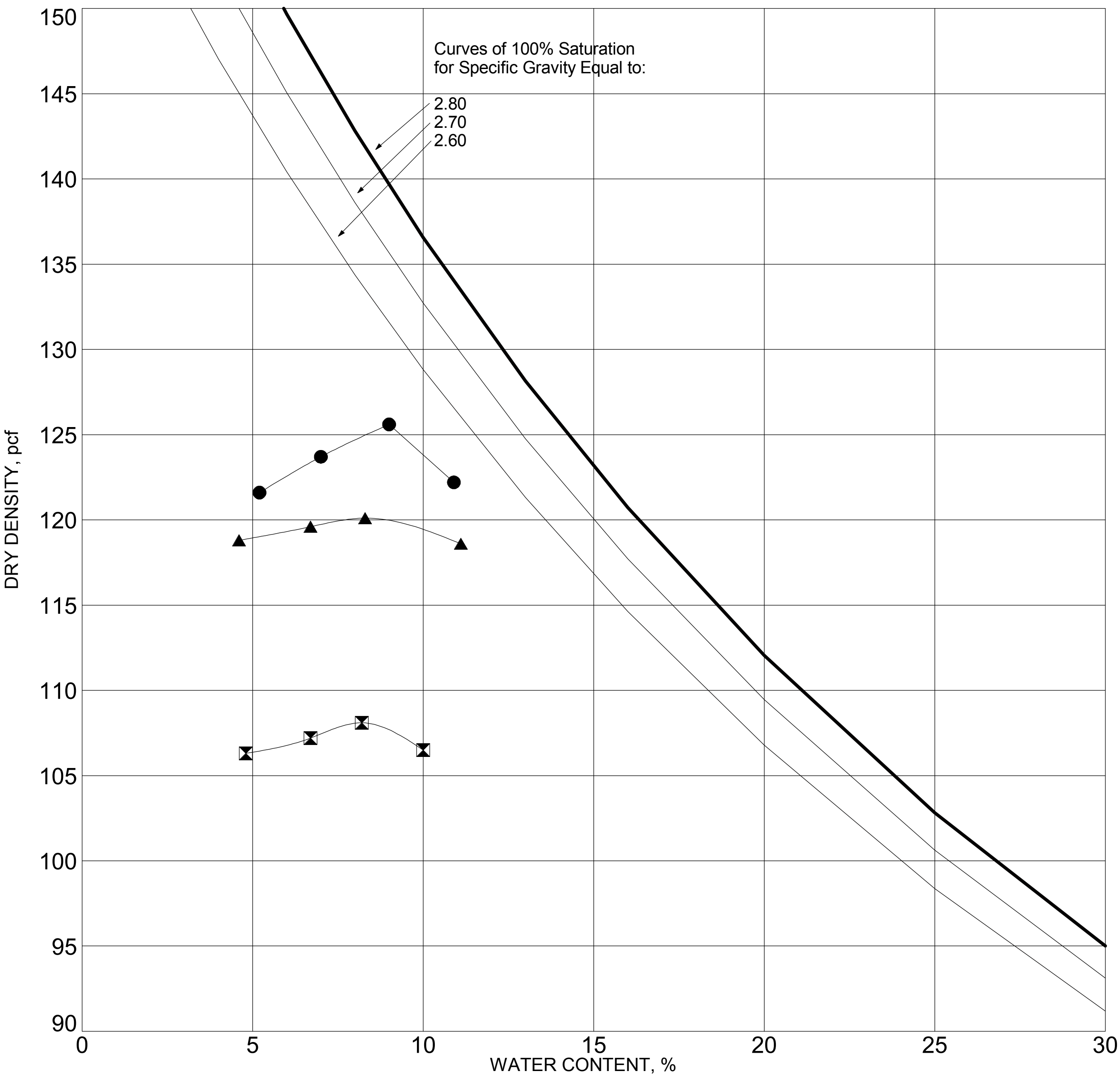
	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH- 7**	**0-5**	**9.5**	**0.472**	**0.306**	**0.155**	**1.0**	**93.0**	**6.0**	
⧗	**PT- 1**	**0-5**	**12.5**	**0.442**	**0.233**		**1.0**	**83.4**	**15.6**	

GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
SENIOR LIVING PROJECT **514 NORTH PROSPECT AVENUE** **REDONDO BEACH, CALIFORNIA** **FOR: BEACH CITIES HEALTH DISTRICT**	**15-31-312-01**	**B-1**

Project ID: 15-31-312-01.GPJ; Template: GRAIN SIZE



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH- 1	0-5	SILTY SAND (SM)	D1557 Method B	9	125.6
⧗	BH- 4	0-5	SILTY SAND (SM)	D1557 Method B	8	108.4
▲	BH-10	1-6	SILTY SAND (SM)	D1557 Method B	7.7	120.2

NOTE:

MOISTURE-DENSITY RELATIONSHIP RESULTS

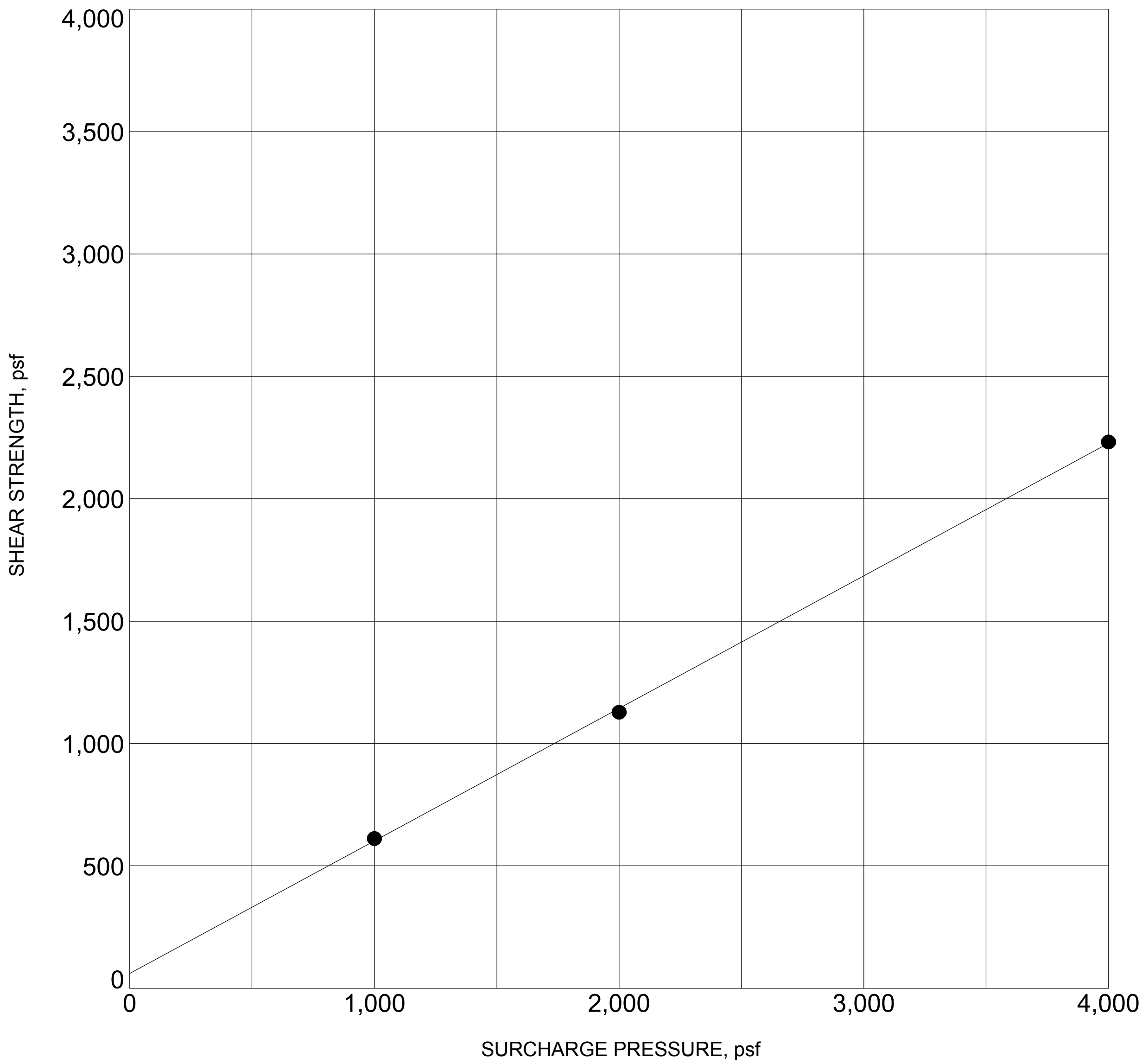
Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.
15-31-312-01

Drawing No.
B-2

Project ID: 15-31-312-01.GPJ; Template: COMPACTION



BORING NO. :	**BH- 1**	DEPTH (ft) :	**5**
DESCRIPTION :	**SAND (SP)**		
COHESION (psf) :	**60**	FRICTION ANGLE (degrees):	**30**
MOISTURE CONTENT (%) :	**6.3**	DRY DENSITY (pcf) :	**114**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

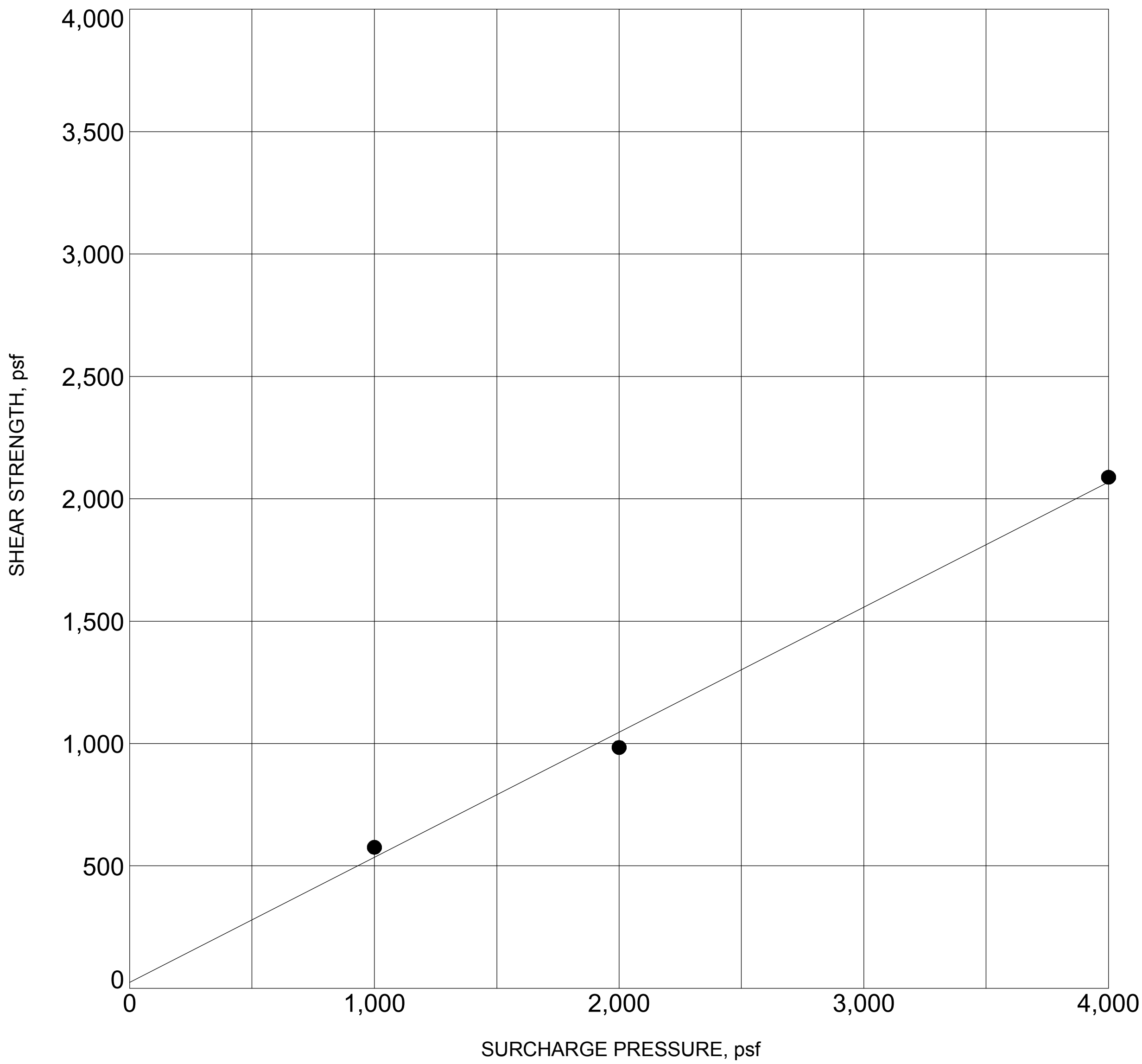
Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **B-3a**

Project ID: 15-31-312-01.GPJ; Template: DIRECT SHEAR



BORING NO. :	**BH- 6**	DEPTH (ft) :	**5**
DESCRIPTION :	**SAND (SP)**		
COHESION (psf) :	**20**	FRICTION ANGLE (degrees):	**28**
MOISTURE CONTENT (%) :	**7.0**	DRY DENSITY (pcf) :	**110.0**

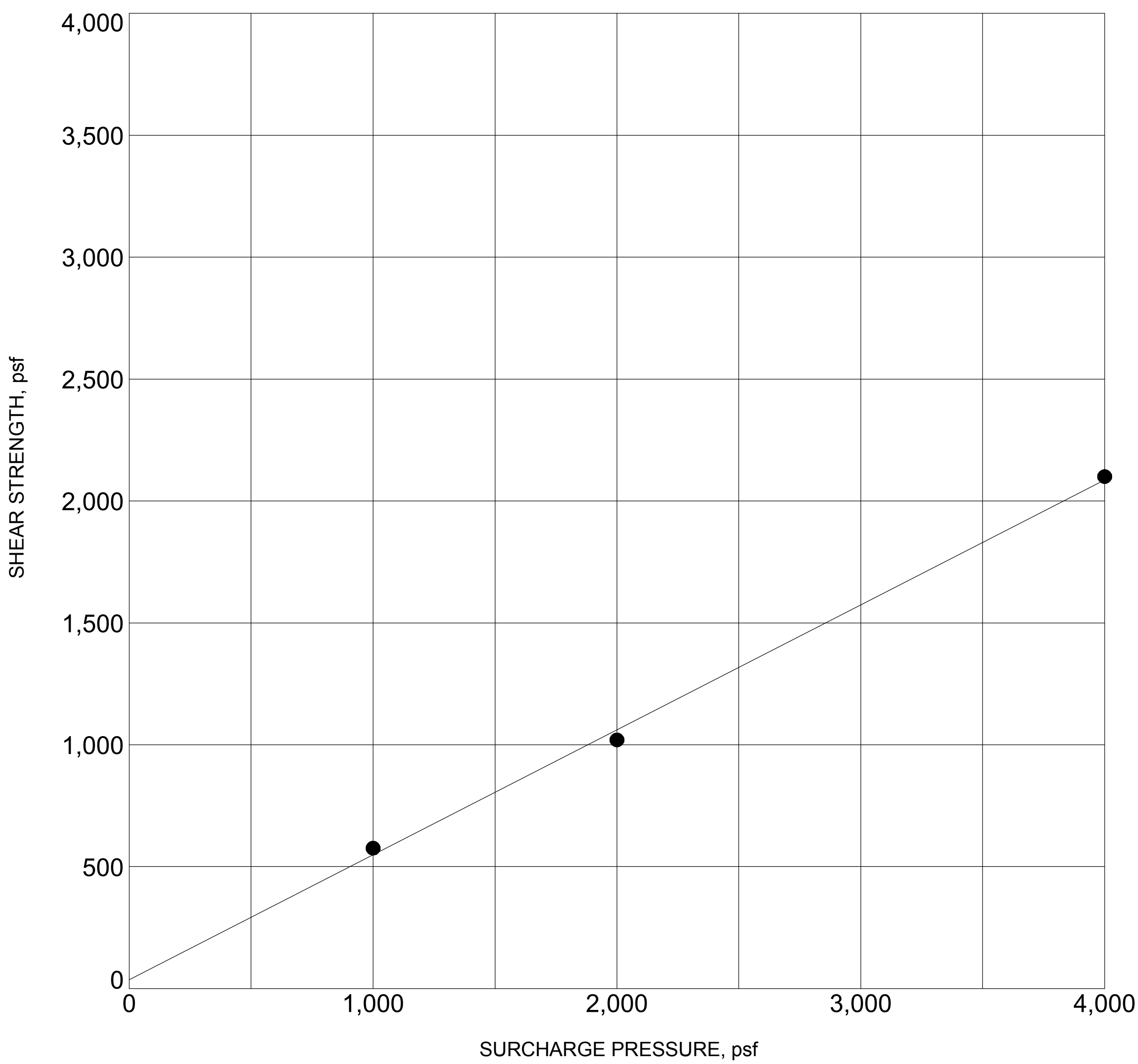
NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
SENIOR LIVING PROJECT **514 NORTH PROSPECT AVENUE** **REDONDO BEACH, CALIFORNIA** **FOR: BEACH CITIES HEALTH DISTRICT**	**15-31-312-01**	**B-3b**

Project ID: 15-31-312-01.GPJ; Template: DIRECT SHEAR



BORING NO. :	BH- 8	DEPTH (ft) :	5
DESCRIPTION :	SAND (SP)		
COHESION (psf) :	40	FRICTION ANGLE (degrees):	28
MOISTURE CONTENT (%) :	4.0	DRY DENSITY (pcf) :	113.0

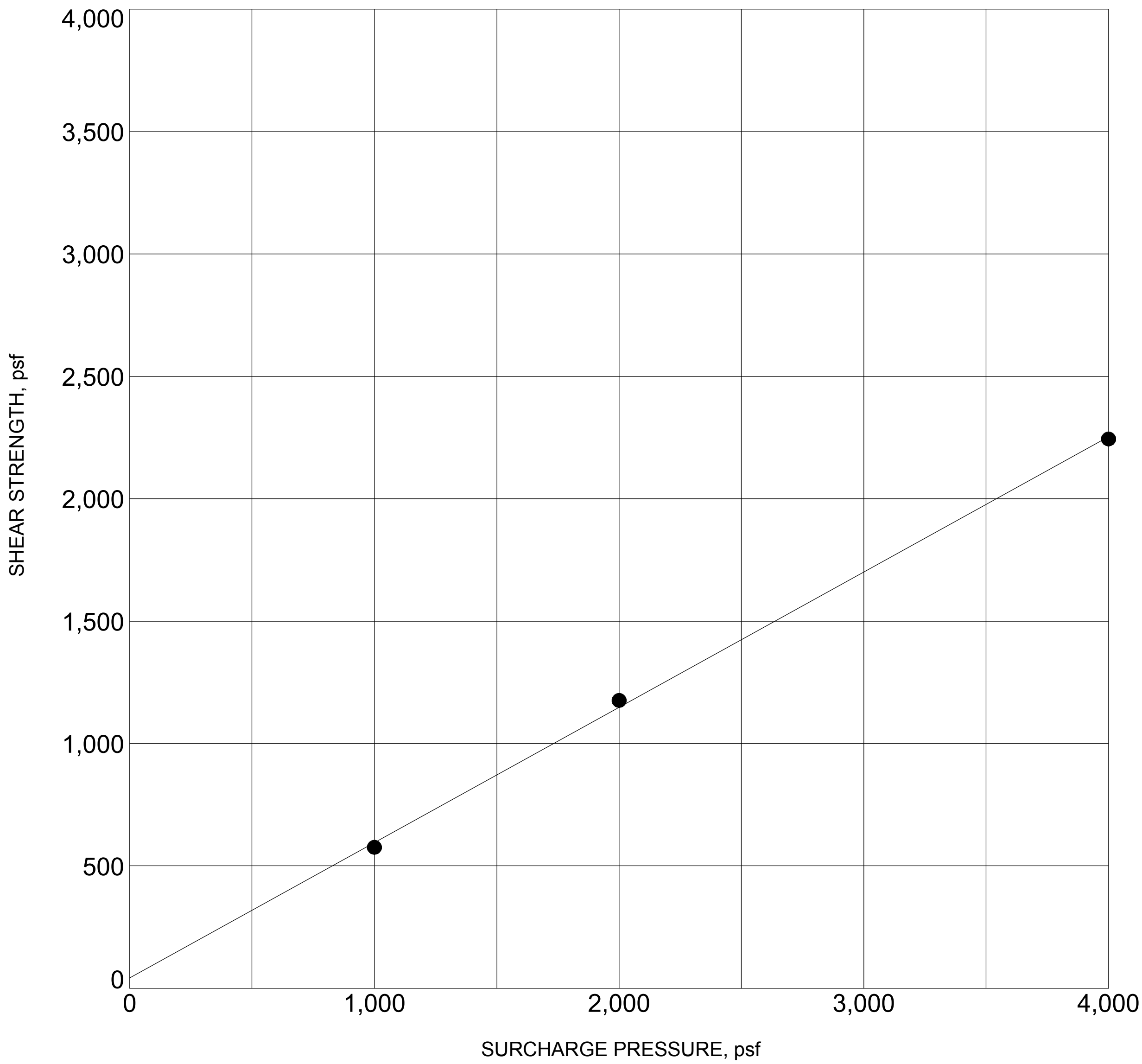
NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Drawing No.
SENIOR LIVING PROJECT **514 NORTH PROSPECT AVENUE** **REDONDO BEACH, CALIFORNIA** **FOR: BEACH CITIES HEALTH DISTRICT**	**15-31-312-01**	**B-3c**

Project ID: 15-31-312-01.GPJ; Template: DIRECT SHEAR



BORING NO.	:	**BH- 9**	DEPTH (ft)	:	**15**
DESCRIPTION	:	**SAND (SP)**			
COHESION (psf)	:	**40**	FRICTION ANGLE (degrees):		**30**
MOISTURE CONTENT (%)	:	**2.0**	DRY DENSITY (pcf)	:	**110.0**

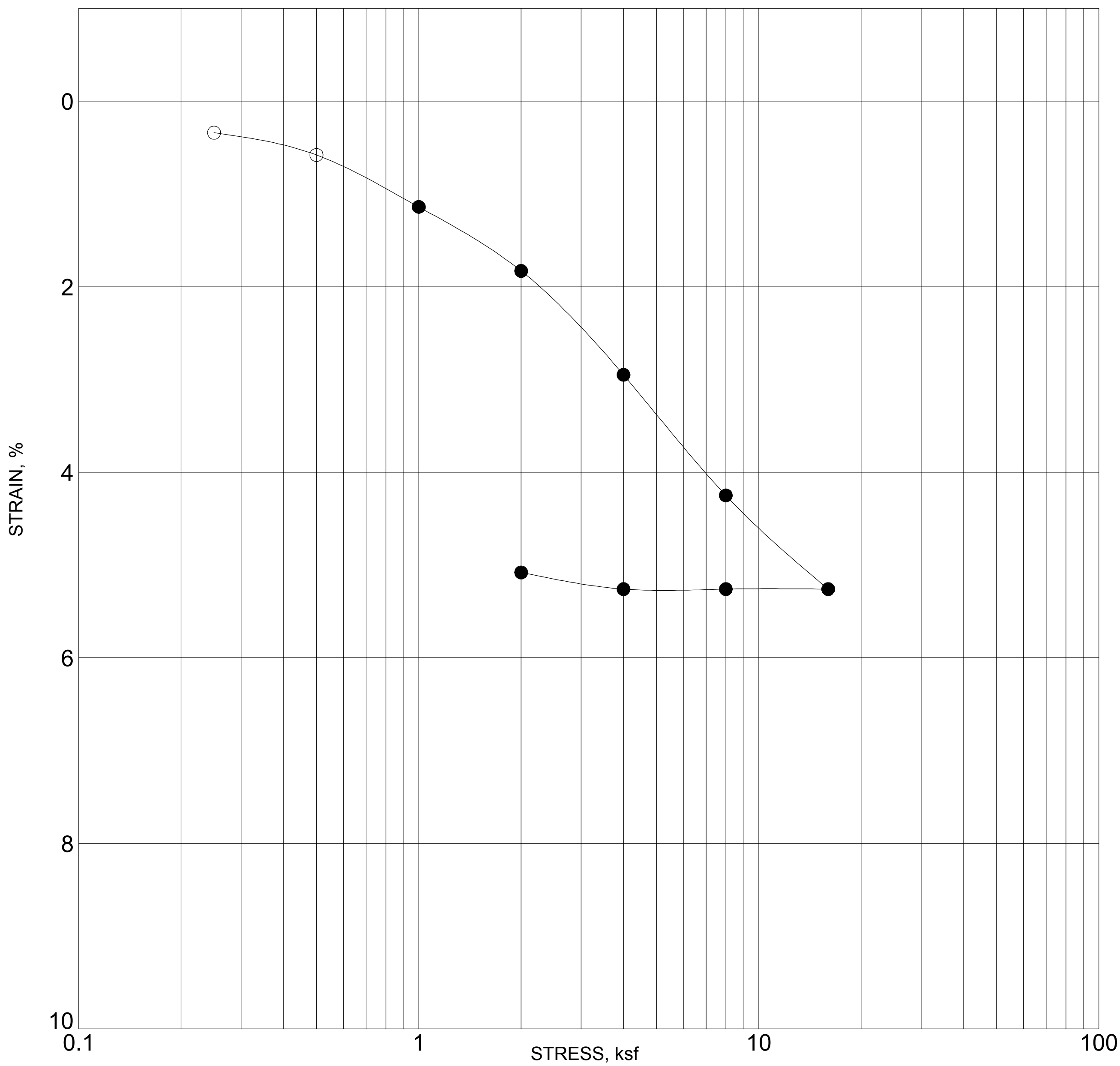
NOTE: Residual Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No.	Drawing No.
15-31-312-01	**B-3d**



BORING NO. :	BH- 2	DEPTH (ft) :	5
DESCRIPTION :	SAND (SP)		
MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL 6	113		

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

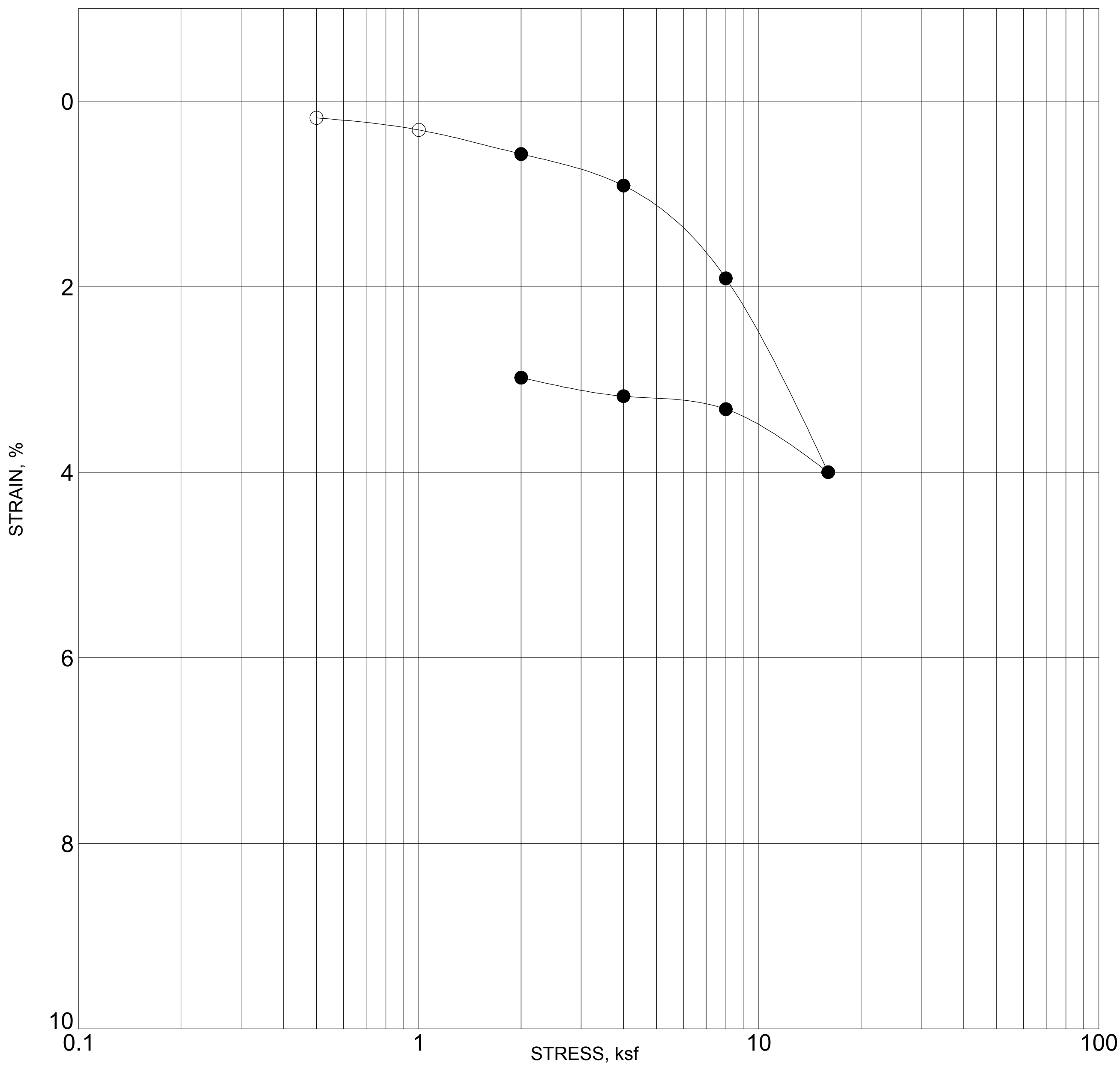
Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **B-4a**

Project ID: 15-31-312-01.GPJ; Template: CONSOLIDATION



BORING NO. :	BH- 4	DEPTH (ft) :	10
DESCRIPTION :	CLAYEY SAND (SC)		

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL 6	109		

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
SENIOR LIVING PROJECT
514 NORTH PROSPECT AVENUE
REDONDO BEACH, CALIFORNIA
FOR: BEACH CITIES HEALTH DISTRICT

Project No. **15-31-312-01**

Drawing No. **B-4b**

APPENDIX C

PERCOLATION TESTING

APPENDIX C

PERCOLATION TESTING

Percolation testing was performed utilizing exploratory borings PT-1 and PT-2 on March 21 and April 5, 2016. The continuous pre-soak falling-head test method for water percolation testing was utilized to evaluate soil infiltration rates of the fill and native soils encountered between depths of 0 to 10 feet below the ground surface at the respective boring locations in accordance with LA County Low Impact Development, Best Management Practices Guidelines. The test locations were prepared by placing a perforated 2-inch diameter PVC pipe surrounded by pea gravel after drilling and sampling. Water was filled to the ground surface to pre-soak prior to testing.

The boring was cased using a two-inch diameter perforated casing. Water was added to the bore hole until the water level was as near the ground surface as could be achieved, and allowed to pre-soak for at least 24 hours. After pre-soak, water was added to the bore hole until the water level was as near the ground surface as could be achieved. The water level was measured to the nearest 1/100-foot and recorded every 10 minutes for 30 minutes. There were four (4) sets of measurements taken for each test and each set consisted of at least three (3) measurements (10 minute intervals). The results of the percolation tests are tabulated below.

Table No. C-1, Percolation Test Results

Boring No.	Depth of Boring* (feet)	Predominant Soil Types (USCS)	Average Percolation Rate (inches/hour)	Lowest Percolation Rate (inches/hour)
PT-1	10	Sand (SP)	13.24	4.18
PT-2	10	Silty Sand (SM) over Sand (SP)	4.84	3.08

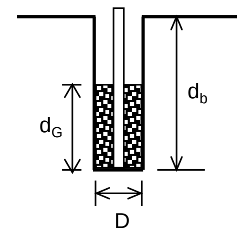
*Approximate

Based on our review of percolation rates, the site soil has fair percolation rates for infiltration systems in general. In accordance with County of Los Angeles requirements, the minimum percolation rate for design of infiltration system for storm water management is 0.3 inch per hour. The project Civil Engineer should review the raw data of percolation test presented herein to determine specific soil layers and percolation rates for design of the proposed infiltration system. Such systems should be constructed a minimum distance of 10 feet laterally from any existing or future planned building or subsurface structure as not to disturb or undermine foundations. The percolation rates were determined in general accordance with Los Angeles County guidelines. The detailed percolation test results are shown on the following data sheet.

Percolation Testing

Job Name: Beach Cities Health District - Senior Living Project
Job No.: 15-31-312-01
Location: PT-1
Test Date: 3.21.16

Test Boring No. **PT-1**
Depth of Boring (d_b): 10.0 feet
Diameter of Boring (D): 0.67 feet
Test Performer: M. Malim



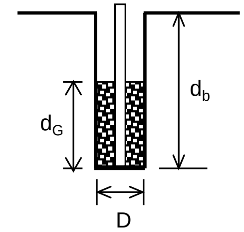
Time of Testing			Water Level Measurement		Water Level Calculations				Percolation Rate Calculations		
Initial Time T_i	Final Time T_f	Time Interval ΔT (hr)	Initial depth to water d_1 (feet)	Final depth to water d_2 (feet)	Initial Height of water column d_i (feet)	Final Height of water column d_f (feet)	Drop in Height $\Delta d = d_i - d_f$ (feet)	Average height of water column L_{ave} (feet)	Pre-adjusted Percolation Rate $k_i = \Delta d / \Delta T$ (inch/hr)	Reduction Factor R_f	Adjusted Percolation Rate $k = k_i / R_f$ (inch/hr)
Presoak	3/21/2016	2									
Percolation Test											
10:15:00 AM	10:25:00 AM	0.17	0.00	6.70	10.00	3.30	6.70	6.65	482.40	20.9	23.14
10:25:00 AM	10:35:00 AM	0.17	6.70	7.80	3.30	2.20	1.10	2.75	79.20	9.2	8.60
10:35:00 AM	10:45:00 AM	0.17	7.80	8.90	2.20	1.10	1.10	1.65	79.20	5.9	13.37
10:45:00 AM	10:55:00 AM	0.17	8.90	9.50	1.10	0.50	0.60	0.80	43.20	3.4	12.75
10:55:00 AM	11:05:00 AM	0.17	9.50	10.00	0.50	0.00	0.50	0.25	36.00	1.7	20.62
12:07:00 PM	12:17:00 PM	0.17	0.00	6.50	10.00	3.50	6.50	6.75	468.00	21.1	22.13
12:17:00 PM	12:27:00 PM	0.17	6.50	7.70	3.50	2.30	1.20	2.90	86.40	9.7	8.95
12:27:00 PM	12:37:00 PM	0.17	7.70	8.30	2.30	1.70	0.60	2.00	43.20	7.0	6.20
2:05:00 PM	2:15:00 PM	0.17	0.00	5.80	10.00	4.20	5.80	7.10	417.60	22.2	18.82
2:15:00 PM	2:25:00 PM	0.17	5.80	7.10	4.20	2.90	1.30	3.55	93.60	11.6	8.07
2:25:00 PM	2:35:00 PM	0.17	7.10	8.30	2.90	1.70	1.20	2.30	86.40	7.9	10.98
2:37:00 PM	2:47:00 PM	0.17	0.00	5.90	10.00	4.10	5.90	7.05	424.80	22.0	19.27
2:47:00 PM	2:57:00 PM	0.17	5.90	7.20	4.10	2.80	1.30	3.45	93.60	11.3	8.28
2:57:00 PM	3:07:00 PM	0.17	7.20	7.70	2.80	2.30	0.50	2.55	36.00	8.6	4.18

Note: Reduction Factor, $R_f = (2*d_i - \Delta d)/D + 1$

Lowest Percolation Rate = 4.18 inch/hr
Average Percolation Rate = 13.24 inch/hr

Reference: Los Angeles County (2014). Adminstrative Manual - Low Impact Development Best Management Practice Guideline for Design, Investigation, and Reporting, 12/31/14.

Percolation Testing



Job Name: Beach Cities Health District - Senior Living Project
Job No.: 15-31-312-01
Location: PT-2
Test Date: 4.5.16

Test Boring No. **PT-2**
Depth of Boring (d_b): 10.0 feet
Diameter of Boring (D): 0.67 feet
Test Performer: M. Malim

Time of Testing			Water Level Measurement		Water Level Calculations				Percolation Rate Calculations		
Initial Time	Final Time	Time Interval	Initial depth to water	Final depth to water	Initial Height of water column	Final Height of water column	Drop in Height	Average height of water column	Pre-adjusted Percolation Rate	Reduction Factor	Adjusted Percolation Rate
T_i	T_f	ΔT	d_1	d_2	d_i	d_f	$\Delta d = d_i - d_f$	L_{ave}	$k_i = \Delta d / \Delta T$	R_f	$k = k_i / R_f$
		(hr)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(inch/hr)		(inch/hr)
Presoak	4/5/2016	2									
Percolation Test											
9:16:00 AM	9:26:00 AM	0.17	2.50	5.50	7.50	4.50	3.00	6.00	216.00	18.9	11.42
9:26:00 AM	9:36:00 AM	0.17	5.50	6.30	4.50	3.70	0.80	4.10	57.60	13.2	4.35
9:36:00 AM	9:46:00 AM	0.17	6.30	7.00	3.70	3.00	0.70	3.35	50.40	11.0	4.58
9:46:00 AM	9:56:00 AM	0.17	7.00	7.40	3.00	2.60	0.40	2.80	28.80	9.4	3.08
9:56:00 AM	10:06:00 AM	0.17	7.40	7.80	2.60	2.20	0.40	2.40	28.80	8.2	3.53
10:06:00 AM	10:16:00 AM	0.17	7.80	8.30	2.20	1.70	0.50	1.95	36.00	6.8	5.28
10:25:00 AM	10:35:00 AM	0.17	2.80	5.10	7.20	4.90	2.30	6.05	165.60	19.1	8.69
10:35:00 AM	10:45:00 AM	0.17	5.10	5.80	4.90	4.20	0.70	4.55	50.40	14.6	3.46
10:45:00 AM	10:55:00 AM	0.17	5.80	6.40	4.20	3.60	0.60	3.90	43.20	12.6	3.42
10:56:00 AM	11:06:00 AM	0.17	3.50	4.80	6.50	5.20	1.30	5.85	93.60	18.5	5.07
11:06:00 AM	11:16:00 AM	0.17	4.80	5.60	5.20	4.40	0.80	4.80	57.60	15.3	3.76
11:16:00 AM	11:26:00 AM	0.17	5.60	6.30	4.40	3.70	0.70	4.05	50.40	13.1	3.85
2:50:00 PM	3:00:00 PM	0.17	4.50	5.50	5.50	4.50	1.00	5.00	72.00	15.9	4.52
3:10:00 PM	3:20:00 PM	0.17	5.50	6.20	4.50	3.80	0.70	4.15	50.40	13.4	3.76
3:20:00 PM	3:30:00 PM	0.17	6.20	6.80	3.80	3.20	0.60	3.50	43.20	11.4	3.77

Note: Reduction Factor, $R_f = (2*d_i - \Delta d)/D + 1$

Lowest Percolation Rate = 3.08 inch/hr
Average Percolation Rate = 4.84 inch/hr

Reference: Los Angeles County (2014). Adminstrative Manual - Low Impact Development Best Management Practice Guideline for Design, Investigation, and Reporting, 12/31/14.

APPENDIX D

EARTHWORK SPECIFICATIONS

APPENDIX D

EARTHWORK SPECIFICATIONS

D1.1 Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good, workmanlike manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fill
- Observation and Testing

D1.2 Site Inspection

1. The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.

2. This *Geotechnical Study Report* by Converse Consultants may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

D1.3 Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.

2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.

3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

D1.4 Site Clearing

1. Clearing and grubbing shall consist of the removal from building areas to be graded of all existing structures, pavement, utilities, and vegetation.

2. Organic and inorganic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

D1.5 Excavations

1. Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

D1.6 Preparation of Fill Areas

1. All organic material, organic soils, incompetent alluvium, undocumented fill soils and debris should be removed from the proposed building areas.

2. In order to provide a relative uniform bearing material below shallow foundations, over-excavation and re-compaction of below the foundations and slab-on-grade are recommended. We recommend a minimum 3 feet of onsite soils below the bottom of foundations should be removed, moisture-conditioned if necessary, and replaced as compacted fill. At least the six (6) inches of soil at bottom of over-excavation, cut and transition areas should be scarified and compacted. All undocumented fill should be removed and replaced with compacted fill. The excavation to remove unsuitable soils should be extended to five (5) feet beyond the building limits and appendages where space is available. All loose, soft or disturbed earth materials should be removed from the bottom of excavations

before placing structural fill. The actual depth of removal should be determined based on observations made during grading. After the required removals have been made, the exposed native earth materials shall be excavated to provide a zone of structural fill for the support of footings, slabs-on-grade, and exterior flatwork. The fill thickness under structures should not vary.

3. The subgrade in all areas to receive fill shall be scarified to a minimum depth of six (6) inches, the soil moisture adjusted within three (3) percent of the optimum moisture for granular soils and at above approximately three (3) percent of the optimum moisture for fine-grained soils, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. Scarification may be terminated on moderately hard to hard, cemented earth materials with the approval of the Geotechnical Engineer.

4. Compacted fill may be placed on native soils that have been properly scarified and recompacted as discussed above.

5. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

D1.7 Placement and Compaction of Fill

1. Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, and driveways will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.

2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

   a. All fill soil particles shall not exceed three (3) inches in nominal size, and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

   b. Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) at about three (3) percent above optimum moisture for fine grained soils, and within three (3) percent of optimum for granular soils.

3. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

4. All fill placed at the site shall be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The on-site soils shall be moisture conditioned within three (3) percent of the optimum moisture for granular soils and at above approximately three (3) percent of the optimum moisture for fine-grained soils. At least the upper 12 inches of subgrade soils underneath the concrete apron, pavement and parking areas should be compacted to a minimum of 95 percent relative compaction.

5. Fill exceeding five (5) feet in height shall not be placed on native slopes that are steeper than 5:1 horizontal:vertical (H:V). Where native slopes are steeper than 5:1 H:V, and the height of the fill is greater than five (5) feet, the fill shall be benched into competent materials. The height and width of the benches shall be at least two (2) feet.

6. Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

7. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

8. It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

D1.8 Trench Backfill

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

1. Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

2. Trench backfill shall be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557 test method.

3. Rocks larger than one (1) inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in largest dimension diameter, and rocks shall be well mixed with finer soil.

4. The pipe design engineer should select bedding material for the pipe. Bedding materials generally should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419 test method.

5. 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 for granular soils and fine-grained soils, then placed in horizontal layers. 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.

6. The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

7. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

8. 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.

9. It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.

10. 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.

D1.9 Observation and Testing

1. During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.

2. Field density tests will be made during grading to provide an opinion on the degree of compaction being obtained by the contractor. Where compaction of

less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary, until the required degree of compaction is obtained.

3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.

APPENDIX E

GUIDE SPECIFICATIONS FOR INSTALLATION AND ACCEPTANCE OF TIE-BACK ANCHORS

APPENDIX E

GUIDE SPECIFICATIONS FOR INSTALLATION AND ACCEPTANCE OF TIE-BACK ANCHORS

E1.1 Installation

1. Tie-back installation shall be performed during continuous observation by Geotechnical Consultant to confirm that the recommended earth materials are penetrated, that the dimensions of the installed anchors are at least as large as that indicated on the shoring plan, and that anchor installation has been performed as specified. The Contractor shall provide access and necessary facilities, including lighting, at their expense, to accommodate observations.

2. All anchors shall be installed at the specified locations, to the required depth, and at the specified angle of inclination. A tolerance of 3o will be permitted on the required angle of inclination.

3. After drilling, all holes shall be cleaned of loose soils. Concrete shall be placed by pumping from the tip of the anchor to the active wedge. Concrete placement shall begin within four hours after completion of drilling. The portion of the anchor within the active wedge shall be backfilled with sand-cement slurry after the anchor has been tested as specified below. However, if excessive caving occurs, the active wedge portion of the excavation can be filled with slurry as the casing is pulled. A zone of soft soil shall (in this case) be placed between the anchor and slurry (before testing).

4. If a hollow-stem auger or casing is used due to caving, concrete shall be placed by pumping as the auger or casing is withdrawn while always maintaining a head of concrete inside the casing or auger.

5. Concrete placement shall be continuous without interruption, and at such a rate that fresh concrete will not be deposited on concrete hardened sufficiently to form seams and planes of weakness.

6. Any anchor deemed by the Owner or Geotechnical Consultant to be defective shall be replaced with substitute anchor(s) as directed by the Owner or Shoring Designer. The cost of installation of such substitute anchors shall be borne by the Contractor. Costs associated with analysis and design of substitute anchor(s) shall also be borne by the Contractor.

E1.2 Acceptance Criteria

1. Actual capacities of anchors shall be determined by testing designated Test Anchors and all Production Anchors. Testing of anchors will enable evaluation of the applicability of design values for the chosen method of tieback construction.

2. All anchors shall be check-tested to at least 150% of the designed working load in accordance with the following procedures:

   a. Test load anchors to 150% of the design-working load, incrementally noting loads, tendon extensions and soldier pile deflections. Hold load for 15 minutes. After pulling slack, the anchor movement shall not exceed 0.10 inch during the 15-minute load period. If the deflection is acceptable, reduce load to 100% of the design load and lock off.

   b. Where an anchor shows excessive movement for additional 15-minute intervals, the load should be reduced until the rate of movement is 0.10 inch per 15 minutes or less. The load at which acceptable movement is attained should be divided by 1.5 to establish the working load of the anchor and additional measures taken to carry the required load.

3. Geotechnical Consultant shall designate at least 5% of all proposed anchors as 200% Test Anchors. Additional anchor steel reinforcement will likely be required for the 200 percent load test anchors, and should be appropriately considered prior to anchor installation. Half of the 200% Test Anchors shall be tested for 30 minutes. The remaining Test Anchors shall be tested for a 24-hour period. Test anchors shall be tested in the following manner:

   a. For the 30-minute test anchors, incrementally load the anchors to 200% of the design-working load noting loads, tendon/bar extensions and soldier pile deflections. Hold load for 30 minutes. Anchor movement shall not exceed 0.3 inch during the 30-minute load period. If the deflection is acceptable, reduce load to design load and lock off; otherwise, reduce the test load by 50% and repeat this step.

   b. For 24-hour test anchors, incrementally load to 200% and hold for 24 hours; check load after 24 hours. If a pre-stress loss of 8% or less is recorded, restore load to 100% of working load and lock off. If loss of pre-stress exceeds 8%, restore load to 150% of working load and hold for an additional 24 hours. Check load after second 24-hour hold and, if loss of pre-stress is less than 8%; restore to 100% and lock off as before.

   c. Where an anchor shows a continuous loss of pre-stress during a subsequent 24-hour period, the test load shall continue to be reduced by 50% until loss of pre-stress is negligible. Then the test load shall be divided by 1.5 to establish the working load of that anchor and additional measures taken to carry the required shoring load.

4. Any anchor pulled more than 12 inches shall not be used.

5. Immediately after testing, the active wedge portion of tieback excavations should be filled with slurry.

APPENDIX F

GUIDE SPECIFICATIONS FOR DRILLED PILE INSTALLATION

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GUIDE SPECIFICATIONS FOR DRILLED PILE INSTALLATION

It should be the responsibility of the contractor to select proper construction equipment and method to correctly install the piles based on his own interpretation of the information presented in this report. The following recommendations are provided as a guide for preparing plans and specifications and for quality control:

F1.1 Drilled Piles

- Prior to starting any foundation work, staking should be checked by the project Civil/Structural Engineer. Variations in the alignment from the vertical greater than ¼-inch per foot of length should not be permitted. Any pile installed having a center more than three (3) inches off plan centerline will require structural analysis.

- Some variations in the final pile tip elevations should be expected. The actual tip elevation should be determined by the project geotechnical engineer during excavation based on observation of the actual field conditions.

- Sandy alluvial soils with gravel were encountered during our filed exploration. Layers with cobbles and boulders also exist within the alluvial soils and will be encountered during drilling of CIDH piles.

- Caving during excavations may occur within the sandy soils. Casing, or other methods approved by the project geotechnical consultant, should be used to support the sides of the pile excavation. Casing should be used at the discretion of the contractor. Casing should be advanced as drilling proceeds by drilling with a flight or bucket auger smaller in diameter than the inside of the casing. Occasional hammering may be required to advance the casing with the excavation. Casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. Drilling fluids should not be used to support the sides of the excavation without prior approval by the project geotechnical consultants. The contractor should have equipment on-site with sufficient pulling capacity to pull the casing at the proper time. The casing should have outside diameter not less than the specified diameter of the pile.

- In the event that the pile excavation becomes bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with sand and Portland Cement slurry. Drilling may continue when the slurry has reached its initial set. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set. Sufficient space should be provided in the pier-reinforcing cage during fabrication to allow insertion of a concrete pump pipe or tremie tube for concrete placement.

- The bottoms of the excavations should be cleaned of any loose cuttings before placing concrete. All applicable state and federal OSHA safety regulations must be satisfied during construction.

- The reinforcing bars in the piles should have a minimum concrete cover of 3 inches. Sufficient space should be provided in the reinforcing cage to allow insertion of a concrete tremie tube for concrete placement.

- The reinforcing cage must be carefully placed in uncased holes to prevent gouging of the sides. This will cause loose material to fall into the hole. The cage of reinforcing steel should be placed to the depth required by the plans, and adequately supported at the top.

- Pile shafts spaced closer than six (6) diameters center-to-center shall be drilled and filled with concrete alternatively, allowing at least 12 hours after concrete placement in one shaft before drilling of an adjacent shaft.

- All piles should be concreted immediately after drilling and clean out. Concrete should be placed through a tremie to prevent segregation and unnecessary splashing on the reinforcing steel. The concrete should be directed towards the center of the pile. Free fall of concrete should not exceed three (3) feet.

- The concrete should be flowable, non-segregating concrete with slump near the maximum allowable to obtain satisfactory consolidation without vibration, and to facilitate filling of all voids outside the casing. Concrete should not exhibit rapid slump loss. The slump for uncased drilled piles should be determined by the structural engineer. When casing is withdrawn, the minimum slump should be 6.0-in for specially designed concrete with retard to prevent arching of concrete during casing withdrawal, or setting of the concrete until after the casing is withdrawn, should be used. The slump can be 8±1 inches for concrete placed under groundwater determined by the structural engineer.

- Casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. The bottom of the casing should be maintained not more than five (5) feet nor less than one (1) foot below the top of the concrete during withdrawal and placing operations.

- Place concrete in pile in one continuous operation. Care should be taken to ensure that the concrete in the hole is dense and homogeneous. After the hole has been filled with concrete, the top 10 feet or the length of the reinforcing, whichever is greater should be vibrated.

- Drilled pile installation shall be performed under continuous observation by the project geotechnical consultant to confirm that the subsurface soils are similar to the soils encountered during our field study, which have formed the basis of our pier design recommendations. Further, the soils consultant should confirm that the dimensions of the installed piers are at least as large as those indicated on the foundation plan, and that pier installation has been performed as specified in this report. The contractor shall provide access and necessary facilities, including droplights, at his expense, to accommodate pier observations.

- Drilled pile installation shall be performed such that compliance with all safety rules and requirements is achieved. Drilling equipment, casing, reinforcement, and other items required for installation shall be kept at a safe distance from all overhead power lines and utilities.