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

April 22, 2021

All-ERA Properties
P.O. Box 11503
Carson, CA 90749

Subject: **PRELIMINARY GEOTECHNICAL INVESTIGATION AND WATER INFILTRATION TEST REPORT**
36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
Converse Project No. 20-81-168-01

Dear Mr. Walker:

Converse Consultants (Converse) has prepared this geotechnical investigation and water infiltration test report to present the findings, conclusions and recommendations for the 36 Unit Residential Development project located 7586 Jurupa Road in the city of Jurupa Valley, Riverside County, California. This report is prepared in accordance with our proposal dated May 8, 2020 and your acceptance of the of the Agreement and Authorization to Proceed, dated August 21, 2020.

Based upon our field investigation, laboratory data, and analyses, the proposed project is considered suitable from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and construction of the project.

We appreciate the opportunity to be of continued service to All-ERA Properties. If you should have any questions, please contact the undersigned at 909-796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, PhD, PE, GE
Regional Manager/Principal Engineer

Dist.: 2/Addressee

HSQ/ZA/RLG/CN

PROFESSIONAL CERTIFICATION

This report has been prepared by the individuals whose seals and signatures appear herein.

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

REGISTERED GEOLOGIST
ROBERT L. GREGOREK II
No. 1257
CERTIFIED ENGINEERING GEOLOGIST
Exp. 8/31/21
STATE OF CALIFORNIA

Zahangir Alam, PhD, EIT
Sr. Staff Engineer

Robert L Gregorek II, PG, CEG
Senior Geologist

REGISTERED PROFESSIONAL ENGINEER
HASHMI S. E. QUAZI
NO. GE 2517
Exp. 06/2021
GEOTECHNICAL
STATE OF CALIFORNIA

Hashmi S. E. Quazi, PhD, PE, GE
Principal Engineer

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1.0 INTRODUCTION

This report contains the findings of the geotechnical investigation and percolation tests performed by Converse for the proposed 36 unit residential development site located at 7586 Jurupa Road in the City of Jurupa Valley, Riverside County, California. The project location is shown in Figure No. 1, *Approximate Project Location Map.*

The purpose of this investigation was to evaluate the current nature and engineering properties of the subsurface soils and groundwater conditions, and to provide geotechnical recommendations for the proposed residential development.

This report is written for the project described herein and is intended for use solely by All-ERA Properties and their 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 PROJECT DESCRIPTION

Based on the referenced tentative tract map and conversations with All-ERA Properties, we understand the property will be developed for 36 detached single-family units. The structures will be one to two-story story homes and founded on shallow footings with slab-on-grade. There will also be one water infiltration device in the southern portion of the site. Even though not indicated on the referenced preliminary site plan it is anticipated that maximum cuts and fills will approximately 5 feet or less.

Associated with the development there will be roadways, parking areas, concrete walkways, block wall and landscaping, as well as above and underground utilities.

The original plans used for exploration was for 44 units, however due to density changes to the project the report was delayed until the current 36 unit plan was finalized.

3.0 SITE DESCRIPTION

The approximately 6.9-acre irregular shaped site is currently vacant and undeveloped, except for 2 abandoned residential structures at the western portion of the site. Some scattered trash and debris are also present on the site. Vegetation consists of a light to moderate growth of grass and weeds with some scattered bushes and trees. The site is bounded on the north by Jurupa Road, on the east by Camino Real, on the west by Kirby Drive and on the south by an elementary school. The site is roughly flat and appears to drain towards the south and southwest. Elevations range from approximately 845 feet above mean sea level (msl) in the northeast portion of the site to approximately 830 feet above msl in the southwest portion of the site.

Approximate Project Location Map

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Project: 36 Unit Residential Development
Location: 7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No
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FIGURE NO.
1

Present site conditions are shown in the photographs no. 1 through 5.

Photograph No. 1: Present site conditions, facing northwest.

Photograph No. 2: Present site conditions, facing southwest.

Photograph No. 3: Present site conditions, facing southeast.

Photograph No. 4: Present site conditions, facing northeast.

4.0 SCOPE OF WORK

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

4.1 Project Set-up

The project set-up consisted of the following tasks.

- Conducted a site reconnaissance to mark the boring and percolation test locations such that drill rig access to all the locations was available.
- Notified Underground Service Alert (USA) at least 48 hours prior to drilling to clear the boring locations of any conflict with existing underground utilities.
- Engaged a California-licensed driller to drill exploratory borings.

4.2 Subsurface Exploration

Five exploratory borings (BH-01 through BH-05) were drilled on August 25, 2020 to investigate subsurface conditions at the project site. The borings were drilled to depths ranging from 16.5 to 51.5 feet below existing ground surface (bgs).

Three exploratory percolation test holes (PT-01 through PT-03) were drilled on August 25, 2020 to perform percolation testing. All percolation test borings were drilled to approximately 4.5 to 6.0 feet below the existing ground surface (bgs).

Approximate boring and percolation testing locations are indicated in Figure No. 2, *Approximate Boring, Percolation Test, and Overexcavation Locations Map*. For a description of the field exploration and sampling program, see Appendix A, *Field Exploration*.

4.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in soil classification, and to evaluate relevant engineering properties. These tests included the following.

- *In-situ* moisture contents and dry densities (ASTM D2216 and D2937)
- Expansion index (ASTM D4829)
- R-value (California Test 301)
- Soil corrosivity (California Test Methods 643, 422, and 417)
- Collapse (ASTM D4546)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Organic Content
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

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

4.4 Historical Aerial Photograph Review

Historical Google aerial photographs of the site, between 1994 to 2020 were reviewed. Based on our review a portion of the site was a citrus grove, up to about 2002 to 2003, From about 2005 to 2009 the site appears to have been utilized as a nursery. After 2009 the site has been vacant except for the except for the residential structures at the western portion of the site.

4.5 Analysis and Report Preparation

Data obtained from the field exploration and laboratory testing program was assembled and evaluated. Geotechnical analyses of the compiled data were performed, followed by the preparation of this report to present our findings, conclusions, and recommendations for the proposed project.

Project: 36 Unit Residential Development

Location: 7586 Jurupa Road
City of Jurupa, Riverside County, California

For: All-ERA Properties

Approximate Boring, Percolation Test, and Overexcavation Locations Map

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Figure No. 2

5.0 SUBSURFACE CONDITIONS

A general description of the subsurface conditions, various materials and groundwater conditions encountered at the site during our field exploration is discussed below.

5.1 Subsurface Profile

Based on exploratory borings and laboratory test results, the subsurface soil at the project site generally consisted primarily of artificial fill, topsoil, and older alluvial fan deposits. These soils were comprised generally of silty sand and trace clay, with scattered trace gravel, up to 1 inch in largest dimension, at various depths.

At approximately 2 feet below ground surface (bgs) in PT-01, an organic layer, about 2 feet thick, in the artificial fill was encountered. A sample was collected and tested in our laboratory to confirm the presence of organic material. Laboratory analyses confirmed the presence of a significant organic content at this location. Detailed observations should be made during clearing and overexcavation of this area to evaluate the actual extent of this material.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawings No. A-2 through A-9, *Logs of Borings,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was encountered during our field investigation in borings BH-01 and BH-03 at depths of approximately 27.1 feet and 24.5 feet bgs, respectively.

The GeoTracker database (SWRCB, 2021) was reviewed for groundwater data from sites within an approximately 1.0-mile radius of both the proposed development. Results of that search are as follows:

- TOSCO/CIRCLE K (Site No. #T0606500530), located approximately 4,000 feet northwest of the project site reported groundwater at depths ranging from 40.46 to 62.30 feet bgs between 1998 and 2010.

- MOBIL SERVICE STATION #18-HTY (#T0606500478), located approximately 3,600 feet northeast of the project site reported groundwater at a depth of 36.90 feet bgs in 2012.

The National Water Information System (USGS, 2021) were reviewed for groundwater data from sites within an approximately 1.0-mile radius of the proposed development and the results of that search are included below.

Table No. 1, Summary of USGS Groundwater Depth Data

Alignment No.	Location	Groundwater Depth Range (ft. bgs)	Date Range
340017117272901	NW corner of Galena Street and Tyrolite Street; approximately 3,520 feet west of project site	39.80	2016

The California Department of Water Resources, Water Data Library (CDWR, 2021) online database was reviewed for groundwater data from sites within close proximity of the project, but no data was identified within a 1.0-mile radius of the project site.

Based on available data, the historical high groundwater level near the site is estimated to be approximately 36.90 feet bgs, and the current groundwater level is estimated to be approximately 24.5 feet bgs. Groundwater is not expected to be encountered during construction of the proposed project, however perched water layers may be present at shallower depths, particularly following high precipitation or irrigation events.

5.3 Excavatability

The subsurface materials of the project site are expected to be excavatable by conventional heavy-duty earth moving equipment. Difficult excavation will occur where high concentration of gravel, cobbles or boulders (possibly) are encountered. Due to the nature of the alluvial fan deposits, boulders could be present at depths below approximately 5 feet to 10 feet bgs at the project site.

The phrase "conventional heavy-duty excavation equipment" is intended to include commonly used equipment such as excavators, scrapers, and trenching machines. It does not include hydraulic hammers ("breakers"), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment model should be done by an experienced earthwork contractor.

5.4 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface soil conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations.

5.5 Caving

Caving was not encountered in any of the exploratory borings. However, localized caving could occur within excavations made into granular soils of the on-site soils.

5.6 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade. Depending on the extent and location below finish subgrade, expansive soils can have a detrimental effect on structures.

Based on the laboratory test results, the expansion index of the upper 5 feet of the site soils was 2, corresponding to a very low expansion potential.

5.7 Collapse Potential

Soil deposits subjected to collapse/hydro-consolidation generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Moreover, some soils may have a different degree of collapse/hydro-consolidation based on the amount of proposed fill or structure loads. Soils susceptible to collapse/ hydro-consolidation include wind-blown silt, weakly cemented sand, and silt where the cementing agent is soluble (e.g., soluble gypsum, halite), alluvial or colluvial deposits within semi-arid to arid climate, and certain weathered bedrock above the groundwater table.

Granular soils may have a potential to collapse upon wetting in arid climate regions. Collapse/hydro-consolidation may occur when the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose/low density configuration from deposition.

The degree of collapse of a soil can be defined by the collapse potential value, which is expressed as a percent of collapse of the total sample using the Collapse Potential Test (ASTM D4546). According to the ASTM guideline, the severity of collapse potential is commonly evaluated by the following Table No.12, *Collapse Potential Values*.

Table No. 2, Collapse Potential Values

Collapse Potential Value (%)	Severity of Problem
0	None
0.1 to 2	Slight
2.1 to 6.0	Moderate
6.0 to 10.0	Moderately Severe
>10	Severe

Based on the laboratory test result (collapse potential of 2.0 percent at a depth of 3.0 feet bgs), a slight problem is anticipated at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

6.0 ENGINEERING GEOLOGY

The regional and local geology within the proposed project area are discussed below.

6.1 Regional Geology

The project site is located within the northern Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the southwest by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Elsinore, and San Andreas fault zones (CGS, 2007), all of which have been known to be active during Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

The site is located within the southeastern portion of the Chino Basin of the Peninsular Ranges province. The Chino Basin is a broad alluvial valley bounded by the San Gabriel Mountains on the north, the San Bernardino Mountains on the east and northeast, the Santa Ana Mountains on the southwest, and the Puente Hills on the west.

6.2 Local Geology

Based on our review of the regional mapping (Morton, 2006), available geotechnical literature, and our current exploration, it is our understanding that the proposed residential development site is primarily underlain by shallow artificial fills and a topsoil layer which overlie Pleistocene-aged older alluvial fan deposits (Qof). The older alluvial fan deposits. A description of the earth material soils encountered are described below:

Artificial Fill, Undocumented (Afu): Undocumented non-engineered artificial fills are present, scattered over southwestern portion of the subject site, likely associated with grading for the previous nursery operations. Based on exploration and geologic mapping, the approximate depth of these fill soils is estimated to about 4 feet deep. Where observed these non-engineered fill soils are generally comprised of silty sand, which is fine to coarse-grained, trace gravel, some organics, medium dense, moist and reddish brown to black. The gravel was up to 1 inch in largest dimension.

Topsoil (no map symbol): Topsoil was encountered in all borings ranging from approximately 1.0 foot to 5.5 feet thick. The thickness and depth of the topsoil likely varies through the site due to grading for the previous nursery operations. Based on the exploratory borings and laboratory test results, these materials primarily consist of silty sand, which is fine to coarse-grained, trace to few gravel, loose to dense, dry to moist and orangish brown. The gravel was up to 1 inch in largest.

Older Alluvial Fan Deposits (Qof): The topsoil is underlain by Pleistocene-aged older alluvial fan deposits. Based on the exploratory borings and laboratory test results, these materials primarily consist of silty sand, which is fine to coarse-grained, trace clay, slightly to moderately desiccated, localized caliche, localized roots and rootlets near the surface, medium dense to very dense, moist and various shades of reddish brown and orangish brown. These materials became wet below the groundwater level of approximately 27.1 feet and 24.5 feet bgs. Portions of the about the upper 0.5 foot to 1.0 feet are weathered.

Bedrock (gdgb): The old alluvial fan deposits are underlain by Cretaceous-aged granitic bedrock and was encountered in BH-03 at a depth of approximately 35 feet bgs and approximately 10.5 feet below the groundwater. The bedrock consists of granodiorite with some gabbro which was slightly weathered and hard to very hard Based on exploratory borings and laboratory test results, the bedrock generally excavates as silty sand, which is fine to coarse-grained, trace clay, very dense, moist to wet and grayish brown.

6.3 Flooding

Review of National Flood Insurance Rate Maps indicates that the project site is within a Flood Hazard Zone "X". The Zone “X” is designated as an area with an area of minimal hazard (FEMA, 2008).

7.0 FAULTING AND SEISMICITY

The approximate distance and seismic characteristics of nearby faults as well as seismic design coefficients are presented in the following subsections.

7.1 Faulting

The proposed site is situated in a seismically active region. As is the case for most areas of Southern California, ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project is high.

The project site is not located within a currently mapped State of California Earthquake Fault Zone for surface fault rupture (CGS, 2007). Table No. 2, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 50 kilometers of the site. The data presented below was calculated using the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

Table No. 3, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
San Jacinto	9.64	strike slip	241	n/a	7.88
Cucamonga	11.29	thrust	28	5	6.70
Chino, alt 1	13.26	strike slip	24	1	6.70
Chino, alt 2	13.27	strike slip	29	1	6.80
Elsinore	14.46	strike slip	241	n/a	7.85
S. San Andreas	15.09	strike slip	548	n/a	8.18
San Jose	15.82	strike slip	20	0.5	6.70
Sierra Madre	18.66	reverse	57	2	7.20
Sierra Madre Connected	18.66	reverse	76	2	7.30
Cleghorn	19.64	strike slip	25	3	6.80
North Frontal (West)	23.8	reverse	50	1	7.20
Puente Hills (Coyote Hills)	25.24	thrust	17	0.7	6.90
Clamshell-Sawpit	28	reverse	16	0.5	6.70

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
San Joaquin Hills	29.75	thrust	27	0.5	7.10
Raymond	32.97	strike slip	22	1.5	6.80
Puente Hills (Santa Fe Springs)	33.16	thrust	11	0.7	6.70
Elysian Park (Upper)	37.7	reverse	20	1.3	6.70
Newport Inglewood Conn. alt 2	38.66	strike slip	208	1.3	7.50
Newport Inglewood Conn. alt 1	38.76	strike slip	208	1.3	7.50
Newport-Inglewood, alt 1	38.76	strike slip	65	1	7.20
Puente Hills (LA)	39.05	thrust	22	0.7	7.00
Newport-Inglewood (Offshore)	39.17	strike slip	66	1.5	7.00
Verdugo	41.43	reverse	29	0.5	6.90
Helendale-So Lockhart	41.59	strike slip	114	0.6	7.40
Pinto Mtn	41.85	strike slip	74	2.5	7.30
North Frontal (East)	43.57	thrust	27	0.5	7.00
Hollywood	45.59	strike slip	17	1	6.70
Santa Monica Connected alt 2	48.54	strike slip	93	2.4	7.40
Palos Verdes Connected	49.58	strike slip	285	3	7.70
Palos Verdes	49.58	strike slip	99	3	7.30

(Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

7.2 CBC Seismic Design Parameters

Seismic parameters based on the 2019 California Building Code (CBSC, 2019) are provided in the following table. These parameters were determined using the generalized coordinates (34.0052N, 117.4481W) and the Seismic Design Maps ATC online tool.

Table No. 4, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.0052 N, 117.4481 W
Site Class	D*
Risk Category	II
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.222g
Mapped 1-second Spectral Response Acceleration, S_1	0.429g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.011
Site Coefficient (from Table 1613.5.3(2)), F_v	1.871

Seismic Parameters	
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.235g
MCE 1-second period Spectral Response Acceleration, SM_1	0.803g
Design Spectral Response Acceleration for short period S_{DS}	0.824g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.535g
Site Modified Peak Ground Acceleration, PGA_M	0.593g

** Stiff Soil Classification*

7.3 Secondary Effects of Seismic Activity

In addition to ground shaking, effects of seismic activity on a project site may include surface fault rupture, soil liquefaction, landslides, lateral spreading, seismic settlement, tsunamis, seiches and earthquake-induced flooding. Results of a site-specific evaluation of each of the above secondary effects are explained below.

Surface Fault Rupture: The project site is not located within a currently designated State of California Earthquake Fault Zone (CGS, 2007). Based on review of existing geologic information, no major surface fault crosses through or extends toward the site. The potential for surface rupture resulting from the movement of a presently unrecognized fault beneath the site is not known with certainty but is considered very low.

Liquefaction: Liquefaction is defined as the phenomenon in a soil mass, because of the development of excess pore pressures, soil mass suffers a substantial reduction in its shear strength. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction. Soil liquefaction occurs in submerged granular soils during or after strong ground shaking. There are several requirements for liquefaction to occur. They are as follows.

- Soils must be submerged.
- Soils must be primarily granular.
- Soils must be contractive, that is, loose to medium-dense.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

This site is located in a Riverside County liquefaction zone designated with a risk factor of “high”.

Based on the relatively dense/fine grained nature of the soils, bedrock being at approximately 35 feet and recommended remedial grading, liquefaction at the site is expected to be negligible.

Seismic Settlement*:* Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. The potential for seismic settlement is not known with certainty. Based on our evaluation of dynamic settlement the potential for dry seismic settlement of the site is expected to be negligible.

Landslides: Seismically induced landslides and other slope failures are common occurrences during or after earthquakes in areas of significant relief. The project site is not adjacent to any steep slopes. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered low.

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. Due to the relatively flat nature of the project site, the relatively dense nature of the soils, recommended remedial grading and the negligible amount of potential liquefaction, the risk of lateral spreading is considered low.

Tsunamis: Tsunamis are tidal waves generated in large bodies of water by fault displacement or major ground movement. Based on the location of the site, tsunamis do not pose a hazard to this site.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Review of the area adjacent to the site indicates that there are no significant up-gradient lakes or reservoirs with the potential of flooding the site.

Earthquake-Induced Flooding: This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the area adjacent to the site indicates the site is not located in any potential inundation path of any reservoir. The potential for flooding of the site due to dam failure is considered very low.

8.0 LABORATORY TEST RESULTS

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Tests results are included in Appendix A, *Field Exploration* and Appendix B, *Laboratory Testing Program*. Discussions of the various test results are presented below:

8.1 Physical Testing

- In-situ Moisture and Dry Density: *In-situ* dry density and moisture content of the soils were determined in accordance with ASTM Standard D2216 and D2937. Results are presented in the log of borings in Appendix A, *Field Exploration.*
  - Dry densities of the artificial fill and topsoil ranged from 94 to 117 per cubic feet (pcf) with moisture contents ranging from 5 to 19 percent.
  - Dry densities of the older fan deposits in the upper 10 feet soils at the site soils ranged from 112 to 131 pcf with moisture contents ranging from 5 to 12 percent.
- Expansion Index: One representative bulk soil sample from the upper 5 feet of the site materials was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result indicated expansion index is 2, corresponding to very low expansion potential.
- R-Value: One representative bulk sample was tested in accordance with Caltrans Test Method 301. The result of the R-value test was 20.
- Collapse Potential: The collapse potential of one relatively undisturbed sample was tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test result showed collapse potential of 2.0 percent, indicating low collapse potential.
- Maximum Dry Density and Optimum Moisture Content: Typical moisture-density relationships of two representative soil samples were performed in accordance with ASTM Standard D1557. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Result*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry densities were 127.0 and 130.5 pounds per cubic feet (pcf), with optimum moisture contents of 11.7 and 9.2 percent.
- Organic Content – One organic content tests were performed in accordance with ASTM Standard D2974 on a representative ring soil sample. The amount of organic material present in the artificial fill soils was 25.1%
- Direct Shear: Two direct shear tests were performed; one direct shear test was performed on a relatively undisturbed sample and one direct shear test was performed on sample remolded to 90% of the maximum dry density under soaked moisture condition in accordance with ASTM Standard D3080. The results of the direct shear tests are presented in Drawings No. B-3 and B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.
- Consolidation Test – One consolidation test was performed on a relatively undisturbed sample of the site soil, in accordance with ASTM Standard D2435. The test result is shown on Drawing No. B-5, *Consolidation Test Results,* in Appendix B, *Laboratory Testing Program.*

8.2 Chemical Testing - Corrosivity Evaluation

One representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of this test was to determine the corrosion potential of site soils when placed in contact with common pipe materials. The test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program and* are summarized in below.

- The pH measurement of the sample tested was 8.3.
- The sulfate content of the sample tested was 0.0064 percent by weight (64 ppm).
- The chloride concentration of the sample tested was 40 ppm.
- The minimum electrical resistivity when saturated was 2,720 ohm-cm.

9.0 PERCOLATION TESTING

Three percolation tests (PT-01 through PT-03) were performed on August 27 and 29, 2020 to evaluate water infiltration rate. The measured percolation test data and calculations are represented in Appendix C, *Percolation Testing*. The estimated infiltration rates at each test hole are presented in the following table.

Table No. 5, Estimated Infiltration Rates

Percolation Test	Test Depth (feet)	Soil Type	Infiltration Rate (inches/hr) (FOS 3)
PT-01	6.0	Silty Sand (SM)	0.41
PT-02	5.0	Silty Sand (SM)	6.29
PT-03	4.5	Silty Sand (SM)	0.34

Due to the presence of organics and debris present in PT-01 from approximately 2 feet to 4 feet bgs, steps were taken to isolate the infiltration test below this layer. Solid pipe was placed in the hole down to approximately 4 feet bgs. Based on the calculated infiltration rate during the final respective intervals in each test, an average infiltration rate of 2.35 inches per hour can be utilized.

10.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

Recommendations for earthwork are presented in the following subsections.

10.1 General

This section contains our general recommendations regarding earthwork for the proposed 36 unit residential development project.

These recommendations are based on the results of our field exploration and laboratory testing, our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may require modification by the geotechnical consultant based on observation of the actual field conditions during remedial grading.

Prior to the start of construction, all underground existing utilities and appurtenances should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing structures or utilities.

All existing structures, debris, deleterious material, highly organic soil and surficial soils containing roots and perishable materials should be stripped and removed from the project site. Deleterious material, including organics, concrete, and debris generated during excavation, should not be placed as fill.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill. Based on these observations, localized areas may require remedial grading deeper than indicated herein. Therefore, some variations in the depth and lateral extent of excavation recommended in this report should be anticipated.

10.2 Private Sewage System Abandonment

From a geotechnical standpoint, any seepage pits, other private sewage systems, and/or other subsurface structures that may be encountered should be located, mapped on the grading plans, removed and/or properly abandoned. Abandonment and/or removal of septic systems that may exist should be in accordance with local codes and recommendations by Converse. Seepage pits, if abandoned in-place, should be pumped clean, backfilled with gravel or clean sand jetted into place, and then capped with a minimum of 2 feet of a 2-sack or greater slurry or concrete for a minimum distance of 2 feet outside the edge of the seepage pit. The top of the slurry or concrete cap should be at a minimum 10 feet below proposed grade.

10.3 Overexcavation

The site is generally underlain by approximately 2.0 to 5.0 feet of potentially compressible soils (artificial fill, topsoil, and the upper weathered portions of the older alluvial fan deposits). However, localized, deeper over-excavation, as much as approximately 6.0 feet to 10.0 feet exist along the southern portion of the site likely associated with grading for the previous nursery operations. These materials may be prone to future settlement under the surcharge of foundation, improvements and/or fill loads. Therefore, these materials should be over-excavated to competent older alluvial fan deposits, within all areas of proposed structures and other improvements, and replaced with compacted fill soils. Within the entire level portions of the building pad areas, over-excavations should also extend at least 4.5 feet below proposed pad grade, as well as at least 2.0 feet below the lowest proposed footings, within the proposed building areas, whichever is deeper. Within proposed wall footings areas over-excavation should also be a minimum of 3.0 feet below proposed pad grade or 2.0 feet below the proposed wall footings areas, whichever is deeper. All over-excavations should extend outside the entire level portions of the building pad area at least 5.0 feet or equal to the depth of over-excavation, whichever is greater. Within wall and pavement areas overexcavations should extend laterally at least 2.0 feet or equal to the depth of over-excavation, whichever is greater. The final bottom surfaces of all excavations should be approved by the project geotechnical consultant prior to placing any fill or structures, based on observations and testing by the geotechnical consultant during grading of the final bottom surfaces of all excavations.

The estimated locations and approximate depths of over-excavation of unsuitable, compressible soil materials are indicated on Figure No. 2, *Approximate Boring,* Percolation Test, *and Overexcavation Locations Map*.

If isolated pockets of very soft, loose, eroded, or pumping soil are encountered, the unstable soil should be excavated as needed to expose undisturbed, firm, and unyielding soils.

The contractor should determine the best manner to conduct the excavations, such that there are no losses of bearing and/or lateral support to the existing structures or utilities (if any).

Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

10.4 Engineered Fill

No fill soils or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project site are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including removal of roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-site soils used as fill should meet the following criteria.

- No particles larger than 6 inches in largest dimension.
- Rocks larger than one inch should not be placed within the upper 12 inches of subgrade soils.
- Free of all organic matter, debris, or other deleterious material.
- Expansion index of 20 or less.
- Sand equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.

- Contain less than 40 percent fines (passing #200 sieve).

Based on the laboratory test results, on-site soils may be utilized as fill materials.

Imported materials, if required, should meet the above criteria prior to being used as compacted fill. Any imported fills should be tested and approved by geotechnical consultant prior to delivery to the site.

10.5 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 6 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

Fill soils should be thoroughly mixed, and moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. Fill soils should be evenly spread in horizontal lifts not exceeding 8 inches in uncompacted thickness.

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. At least the upper 2 feet of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and compacted to at least 95 percent of the laboratory maximum dry density.

To reduce differential settlement, variations in the soil type, degree of compaction and thickness of the engineered fill placed underneath the foundations should be minimized.

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

10.6 Shrinkage and Subsidence

The volume of excavated and recompacted soils will decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. Based on our previous experience in the other projects in close vicinity of this site, for the preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the upper 10 feet of soils is estimated. An average value of 5 to 10 percent in the upper 5 feet and an average value of 0 to 5 percent from 5 feet to 10 feet may be used for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading) would depend on the construction methods including type of equipment utilized. Ground subsidence is estimated to be approximately 0.10 foot to 0.15 foot.

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.

10.7 Site Drainage

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. A desirable drainage gradient is 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

10.8 Utility Trench Backfill

The following sections present earthwork recommendations for utility trench backfill, including subgrade preparation and trench zone backfill.

Open cuts adjacent to existing roadways or structures are not recommended within a 1:1 (horizontal: vertical) plane extending down and away from the roadway or structure perimeter (if any).

Soils from the trench excavation should not be stockpiled more than 6 feet in height or within a horizontal distance from the trench edge equal to the depth of the trench. Soils should not be stockpiled behind the shoring, if any, within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

10.8.1 Pipeline Subgrade Preparation

The final subgrade surface should be level, firm, uniform, and 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 2 inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

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

10.8.2 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated site soils free of oversize particles and deleterious matter may be used to backfill the trench zone. Detailed trench backfill recommendations are provided below.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper 1 foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.
- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened, or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should 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 working conditions during all phases of construction.

- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

11.0 DESIGN RECOMMENDATIONS

The various design recommendations provided in this section are based on the assumption that the above earthwork and grading recommendations will be implemented in the project design and construction.

11.1 General Evaluation

The various design recommendations provided in this section are based on the exploration and laboratory testing as well as the assumption that in preparing the site, the earthwork recommendations provided in this report will be implemented.

11.2 Preliminary Shallow Foundation Design Parameters

The proposed one- and two-story buildings and possible retaining walls may be supported on continuous or isolated spread footings founded completely within in competent compacted fill. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

Table No. 5, Recommended Foundation Parameters

Parameter	1-Story Value	2-Story Value
Minimum continuous footing width (interior and exterior)	12 inches	15 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade (interior and exterior)	15 inches	18 inches
Allowable net bearing capacity	3,000 psf	3,000 psf

Isolated interior footings should be at least 24 inches wide. The footing dimensions and reinforcement should be based on structural design. The allowable bearing capacity can be increased by 500 pounds per square foot (psf) with each foot of additional embedment and 100 psf with each foot of additional width up to a maximum of 4,000 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. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

11.3 Lateral Earth Pressures and Resistance to Lateral Loads

In the following subsections, the lateral earth pressures and resistance to lateral loads are estimated by using on-site native soils strength parameters obtained from laboratory testing.

11.3.1 Active Earth Pressures

The active earth pressure behind any buried wall or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The lateral earth pressures for the project site are presented in the following tables.

Table No. 6, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure[1] (psf)	Lateral Earth Pressure[2] (psf)
	Level backfill	2:1 backfill
Active earth conditions (wall is free to deflect at least 0.001 radian)	35	65
At-rest (wall is restrained)	55	80

These pressures assume a level ground surface around the structure for a distance greater than the structure height, no surcharge, and no hydrostatic pressure.

If water pressure is allowed to build up behind the structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

11.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.40 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 260 psf per foot of depth may be used for the sides of footings poured against recompacted soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,600 psf for compacted fill.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

11.4 Retaining Walls Drainage

The recommended lateral earth pressure values, for any future retaining walls, do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures. Behind wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels or weep holes. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, or other suitable location for disposal. We recommend drain rock should consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of 110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

11.5 Slabs-on-Grade

Slab-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 10.5 *Compacted Fill Placement*.

Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing of more heavily-loaded slabs will be dependent upon the anticipated loading conditions and the modulus of subgrade reaction (200 kcf) of the supporting materials 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). Care should be taken during concrete placement to avoid slab curling. Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

Subgrade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted.
If moisture-sensitive flooring or environments are planned, slabs-on-grade should be protected by 10-mil-thick polyethylene vapor barriers. The sub-grade surface should be free of all exposed rocks or other sharp objects prior to placement of the barrier. The barrier should be overlain by 2 inches of sand, to minimize punctures and to aid in the

concrete curing. At discretion of the structure engineer, the sand layer may be eliminated.

In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking or curling of the slabs. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of the water/cement ratio (maximum 0.40).

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least 7 days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof may be used after finishing operations have been completed. The edges of concrete slabs exposed after removal of forms should be immediately protected to provide continuous curing.

11.6 Settlement

The total settlement of shallow footings, designed as recommended above, from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1/2 inch or less. The static differential settlement can be taken as equal to one-half of the static total settlement over a lateral distance of 40 feet.

The potential dynamic settlement for the project site from liquefaction and dynamic differential settlement is considered negligible.

11.7 Soil Corrosivity

The results of chemical testing of a representative sample of site soil were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel. The test results are presented in Appendix B, *Laboratory Testing Program*, Summary of Corrosivity Test Results, and are discussed below.

The sulfate content of the sampled soil corresponds to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, Table 19.3.1.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-14, Table 19.3.2.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures such as footings, slab, and flatwork will be exposed to moisture from precipitation and irrigation. Based on the project location and the results of chloride testing of the site soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table

19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a minimum compressive strength of 2,500 psi, and a maximum chloride content of 0.3 percent.

According to Romanoff, 1957, the following table provides general guideline of soil corrosion based on electrical resistivity.

Table No. 7 Correlation Between Resistivity and Corrosion

Soil Resistivity (ohm-cm) per Caltrans CT 643	Corrosivity Category
Over 10,000	Mildly corrosive
2,000 – 10,000	Moderately corrosive
1,000 – 2,000	corrosive
Less than 1,000	Severe corrosive

The measured value of the minimum electrical resistivity when saturated was 2,720. This indicates that the soils tested are moderately corrosive for ferrous metals in contact with the soil (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for ferrous metals in contact with the site soils.

11.8 Pavement Recommendations

One soil sample was tested to determine the R-value of the subgrade soils. Based on laboratory testing, R-value was 20. For pavement design, we have utilized R-value of 50 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2017), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table below. City of Jurupa Valley minimum asphalt pavement and aggregate base thickness requirements should also be considered in the pavement design.

Table No. 8, Recommended Preliminary Pavement Sections

R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
50	5	3.5	6.0	5.5
	6	4.5	7.5	7.0
	7	5.0	10.0	8.5
	8	6.0	11.2	9.5

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

Prior to placement of aggregate base and full AC, at least the upper 2 feet of subgrade soils should be scarified, moisture-conditioned if necessary, and recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method.

Base materials should conform with Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302.5 of the SSPWC.

11.9 Concrete Flatwork

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2018).

The subgrade soils under the above structures should consist of compacted fill placed as described in this report. Prior to placement of concrete, the upper 2 feet of subgrade soils should be moisture conditioned to between within 3 percent of optimum moisture content for coarse-grained soils and 0 and 2 percent above optimum for fine-grained soils.

The thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with a longitudinal control joint.

Concrete walks subjected to pedestrian and bicycle loading should be at least 4 inches thick, or as required by the civil or structural engineer. Transverse joints should be spaced 15 feet or less and should be cut to a depth of one-fourth the slab thickness.

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

12.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

12.1 General

Prior to the start of construction, all existing underground utilities should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Sloped excavations may not be feasible in locations adjacent to existing utilities, pavement, or structure (if any). Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing structures may require vertical sidewall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

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

12.2 Temporary Sloped Excavations

Temporary open-cut trenches may be constructed with side slopes as recommended in the following table. Temporary cuts encountering soft and wet fine-grained soils; dry loose, cohesionless soils or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 9, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal: Vertical)[1]
Silty Sand (SM),	C	0-10	1.5:1

[1] Slope ratio assumed to be uniform from top to toe of slope.

For shallow excavations up to 4 feet bgs, vertical excavations can be considered. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trench shields should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring are provided in the following section.

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 materials, should not be placed within 5 feet of the unsupported slope edge. Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

13.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should review plans and specifications as the project design progresses. Such review is necessary to identify design elements, assumptions, or new conditions which require revisions or additions to our geotechnical recommendations.

The project geotechnical consultant should be present to observe conditions during construction. Geotechnical observation and testing should be performed as needed to verify compliance with project specifications. Additional geotechnical recommendations may be required based on subsurface conditions encountered during construction.

14.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by Mehas Construction Inc., and their authorized agents, to assist in the development of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations 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. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction.

Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, a continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that the 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 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.

15.0 REFERENCES

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APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling soil borings. During the site reconnaissance, the surface conditions were noted, and the borings were marked in the field using approximate distances from local streets as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Five borings (BH-01 through BH-05) were drilled on August 25, 2020 within the project site to investigate subsurface conditions. All borings were drilled to approximately 16.5 to 51.5 feet below ground surface bgs.

Three test holes (PT-01 through PT-03) were drilled on August 25, 2020 within the project site to perform percolation testing. All borings were drilled to were drilled to approximately 4.5 to 6.0 feet bgs.

The borings were advanced using a CME 75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. Encountered materials were continuously logged by a Converse geologist and classified in the field by visual classification in accordance with the Unified Soil Classification System. Where appropriate, the field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin 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. Blow counts at each sample interval are presented on the boring logs for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Bulk samples of typical soil types were also obtained.

Standard Penetration Testing (SPT) was also performed in borings BH-01 and BH-03 in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet bgs 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, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

The exact depths at which material changes occur cannot always be established accurately. Unless a more precise depth can be established by other means, changes in material conditions that occur between drive samples are indicated on the logs at the top of the next drive sample.

Following the completion of logging and sampling, the borings were backfilled with soil cuttings and compacted by pushing down using drill rig weight. The surface was patched with concrete, where applicable. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing No. A-1, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-9, *Logs of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	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
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)

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 (CMS).

DRIVE SAMPLE No recovery

BULK SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

DRILLING METHOD SYMBOLS

Auger Drilling

Mud Rotary Drilling

Dynamic Cone or Hand Driven

Diamond Core

SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name:
Project Location:
For:

Project No.

Drawing No.
A-1a

Project ID: ; Template: KEY

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	SPT Blow Counts	Pocket Penetrometer (tsf)	CA Sampler	Torvane (tsf)	Field Approximation
Very Soft	<0.25	< 2	<0.25	<3	<0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	2 - 4	0.25 - 0.50	3 - 6	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	5 - 8	0.50 - 1.0	7 - 12	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	9 - 15	1.0 - 2.0	13 - 25	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	16 - 30	2.0 - 4.0	26 - 50	1.0 - 2.0	Readily indented by thumbnail
Hard	>4.0	>30	>4.0	>50	>2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60}- Value (blows / foot)	CA Sampler
Very Loose	<4	<5
Loose	4- 10	5 - 12
Medium Dense	11 - 30	13 - 35
Dense	31 - 50	36 - 60
Very Dense	>50	>60

MOISTURE

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

PERCENT OF PROPORTION OF SOILS

Descriptor	Criteria
Trace (fine)/ Scattered (coarse)	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION/ Induration

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name:
Project Location:
For:

Project No.

Drawing No.
A-1b

LEGEND OF ROCK MATERIALS
IGNEOUS ROCK
SEDIMENTARY ROCK
METAMORPHIC ROCK

BEDDING SPACING

Description	Thickness/Spacing
Massive	Greater than 10 ft
Very Thickly Bedded	3 ft - 10 ft
Thickly Bedded	1 ft - 3 ft
Moderately Bedded	4 in - 1 ft
Thinly Bedded	1 in - 4 in
Very Thinly Bedded	1/4 in - 1 in
Laminated	Less than 1/4 in

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Leaching		
Description	Body of Rock	Fracture Surfaces		Texture	Leaching	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No leaching	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, grain boundary conditions	All fracture surfaces are discolored or oxidized; surfaces friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

PERCENT CORE RECOVERY (REC)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\frac{\Sigma \text{ Length of intact core pieces } \geq 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

RQD* indicates soundness criteria not met.

ROCK HARDNESS

Description	Criteria
Extremely Hard	Cannot be scratched with a pocketknife or sharp pick. Can only be chipped with repeated heavy hammer blows
Very Hard	Cannot be scratched with a pocketknife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Can be scratched with a pocketknife or sharp pick with difficulty (heavy pressure). Breaks with heavy hammer blows.
Moderately Hard	Can be scratched with a pocketknife or sharp pick with light or moderate pressure. Breaks with moderate hammer blows
Moderately Soft	Can be grooved 1/16 in. deep with a pocketknife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Can be grooved or gouged easily with a pocketknife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Can be readily indented, grooved or gouged with fingernail, or carved with a pocketknife. Breaks with light manual pressure.

Fracturing Spacing

Description	Observed Fracture Density
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 ft.
Slightly Fractured	Core lengths mostly from 1 to 3 ft.
Moderately Fractured	Core lengths mostly 4 in. to 1 ft.
Intensely Fractured	Core lengths mostly from 1 to 4 in.
Very Intensely Fractured	Mostly chips and fragments.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

BEDROCK CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No. 20-81-168-01

Drawing No. A-1c

Log of Boring No. BH-01

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 835 Depth to Water (ft): 27.11

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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, dense, moist, orangish brown.			9/30/50	5	94	CP
		OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, trace clay, very dense, moist, light reddish brown, slight desiccation.			25/37/49	8	126	
5								
		- @6.0': no noticible desiccation.			9/10/17	8	126	DS
					12/16/20	6	128	
10								
		- @14.0': possible caliche pockets, slight desiccation.			22/39/50-5"	8	130	
15								
		- @19.0': dark reddish brown, moderate desiccation.			50-6"	11	114	
20								
		- @24.0' : medium dense.			12/12/12	15		
25								
		- @27.1': groundwater.						
		- @29.0': wet, grayish brown.			25/50-6"	17	113	
30		End of boring at 30.0 feet bgs. Groundwater encountered at 27.1 feet. Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.						

Log of Boring No. BH-02

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 838 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
	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.						
	TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1" in largest dimension, loose, dry, orangish brown.						
	OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, trace clay, roots and rootlets, dense, moist, reddish brown, slight desiccation.	■	▨	15/18/23	11	131	EI, CR, CP, DS
5	- @5.0': very dense, dark reddish brown, moderate desiccation.	■		6/15/50	12	118	
	- @8.0': trace clay, slight mottling, possible caliche, orangish brown, slight desiccation.	■		14/50-6"	11	117	
10		■		17/50-5"	12	114	
15		■		35/50-3"	12	118	

End of boring at 16.8 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No. **20-81-168-01** Drawing No. **A-3**

Log of Boring No. BH-03

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 836 Depth to Water (ft): 24.5

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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1" in largest dimension, loose, dry, orangish brown.	■	▩	16/31/34	6	119	
5		**OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, trace clay, very dense, moist, orangish brown, slight to moderate desiccation.	■	▩	22/50-6"	8	123	
			■		50-6"	7	113	
10		- @10.0': reddish brown, moderate desiccation.	■	▩	16/50-6"	12	118	
15			■		32/50-5"	12	116	
20		- @20.0': dense.	☒		10/14/17	11		
25		- @24.5': groundwater. - @25.0': very dense.	■		21/50-6"	11	127	
30			☒		30/50-3"	13		

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No. 20-81-168-01

Drawing No. A-4a

Log of Boring No. BH-03

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 836 Depth to Water (ft): 24.5

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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		BEDROCK (gdgb): Granodiorite/gabbro, slightly weathered, wet, light to dark gray. **Excavates as: SILTY SAND (SM):** fine to coarse-grained, trace clay, very dense, moist, grayish brown to black.			50-4"	9	131	
40		- @40.0': wet.			37/50-6"	16		
45		- @45.0': Coarse, black sand grains.			16/50-6"			
50					50-6"	17		
		End of boring at 50.5 feet bgs. Groundwater encountered at 24.5 feet. Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.						

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No. 20-81-168-01

Drawing No. A-4b

Log of Boring No. BH-04

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 844 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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1" in largest dimension, trace clay, loose, moist, orangish brown, slight desiccation.			2/4/5	8	111	
		OLD ALLUVIAL FAN DEPOSITS **SILTY SAND (SM):** fine to coarse-grained, trace clay, dense, moist, reddish brown, roots and rootlets.			10/22/31	5	127	cl
5								
		- @6.0': very dense.			50-5"	5	119	
		- @9.0' moderate desiccation.			32/50-2"	6	113	
10								
		- @14.0': dark reddish brown.			50-6"	10	95	
15								
		End of boring at 17.0 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.						

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No. **20-81-168-01**

Drawing No. **A-5**

Log of Boring No. BH-05

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 841 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	SUMMARY OF SUBSURFACE CONDITIONS	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
	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.				
0	**TOPSOIL** **SILTY SAND (SM):** fine to coarse-grained sand, few gravel up to 1" in largest dimension, trace clay, dense, moist, dark orangish brown, slight desiccation.	12/22/23	13	117	r
5	**OLD ALLUVIAL FAN DEPOSITS** **SILTY SAND (SM):** fine to coarse-grained, trace clay, medium dense, moist, reddish brown.	5/11/12	12	112	cu
		20/50-6"	12	113	
10		16/32/45	9	129	
15	- @14.0': dark reddish brown.	9/17/33	8	117	

End of boring at 17.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.

Log of Boring No. PT-01

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 831 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)	Description	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
	ARTIFICIAL FILL **SILTY SAND (SM):** fine to coarse-grained, trace gravel up to 1" in largest dimension, medium dense, moist, reddish brown. - @2.0': abundant organic and debris, black.	■		4/12/26	13	97	
5	**TOPSOIL** **SILTY SAND (SM):** fine to medium-grained, dense, dry, orangish brown.	■		8/9/12	3	117	
	- @6.0': loose, moist.	■		2/3/2	5		

End of boring at 7.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.

Log of Boring No. PT-02

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 832 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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, trace gravel up to 1" in largest dimension, loose, moist, brown.	■		5/4/6	5	104	
		- @ 3.0': medium dense.	■		4/5/9	19	66	
5		End of boring at 5.0 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.						

Log of Boring No. PT-03

Dates Drilled: 8/25/2020 Logged by: Catherine Nelson Checked By: Robert Gregorek

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

Ground Surface Elevation (ft): 836 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	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1" in largest dimension, trace clay, medium dense, moist, orangish to reddish brown.	■		7/9/9	6	116	
5			■		5/5/6	7	111	
		End of boring at 5.5 feet bgs. No groundwater encountered. Borehole backfilled with soil cuttings and tamped with auger using weight of drill rig on 08/25/2020.						

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the Logs of Borings, in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

In-Situ **Moisture Content and Dry Density**

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance with ASTM Standard D2216 and D2937 to aid soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Logs of Borings in Appendix A, Field Exploration.

Expansion Index

One representative bulk sample was tested to evaluate the expansion potential of materials encountered at the site in accordance with ASTM D4829 Standard. The test result is presented in the following table.

Table No. B-1, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-02	2-5	Silty Sand (SM)	2	Very Low

R-value

One representative bulk soil sample was tested for resistance value (R-value) in accordance with California Test Method CT301. This test is designed to provide a relative measure of soil strength for use in pavement design. The test result is presented in the following table.

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

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-05	1-5	Silty Sand, trace clay (SM)	20

Soil Corrosivity

One representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of the test was to determine the corrosion potential of sites soils when placed in contact with common construction materials. The test was performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.

Table No. B-3, Summary of Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-02	2-5	8.3	64	40	2,720

Collapse

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, one collapse test was performed in accordance with the ASTM Standard D4546 laboratory procedure. The sample was loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test result is presented in the following table.

Table No. B-4, Collapse Test Result

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-04	3.0-4.5	Silty Sand (SM)	-2.0	Low

Maximum Dry Density and Optimum Moisture Content

Laboratory maximum dry density-optimum moisture content relationship tests were performed on two representative bulk samples. These tests were conducted in accordance with the ASTM Standard D1557 test method. The test results are presented in Drawing No. B-1, *Moisture-Density Relationship Results,* and is summarized in the following table.

Table No B-6, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-01	0-2.5	Silty Sand (SM)	11.7	127.0
BH-02	2-5	Silty Sand (SM)	9.2	130.5

Organic Content

One Test was performed on five select samples of onsite soils to determine the organic content, in accordance with the ASTM Standard D2974 test, Methods A and C. Test results are summarized in the table below.

Table No. B-1, Summary of Organic Content Test Results

Test Pit No.	Depth (feet)	Soil Description	Total Organic Content (%)
PT-01	2.0-3.5	Silty Sand (SM), Some Organics	25.1

Direct Shear

Two direct shear tests were performed; one direct shear test was performed on a relatively undisturbed sample and another direct shear test was performed on a sample remolded to 90% of the maximum dry density under soaked moisture conditions in accordance with ASTM D3080. For these tests, 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 0.025 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-2 and B-3*, Direct Shear Test Results,* and the following table.

Table No. B-7, Summary of Direct Shear Test Results

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01	6.0-7.5	Silty Sand (SM)	37.0	90.0
BH-02*	1.0-5.0	Silty Sand (SM)	33.0	100.0

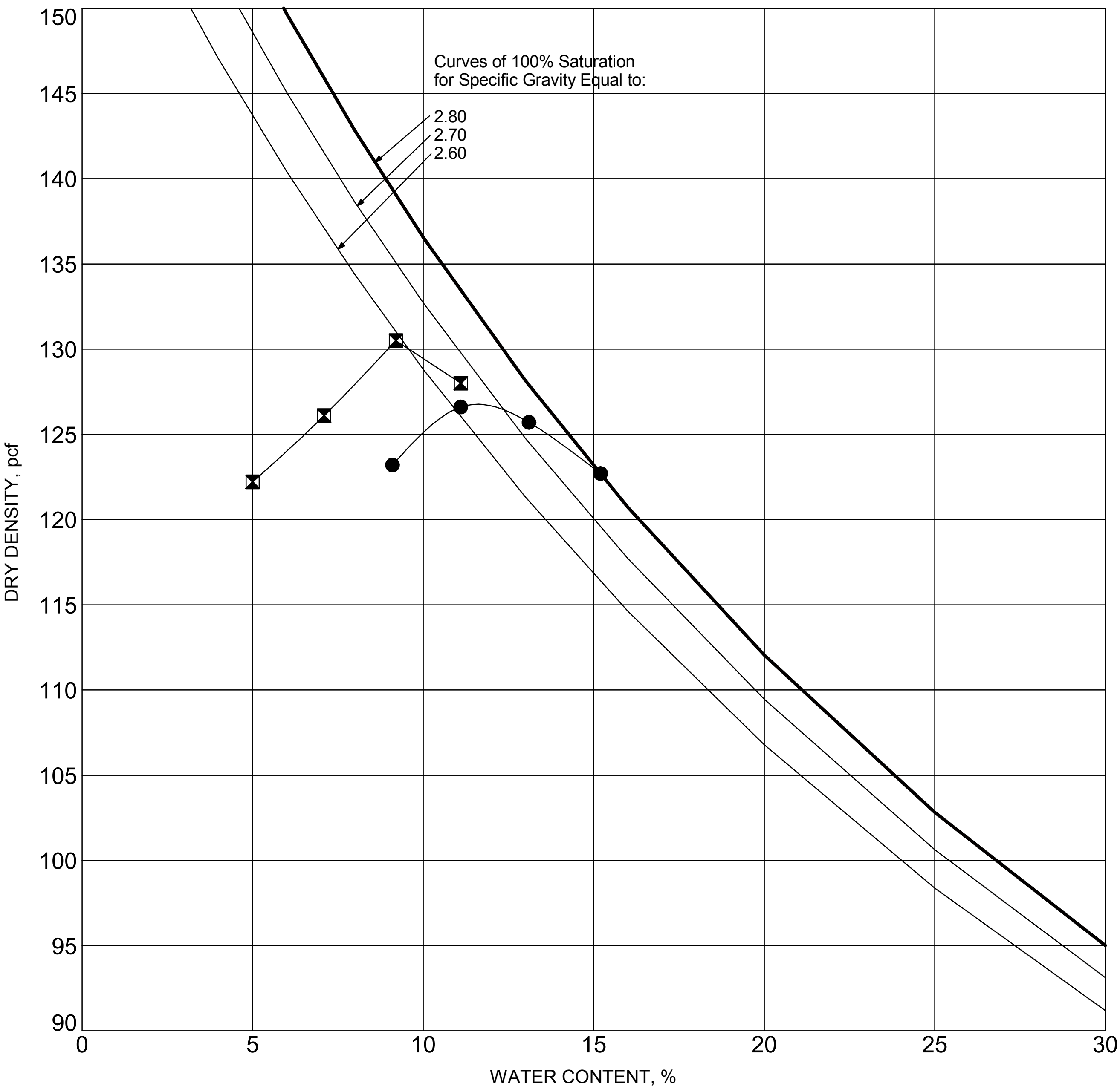
*(*Sample remolded to 90% of the maximum dry density)*

Consolidation

One test was conducted in accordance with ASTM Standard D2435 method. Data obtained from the test performed on relatively undisturbed ring samples was used to evaluate the settlement characteristics of the on-site soils under load. Preparation for these tests involved trimming the sample, placing it in a 1-inch-high brass ring, and loading it into the test apparatus, which contained porous stones 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. For test results, including sample density and initial moisture content, see Drawing No. B-4, *Consolidation Test Results*.

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.



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**0 to 2.5**	**Silty Sand (SM), reddish brown**	**D1557 - A**	**11.7**	**127**
⧗	**BH-02**	**2 to 5**	**Silty Sand (SM), reddish brown**	**D1557 - A**	**9.2**	**130.5**

MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

36 Unit Residential Development
7586 Jurupa Road
City of Jurupa Valley, Riverside County, California
For: All-ERA Properties

Project No.
20-81-168-01

Drawing No.
B-1

APPENDIX C

PERCOLATION TESTING

Percolation testing was performed at three locations (PT-01 through PT-03) on August 25 and 27, 2020. The testing was in general accordance with the Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011). The percolation testing method was used to estimate infiltration rates.

Upon completion of drilling the test holes, approximately 2-inch thick gravel layer was placed at the bottom of each hole and a 2.0-inch diameter perforated pipe was installed above the gravel to the ground surface. The boring annulus around the pipe was filled with gravel. The purpose of the pipe and gravel was to reduce the potential for erosion and caving due to the addition of water to the hole.

Each test hole was presoaked by filling with water to at least 5 times the radius of the test hole. More than 6 inches of water seeped into the test holes in less than 25 minutes for 2 consecutive measurements, meeting the criteria for testing as “sandy soil”. Percolation testing was conducted immediately after presoaking. During testing, the water level and total depth of the test hole were measured from the top of the pipe every 10 minutes for one hour. Following the completion of percolation testing, the pipe was removed from each test hole and the percolation test hole was backfilled with soil cuttings, tamped, and patched with concrete mixed with black dye.

Percolation rates describe the movement of water horizontally and downward into the soil from a boring. Infiltration rates describe the downward movement of water through a horizontal surface, such as the floor of a retention basin. Percolation rates are related to infiltration rates but are generally higher and require conversion before use in design. The percolation test data was used to estimate infiltration rates using the Porchet Inverse Borehole Method, in accordance with the Riverside County guidelines. A factor of safety of 3 was applied to the measured infiltration rates to account for subsurface variations, uncertainty in the test method, and future siltation. The infiltration structure designer should determine whether additional design-related safety factors are appropriate.

The measured percolation test data, calculations and estimated infiltration rates are shown on Plates Nos. 1 through 6. The estimated infiltration rates at the test holes are presented in the following table.

Table C-1, Estimated Infiltration Rates

Percolation Test	Test Depth (feet)	Soil Type	Infiltration Rate (inches/hour) (FOS 3)
PT-01	6.0	Silty Sand (SM)	0.41
PT-02	5.0	Silty Sand (SM)	6.29
PT-03	4.5	Silty Sand (SM)	0.34

Based on the calculated infiltration rate during the final respective intervals in each test, an average infiltration rate of 2.35 inches per hour can be utilized.

Estimated Infiltration Rate from Percolation Test Data, PT-01

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-01
Test Location	SW corner of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/28/2020

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	72
Inside Diameter of Pipe, I (inches)	2.88
Outside Diameter of Pipe, O (inches)	3.13
Factor of Safety (FOS), F	3

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	25.00	36.00	48.60	25.00	36.00	23.40	12.60	29.70	1.91		0.64
2	25.00	36.00	46.92	50.00	36.00	25.08	10.92	30.54	1.61		0.54
3	10.00	36.00	42.60	60.00	36.00	29.40	6.60	32.70	2.28		0.76
4	10.00	36.00	40.92	70.00	36.00	31.08	4.92	33.54	1.66		0.55
5	10.00	36.00	39.72	80.00	36.00	32.28	3.72	34.14	1.24		0.41
6	10.00	36.00	39.96	90.00	36.00	32.04	3.96	34.02	1.32		0.44
7	10.00	36.00	40.80	100.00	36.00	31.20	4.80	33.60	1.62		0.54
8	10.00	36.00	39.72	110.00	36.00	32.28	3.72	34.14	1.24		0.41
9	10.00	36.00	40.80	120.00	36.00	31.20	4.80	33.60	1.62		0.54
10	10.00	36.00	41.16	130.00	36.00	30.84	5.16	33.42	1.75		0.58
11	10.00	36.00	39.72	140.00	36.00	32.28	3.72	34.14	1.24		0.41
12	10.00	36.00	39.84	150.00	36.00	32.16	3.84	34.08	1.28		0.43

Recommended Design Infiltration Rate (inches/hr)	**0.41**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

Plate No.
1

Infiltration Rate versus Time, PT-01

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-01
Test Location	SW corner of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/28/2020

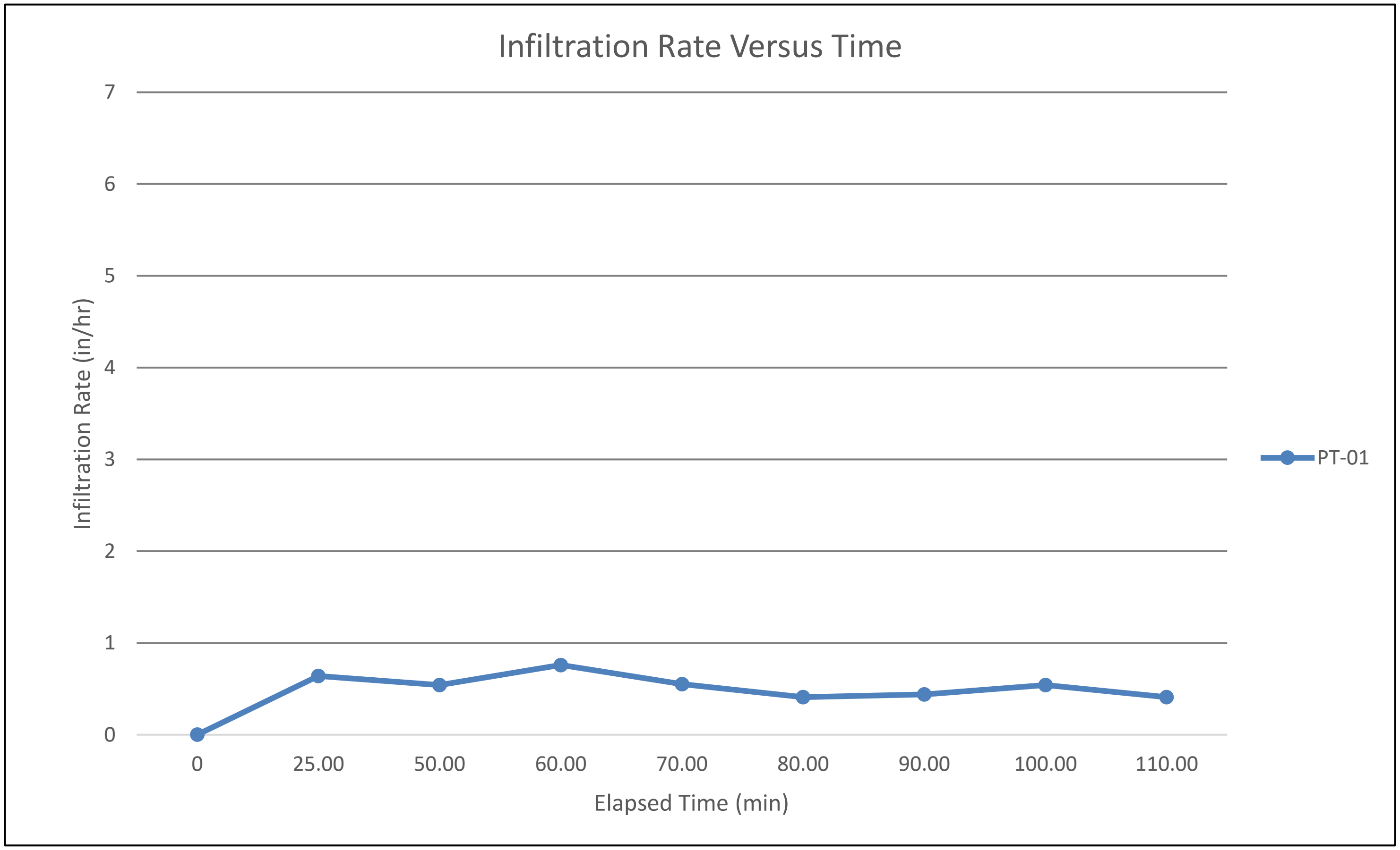


Plate No.
2

Estimated Infiltration Rate from Percolation Test Data, PT-02

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-02
Test Location	S center of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/28/2020

Shaded cells contain calculated values.

Test Hole Radius, r (inches)	4
Total Depth of Test hole, D_T (inches)	60
Inside Diameter of Pipe, I (inches)	2.88
Outside Diameter of Pipe, O (inches)	3.13
Factor of Safety (FOS), F	3

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	25.00	33.60	58.60	25.00	26.40	1.40	25.00	13.90	7.55		2.52
2	25.00	33.60	58.60	50.00	26.40	1.40	25.00	13.90	7.55		2.52
3	10.00	33.60	58.60	60.00	26.40	1.40	25.00	13.90	18.87		6.29
4	10.00	33.60	58.60	70.00	26.40	1.40	25.00	13.90	18.87		6.29
5	10.00	33.60	58.60	80.00	26.40	1.40	25.00	13.90	18.87		6.29
6	10.00	33.60	58.60	90.00	26.40	1.40	25.00	13.90	18.87		6.29
7	10.00	33.60	58.60	100.00	26.40	1.40	25.00	13.90	18.87		6.29
8	10.00	33.60	58.60	110.00	26.40	1.40	25.00	13.90	18.87		6.29
9	10.00	33.60	58.60	120.00	26.40	1.40	25.00	13.90	18.87		6.29
10											
11											
12											

Recommended Design Infiltration Rate (inches/hr)	**6.29**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

Plate No.
1

Infiltration Rate versus Time, PT-02

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-02
Test Location	S center of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/28/2020

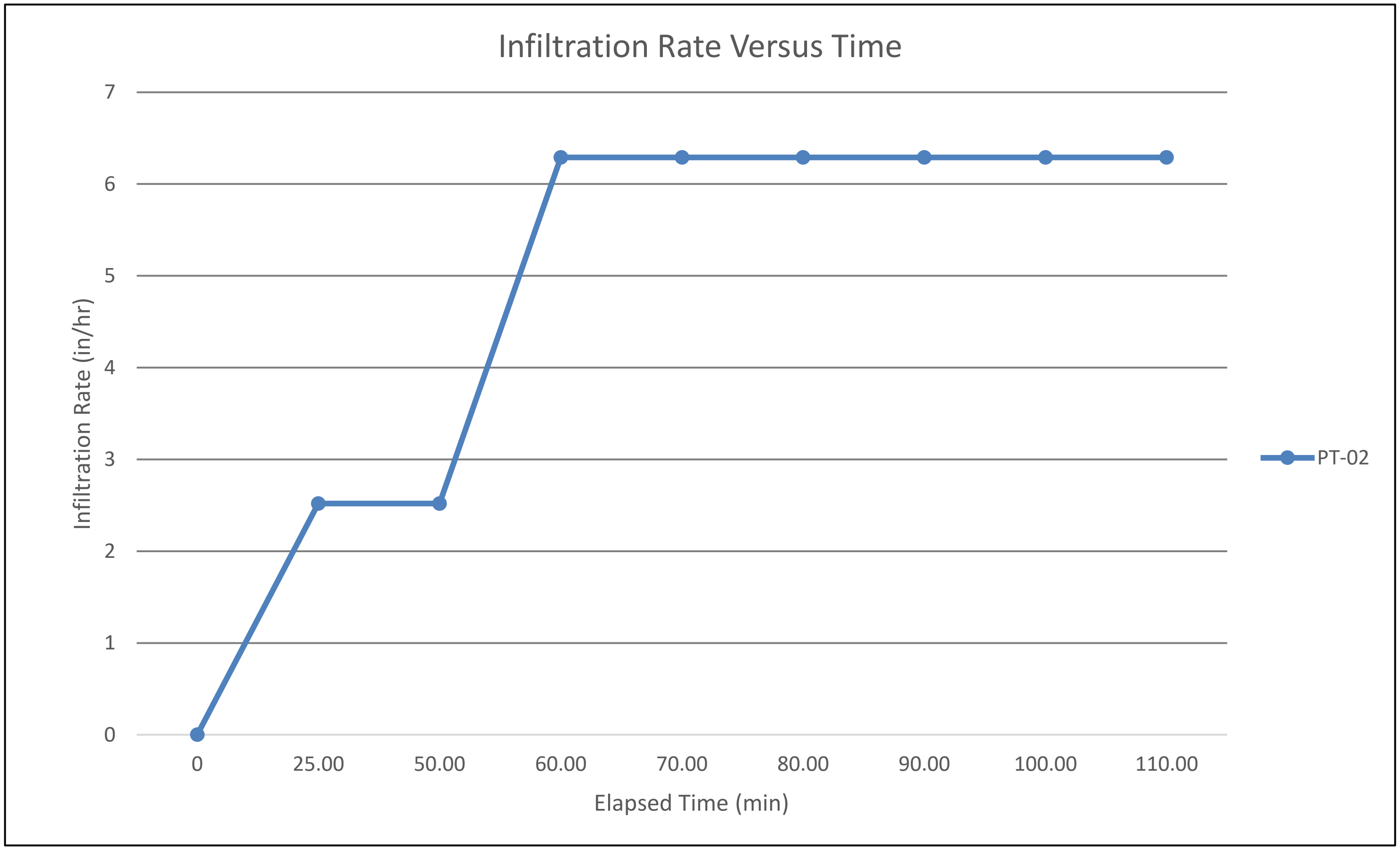


Plate No.
2

Estimated Infiltration Rate from Percolation Test Data, PT-03

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-03
Test Location	SE corner of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/30/2020

Shaded cells contain calculated values.

Test Hole Radius, r (inches)			4
Total Depth of Test hole, D_T (inches)			54
Inside Diameter of Pipe, I (inches)			2.88
Outside Diameter of Pipe, O (inches)			3.13
Factor of Safety (FOS), F			3

Interval No.	Time Interval, Δt (min)	Initial Depth to Water, D_0 (inches)	Final Depth to Water, D_f (inches)	Elapsed Time (min)	Initial Height of Water, H_0 (inches)	Final Height of Water, H_f (inches)	Change in Height of Water, ΔH (inches)	Average Head Height, H_{avg} (inches)	Infiltration Rate, I_t (inches/hr)		Infiltration Rate with FOS, I_f (inches/hr)
				0							0
1	25.00	36.00	41.04	25.00	18.00	12.96	5.04	15.48	1.38		0.46
2	25.00	36.00	41.04	50.00	18.00	12.96	5.04	15.48	1.38		0.46
3	30.00	36.00	40.92	80.00	18.00	13.08	4.92	15.54	1.12		0.37
4	30.00	36.00	40.92	110.00	18.00	13.08	4.92	15.54	1.12		0.37
5	30.00	36.00	40.92	140.00	18.00	13.08	4.92	15.54	1.12		0.37
6	30.00	36.00	40.92	170.00	18.00	13.08	4.92	15.54	1.12		0.37
7	30.00	36.00	40.56	200.00	18.00	13.44	4.56	15.72	1.03		0.34
8	30.00	36.00	40.56	230.00	18.00	13.44	4.56	15.72	1.03		0.34
9	30.00	36.00	40.56	260.00	18.00	13.44	4.56	15.72	1.03		0.34
10	30.00	36.00	40.56	290.00	18.00	13.44	4.56	15.72	1.03		0.34
11	30.00	36.00	40.56	320.00	18.00	13.44	4.56	15.72	1.03		0.34
12	30.00	36.00	40.56	350.00	18.00	13.44	4.56	15.72	1.03		0.34

Recommended Design Infiltration Rate (inches/hr)	**0.34**

Infiltration calculations are based on the Porchet Inverse Borehole Method presented in Riverside County BMP Design Handbook, Appendix A, Infiltration Testing (Riverside County, 2011)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

$H_{avg} = (H_0 + H_f) / 2$

$I_t = (\Delta H * (60 * r)) / (\Delta t * (r + (2 * H_{avg}))$

Plate No.
1

Infiltration Rate versus Time, PT-03

Project Name	All-ERA 44-unit Development
Project Number	20-81-168-01
Test Number	PT-03
Test Location	SE corner of site
Personnel	Catherine Nelson
Presoak Date	8/28/2020
Test Date	8/30/2020

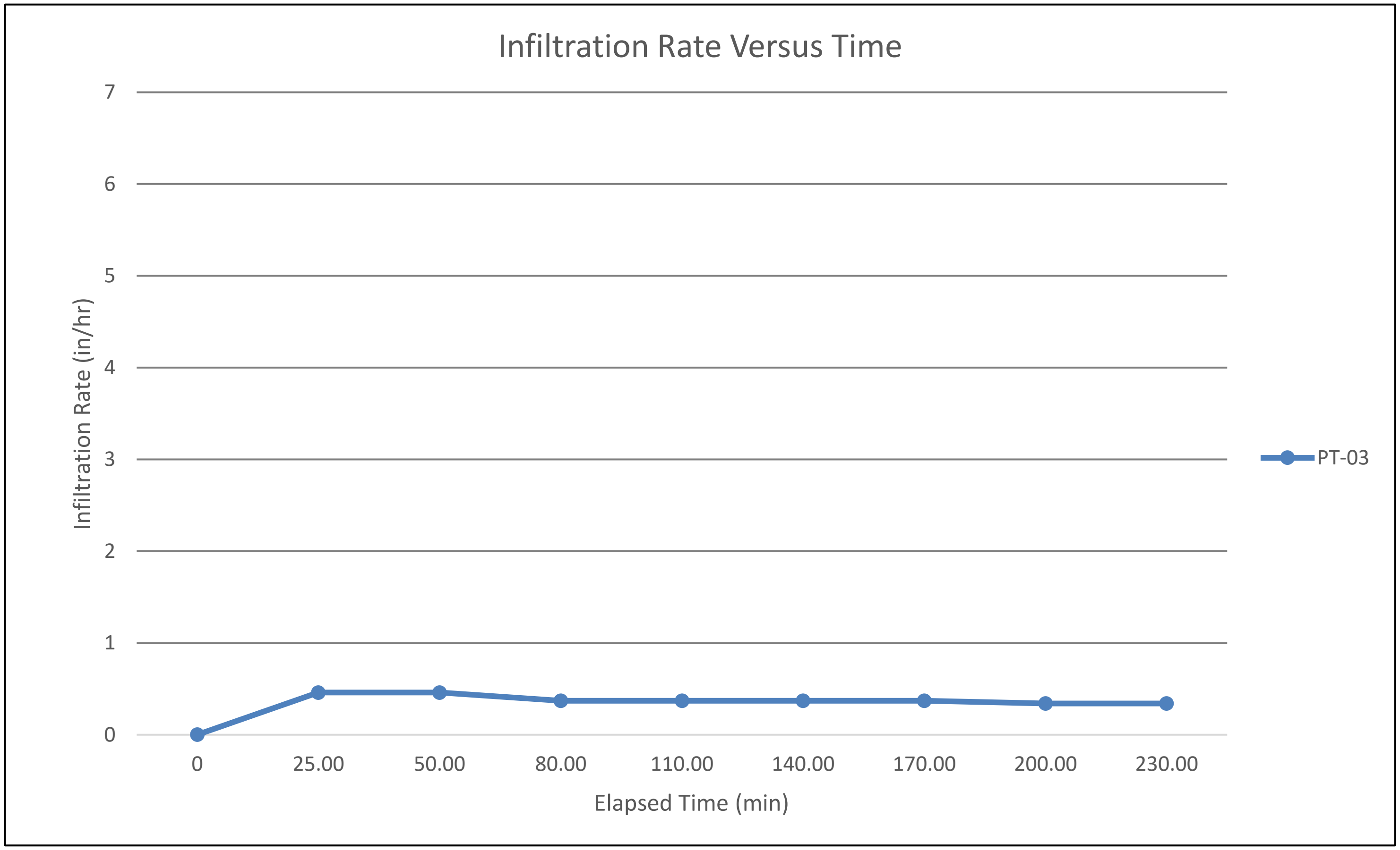


Plate No.
2