# UPDATED PRELIMINARY GEOTECHNICAL INVESTIGATION & WATER INFILTRATION TEST REPORT

## LINDEN BLOOMINGTON CONDOS, TENTATIVE TRACT 20481

10598 ORCHARD STREET
BLOOMINGTON AREA, SAN BERNARDINO COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 21-81-176-02

*Prepared For:*

**ALL-ERA PROPERTIES, LLC**
P.O. Box 11503
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*Presented By:*


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September 9, 2022

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

September 9, 2022

Mr. Byron Walker
Owner
All-ERA Properties, LLC
P.O. Box 11503
Carson, CA 90749

Subject: **UPDATED PRELIMINARY GEOTECHNICAL INVESTIGATION AND WATER INFILTRATION TEST REPORT**
**Linden Bloomington Condos, Tentative Tract 20481**
10598 Orchard Street
Bloomington Area, San Bernardino County, California
Converse Project No. 21-81-176-02

Dear Mr. Walker:

Converse Consultants (Converse) has prepared this updated geotechnical investigation and water infiltration test report to present the findings, conclusions and recommendations, for the proposed Linden Bloomington Condos residential development project**,** Tentative Tract 20481, located at 10598 Orchard Street in the Bloomington Area, San Bernardino County, California. This report is prepared in accordance with our proposal dated June 26, 2021, and your e-mail acceptance of the Agreement and Authorization to Proceed, dated August 23, 2022.

Based upon our field investigation, laboratory data, and analyses, as well as review of the referenced conceptual grading plan, the proposed project is considered feasible 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, LLC. 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.:1/Addressee (electronic)
HSQ/RLG/kvg

# 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.

Robert L Gregorek II, PG, CEG
Senior Geologist

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

# TABLE OF CONTENTS

## FIGURES



## TABLES

## APPENDICES

# 1.0 INTRODUCTION

This report contains the findings of the updated preliminary geotechnical investigation and percolation tests performed by Converse for the proposed Linden Bloomington Condos residential development project**,** Tentative Tract 20481, located at 10598 Orchard Street in the Bloomington Area, San Bernardino County, California. The project location is shown in Figure No. 1, *Approximate Site 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.

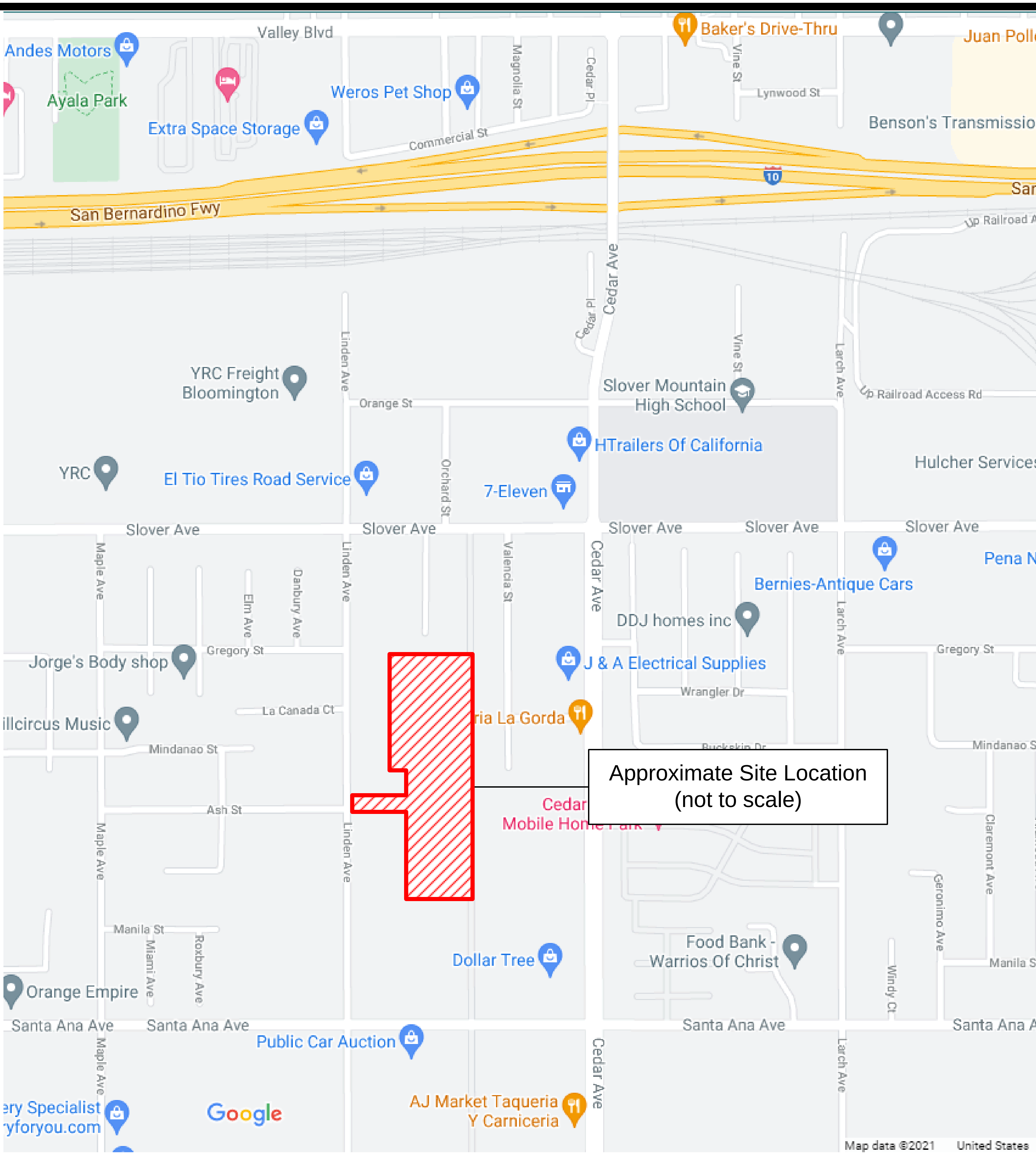
This updated report is written for the project described herein and is intended for use solely by All-ERA Properties, LLC and their design team. It should not be used as a bidding document but may be made available to the potential contractors for information on factual data only. For bidding purposes,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

# 2.0 PROJECT DESCRIPTION

Based on the review of the referenced tentative tract map, as well conversations with Mr. Kevin Kent of TK Management and Mr. Aaron Skeers of Encompass Associates, Inc., the proposed development will now consist of 180 one to two-story single-family residential buildings and are anticipated to be wood framed structures founded on shallow footings with slab-on-grade construction. There will also be one water infiltration device, approximately 10 feet to 15 feet deep, at the southern portion of the site. Associated with the development will be roadways, parking areas, concrete walkways, paseos, open space areas, block walls, above and underground utilities as well as landscaping. Based on the referenced tentative tract map, grading will consist of cuts and fills of approximately 7 feet and 3 feet, respectively. Cut or fill slopes, as well as retaining wall are not indicated on the subject plan.

# 3.0 SITE DESCRIPTION

The site is now approximately 12.9 acres in size, from the previous approximately 11.5 acres and is still currently vacant undeveloped land. The site is located at the south end of Orchard Street and is bounded on the north and west by residential developments, on the east by San Bernardino County Flood Control District right of way and on the south by vacant land and some residential structures. Some scattered trash and debris were observed on the site. Vegetation consists of a light to heavy growth of grass and weeds with some scattered bushes and trees at the northeast portion of the site. The site is roughly flat and appears to drain towards the south and southeast. Elevations range from approximately 1,062 feet above mean sea level (msl) in the northwest

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# Approximate Site Location Map

Project: Linden Bloomington Condos, Tentative Tract 20481
Location: 10598 Orchard Street
Bloomington Area, San Bernardino County, California
For: All-ERA Properties, LLC

Project No
21-81-176-01

**Converse Consultants**

FIGURE NO.
**1**

portion of the site to approximately 1,042 feet above msl in the southeast portion of the site.

Present site conditions are shown below in the Photograph Nos. 1 and 2.

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

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

# 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*

Six exploratory borings (BH-01 through BH-06) were drilled on August 02, 2021, to investigate the subsurface conditions at the project site. The drilling was performed with a CME-75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers and a drive sampler for soil sampling. The borings were drilled to depths ranging from approximately 13.5 to 51.0 feet below existing ground surface (bgs).

Three exploratory borings (BH-01/PT-01 through BH-03/PT-03) were prepared for percolation testing. Percolation test borings were drilled to depths ranging from approximately 13.5 to 16.5 feet below the existing ground surface (bgs).

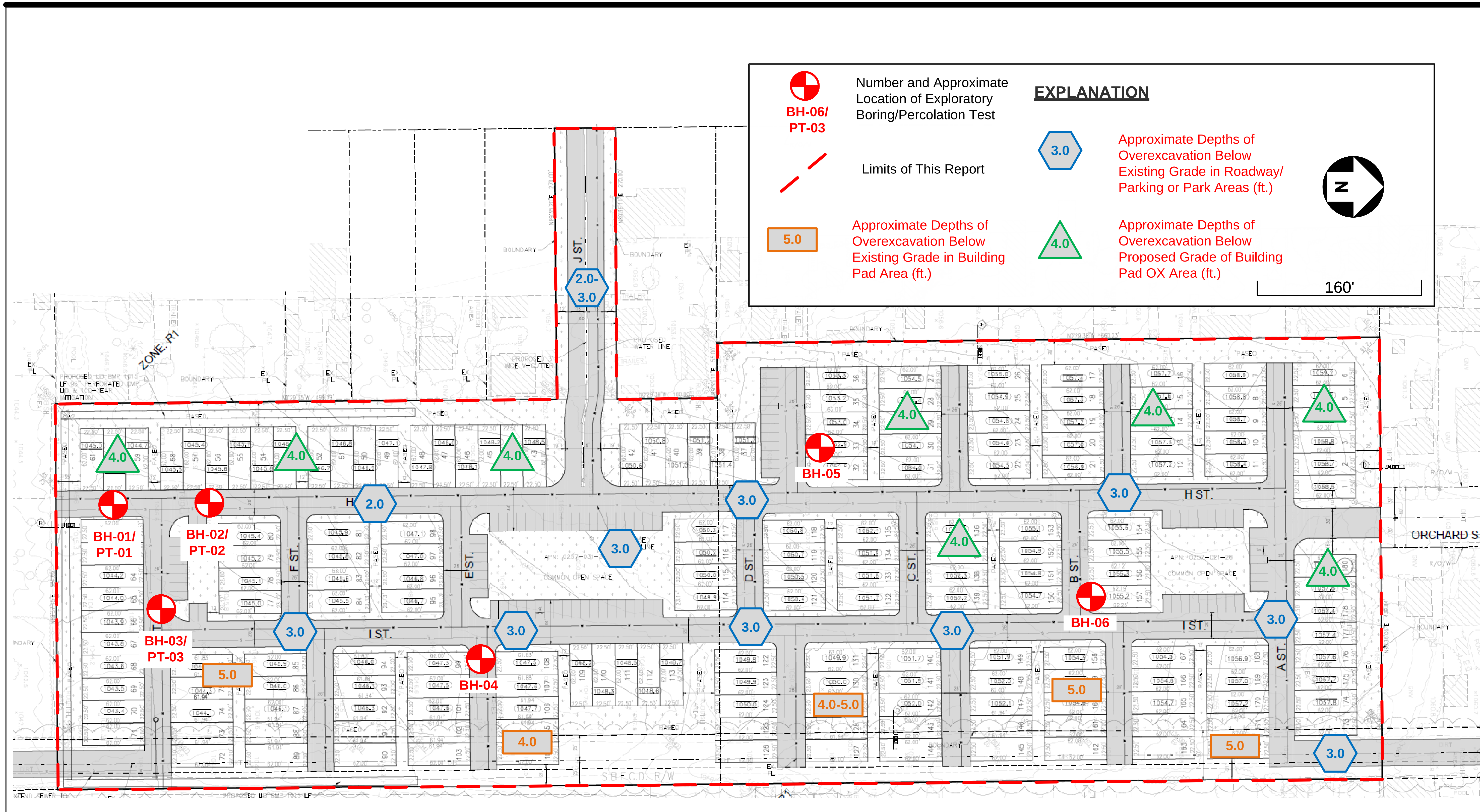
Approximate boring and percolation testing locations are indicated in Figure No. 2, *Approximate Boring, Percolation Test, and Overexcavation Locations Map*. For a description of the exploration and sampling program, see Appendix A, *Field Exploration*.

## *4.3 Site Reconnaissance*

A Converse geologist conducted a current site reconnaissance to make observations and document the existing geotechnical and geologic surface site conditions, on September 8, 2022. This was accomplished in order to determine if there were any significant changes to the site since our field observations in August 2021. No significant changes were observed.

## *4.4 Laboratory Testing*

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




Project: Linden Bloomington Condos, Tentative Tract 20481

Location: 10598 Orchard Street
Bloomington Area, San Bernardino County, California

For: All-ERA Properties, LLC

# Approximate Boring, Percolation Test and Overexcavation Locations Map

Project No. **21-81-176-02**

Converse Consultants

Figure No. **2**

- *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)
- Grain size Analysis (ASTM 6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)

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*.

### *4.5 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.

## 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 the exploratory borings and laboratory test results, the subsurface at the project site generally consisted primarily of young and old alluvial fan deposits.

The various subsurface profiles and description of the earth material soils encountered are discussed below.

<u>Young Alluvial Fan Deposits:</u> Holocene-aged young alluvial fan deposits were encountered in all of the exploratory borings below the surface. These materials were comprised of sand, silty sand and sandy silt which are fine to coarse-grained, has little to some gravel up to 3 inches in maximum dimension, locally slightly to moderately desiccated, some oxidation staining, medium dense to very dense/stiff to very stiff, dry to moist and are various shades of gray, brown, red and yellow. Where observed, in boring BH-04, these materials were approximately 36.5 feet thick.

<u>Old Alluvial Fan Deposits:</u> Late to Middle Pleistocene-aged older alluvial deposits were encountered in exploratory boring BH-04 below the young alluvial fan deposits at a depth of approximately 36.5 feet bgs. These materials were comprised of sand and silty sand which are fine to coarse-grained, has little gravel up to 3 inches in maximum

dimension, some cobbles, locally moderately desiccated, very dense, dry and are various shades of gray, brown and red.

For a detailed description of the subsurface materials encountered in the exploratory borings, see the logs, Drawings No. A-2 through A-7, in Appendix A, *Field Exploration.*

### *5.2 Groundwater*

Groundwater was not encountered during our field investigation in any borings, to the maximum depths explored of 51.0 feet bgs. The GeoTracker database (SWRCB, 2021) was reviewed for groundwater data from sites within an approximately 1.0-mile radius of the proposed development, but no results were found.

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 |
|---|---|---|---|
| 340402117234501 | W end of Cedar Place; approximately 2194 feet north of project site | 250.94-260.81 | 2001-2008 |
| 340402117234601 | W end of Cedar Place; approximately 2185 feet north of project site | 240-288 | 1956-2001 |

The California Department of Water Resources database (DWR, 2021) was reviewed for historical groundwater data from sites within a 1.0-mile radius of the project site. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Details of that record are listed below.

- Well No. Santa Fe Gas 2A (Station 340470N1174020W0011), located approximately 4,164 feet south of the project site, reported groundwater at depths ranging from 176.33-187.16 feet bgs between 2011-2021.

Based on available data, the historical high groundwater level near the site is estimated to be approximately 176 feet bgs, and the current groundwater level is estimated to be deeper than 51.0 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 and trenching equipment. <u>However, difficult excavation may occur, approximately 8 feet to 10 feet bgs, due to high concentrations of gravel and the very nature of the alluvial fan deposits.</u>

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 6 feet of the site soils was 0, corresponding to a very low expansion potential.

## 6.0 ENGINEERING GEOLOGY

The regional and local geology within the proposed project area is 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 available geological and geotechnical literature (Dibblee and Minch, 2004; Morton and Miller, 2006) as well as the results of our exploration and laboratory testing, it is our understanding that the site is primarily underlain by young and old alluvial fan deposits, comprised of sand, silt and gravel with some cobbles.

### *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. 2, Summary of Regional Faults**

| Fault Name and Section | Closest Distance (km) | Slip Sense | Length (km) | Slip Rate (mm/year) | Maximum Magnitude |
|---|---|---|---|---|---|
| San Jacinto | 8.15 | strike slip | 241 | n/a | 7.80 |
| Cucamonga | 13.53 | thrust | 28 | 5 | 6.70 |
| S. San Andreas | 16.68 | strike slip | 548 | n/a | 8.18 |
| Cleghorn | 25.01 | strike slip | 25 | 3 | 6.80 |
| San Jose | 27.55 | strike slip | 20 | 0.5 | 6.70 |
| Chino, alt 1 | 28.65 | strike slip | 24 | 1 | 6.70 |
| Chino, alt 2 | 28.71 | strike slip | 29 | 1 | 6.80 |
| Elsinore | 30.75 | strike slip | 241 | n/a | 7.85 |
| North Frontal (West) | 30.85 | reverse | 50 | 1 | 7.20 |
| Sierra Madre | 32.27 | reverse | 57 | 2 | 7.20 |
| Sierra Madre Connected | 32.27 | reverse | 76 | 2 | 7.30 |
| Clamshell-Sawpit | 45.81 | reverse | 16 | 0.5 | 6.70 |
| Puente Hills (Coyote Hills) | 46.94 | thrust | 17 | 0.7 | 6.90 |

(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) and ASCE 7-16 are provided in the following table. These parameters were determined using the generalized coordinates (34.0606N, 117.3993W) and the Seismic Design Maps ATC online tool.

**Table No. 3, CBC Seismic Design Parameters**

| Seismic Parameters | |
|---|---|
| Site Coordinates | 34.0606 N, 117.3993 W |
| Site Class | D* |
| Risk Category | II |
| Mapped Short period (0.2-sec) Spectral Response Acceleration, $S_s$ | 1.550g |
| Mapped 1-second Spectral Response Acceleration, $S_1$ | 0.601g |
| Site Coefficient (from Table 1613.5.3(1)), $F_a$ | 1.00 |
| Site Coefficient (from Table 1613.5.3(2)), $F_v$ | 1.70 |
| MCE 0.2-sec period Spectral Response Acceleration, $S_{MS}$ | 1.550g |
| MCE 1-second period Spectral Response Acceleration, $SM_1$ | 1.022g |
| Design Spectral Response Acceleration for short period $S_{DS}$ | 1.033 g |
| Design Spectral Response Acceleration for 1-second period, $S_{D1}$ | 0.681g |
| Site Modified Peak Ground Acceleration, $PGA_M$ | 0.724g |

** 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 or San Bernardino County Earthquake Fault Zone (CGS, 2007; SBC, 2021b). 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 not located in a State of California or San Bernardino County designated liquefaction zone (CGS, 2007; SBC 2021b). Based on the lack of shallow groundwater (within 50.5 feet bgs), dense soil conditions and high blow counts, liquefaction potential 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. Based on the relatively dense nature of the soils, high blow counts and recommended remedial grading, 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 in a State of California or San Bernardino County designated landslide susceptibility area. The 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 very 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. The project site is not located in a State of California or County of San Bernardino designated dam inundation zone (DSOD, 2021; SBC 2021a). 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*

- <u>In-situ Moisture and Dry Density:</u> *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 upper 10 feet ranged from 105 to 129 per cubic feet (pcf) with moisture contents ranging from 1 to 9 percent.
  - Dry densities of the below the upper 10 feet of soils at the site ranged from 99 to 125 pcf with moisture contents ranging from 1 to 11 percent.
- <u>Expansion Index:</u> Two representative bulk soil samples from the upper 6 feet of the site materials were tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test results both indicated expansion indices of 0, corresponding to very low expansion potential.
- <u>R-Value:</u> Two representative bulk samples were tested in accordance with Caltrans Test Method 301. The results of the R-value tests were 67 and 77.
- <u>Grain Size Analysis</u> – Three representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. The test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.*
- <u>Maximum Dry Density and Optimum Moisture Content:</u> 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 132.0 pounds per cubic feet (pcf), with optimum moisture contents of 7.0 and 5.5 percent, respectively.
- <u>Direct Shear:</u> One direct shear test was performed on a sample remolded to 90% of the maximum dry density under soaked moisture condition in accordance with ASTM Standard D3080. The result of the direct shear test is presented in Drawing No. B-3, *Direct Shear 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 7.5.
- The sulfate content of the sample tested was 21 ppm (0.0021 percent by weight ppm).
- The chloride concentration of the sample tested was 19 ppm.
- The minimum electrical resistivity when saturated was 12,753 ohm-cm.

## 9.0 PERCOLATION TESTING

Three percolation tests (PT-01 through PT-03) were performed on August 03, 2021, to estimate the water infiltration rate, within the area of the proposed water infiltration device, located in the southwest corner of the site. 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. 4, Estimated Infiltration Rates**

| Percolation Test | Test Depth (feet) | Soil Type | Infiltration Rate (inches/hr) (FOS 2) |
|---|---|---|---|
| PT-01 | 15.1 | Sand/Silty Sand, with Gravel (SP/SM) | 11.62 |
| PT-02 | 13.1 | Sand/Silty Sand, with Gravel (SP/SM) | 11.53 |
| PT-03 | 13.9 | Sand/Silty Sand, with Gravel (SP/SM) | 11.57 |

Based on the calculated infiltration rate during the final respective intervals in each test, an average infiltration rate of 11.57 inches per hour can be utilized for design.

## 10.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

Earthwork for the project will include grading, trench excavation, pipe subgrade preparation, pipeline bedding placement and trench backfill, as well as roadway pavement construction. Recommendations for earthwork are presented in the following subsections. General Earthwork Specifications are presented in Appendix D, *Earthwork Specifications.*

### *10.1 General*

This section contains our general recommendations regarding earthwork for the proposed Linden Bloomington Condos 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 debris, deleterious material 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*

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 feet to 5.0 feet of potentially compressible soils (upper low-density portions of the young alluvial fan deposits), which may be prone to future adverse settlement under the surcharge of foundation, improvements and/or fill loads. Therefore, these materials should be over-excavated to competent alluvial fan deposits, within all areas of proposed structures, walls and other improvements, and replaced with compacted fill soils.

*<u>Building Pad Areas:</u>* Within the entire level portions of the building pad areas overexcavations should be approximately 4.0 feet to 5.0 feet below existing grade or and least 4.0 feet below proposed grade, as well as 2.0 feet below the bottom of the proposed building footings, whichever is deeper. All over-excavations should extend laterally at least 5.0 feet or equal to the depth of over-excavation, whichever is greater, outside the entire level portions of the building pad area.

*<u>Improvements Outside of the Building Pad Areas</u>:* For areas of proposed roadways, parking, flatwork, walls and other improvements, overexcavations should be at least 2.0 to 3.0 feet below existing grade. Within wall areas overexcavations should also be a minimum of 2.0 feet below the proposed wall footings, all over-excavations should extend laterally at least 3.0 feet or equal to the depth of over-excavation, whichever is greater.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. However, localized deeper over-excavation could be encountered, based on observations and density testing by the geotechnical consultant during grading of the final bottom surfaces of all excavations.

The estimated locations and approximate depths of overexcavation of unsuitable, compressible soil materials are indicated on Figure No. 2, *Approximate Boring, Percolation Testing 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).

Following overexcavation 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 Cut/Fill Transition and Fill Differentials*

To mitigate distress to structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all level portions of the building pad areas. This should be accomplished by overexcavating the entire “cut” portion of the entire building pad area by at least 4.0 feet below proposed grade and replacing the excavated materials as properly compacted fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 2.0 feet of fill below footings for proposed structures and 2.0 feet below footings for proposed walls. Recommended depths of over-excavation are provided in the following table.

**Table No. 5, Overexcavation Depth for Cut/Fill Transitions**

| Depth of Fill ("Fill" Portion) | Depth of Overexcavation ("Cut" Portion) |
|---|---|
| Up to 12.0 feet | 4.0 feet |
| Greater than 12.0 feet | One-third the maximum thickness of fill placed on the "fill" portion (15 feet maximum) |

## *10.5 Engineered Fill*

No fill should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The existing 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 3 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 field investigation and laboratory testing results, on-sites soils may be suitable 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 the geotechnical consultant prior to delivery to the site.

## *10.6 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. Prior to placement of pavement sections at least the upper 1 foot 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.

### *10.7 Backfill Recommendations Behind Walls*

Compaction of backfill adjacent to perimeter wall or any retaining walls, which may be proposed in the future, can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls.

### *10.8 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 exploration, laboratory test results, as well as 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 to range from approximately 0 to 13 percent. An average value of 6 percent 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 0.15 foot to 0.20 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.

### *10.9 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.

### *10.10 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.

#### 10.10.1 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.

### 10.10.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Recommendations for pipe bedding are provided below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

The type and thickness of the granular bedding placed underneath and around the pipe, if any, should be selected by the pipe designer. The load on the rigid pipes and deflection of flexible pipes and, hence, the pipe design, depends on the type and the amount of bedding placed underneath and around the pipe.

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the spring line of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on 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.

### 10.10.3 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.
- 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 Shallow Foundation Design Parameters*

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

**Table No. 6, 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 | 2,500 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 3,500 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.

## *11.2 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.

### 11.2.1 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. 7, 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) | 40 | 60 |
| At-rest (wall is restrained) | 60 | 109 |

These pressures assume 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.

### 11.2.2 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 0.35 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 270 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 2,500 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.

### *11.3 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.

### *11.4 Slabs-on-Grade*

Slabs-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 10.6 *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.

### *11.5 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.

Based on the absence of shallow groundwater, within 50 feet bgs, dense nature of the soils and high blow counts, the potential dynamic settlement for the project site from liquefaction and dynamic differential settlement is considered negligible.

### *11.6 Expansion Potential*

Based on the results of the expansion testing of representative site soils, on-site soils have expansion index of 0.

The expansion indices of the final finish-grade soils will vary from the results obtained during our investigation. The expansion potential of the finish-grade soils should be confirmed by additional testing at the completion of grading and revise the foundation design parameters if necessary. During construction, the contractor should determine effective methods to minimize moisture variations.

### *11.7 Pipe Design for Underground Utilities*

Structural design of pipes requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on buried pipes depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for the pipe design are provided in Table No. 8, *Soil Parameters for Pipe Design*.

Where pipes are connecting to rigid structures near, or at its lower levels, and then are subjected to significant loads as the backfill is placed to finish grade, we recommend that provisions be incorporated in the design to provide support of these pipes where they exit the structure. Consideration can be given to flexible connections, concrete slurry support beneath the pipes where they exit the structures, overlaying and supporting the pipes with a few inches of compressible material, (i.e., Styrofoam, or other materials), or other techniques. Automatic shutoffs should be installed to limit the potential leakage from seismic event related damage.

**Table No. 8, Soil Parameters for Pipe Design**

| Soil Parameters | Parameters |
|---|---|
| Total unit weight of compacted backfill (assuming 92% average relative compaction), $\gamma$ | 128 pcf |
| Angle of internal friction of soils, $\phi$ | 32º |
| Soil cohesion, c | 0 psf |
| Coefficient of friction between concrete and native soils, fs | 0.35 |
| Coefficient of friction between pipe and compacted fill or native soils, fs | 0.25 for metal or HDPE pipe<br>0.30 for CML&C pipe |
| Bearing pressure against compacted fill or natural soils | 2,500 psf |
| Coefficient of passive earth pressure, Kp | 3.25 |
| Coefficient of active earth pressure, Ka | 0.31 |
| Modulus of Soil Reaction, E' | 1,500 psi |

## *11.8 Soil Corrosivity*

The results of chemical testing of a representative sample of site soils with respect to common construction materials such as concrete and steel are presented in Appendix B, *Laboratory Testing Program*, and a general discussion are presented below.

The sulfate content of the sampled soils 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. 9, 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 12,753 ohm-cm. This indicates that the soils tested are <u>mildly corrosive</u> for ferrous metals in contact with the soil (Romanoff, 1957). <u>Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for ferrous metals in contact with the site soils</u>.

### *11.9 Pavement Recommendations*

Two soil samples were tested to determine the R-value of the subgrade soils. Based on laboratory testing, the R-values were 67 and 77. For pavement design, we have utilized a maximum design R-value of 50 for 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, 2020), 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 Bloomington minimum asphalt pavement and aggregate base thickness requirements should also be considered in the pavement design.

**Table No. 10, Recommended Preliminary Pavement Sections**

| Design R-value | Traffic Index (TI) | Pavement Section | | |
|---|---|---|---|---|
| | | Option 1 | | Option 2 |
| | | Asphalt Concrete (inches) | Aggregate Base (inches) | Full AC Section (inches) |
| 50 | 5 | 3.0 | 4.0 | 4.5 |
| | 6 | 3.5 | 4.0 | 5.5 |
| | 7 | 4.0 | 4.5 | 7.0 |
| | 8 | 4.5 | 6.0 | 8.0 |

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 AC, at least the upper 1 foot 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.

### *11.10 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 1 foot 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 utilities or structures (if any) 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. 11, Slope Ratios for Temporary Excavations**

| Soil Type | OSHA Soil Type | Depth of Cut (feet) | Recommended Maximum Slope (Horizontal: Vertical)[1] |
|---|---|---|---|
| Silty Sand (SM) and Sandy Silt (ML) | 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, a slope ratio of 1:1 can be used 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 can be provided if requested.

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 All-ERA Properties, LLC 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

# 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. Description of the field investigation method is presented below.

Six borings (BH-01 through BH-06) were drilled on August 02, 2021, within the project site to investigate the subsurface conditions. The borings were drilled to depths ranging from approximately 13.5 to 51.0 feet below ground surface (bgs).

Three exploratory borings (BH-01 through BH-03) were utilized as percolation test holes (PT-01 through PT-03) to perform percolation testing. Percolation test borings were drilled to depths ranging from approximately 13.5 to 16.5 feet below the existing ground surface (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. Some ring samples collected from each borehole were disturbed or contained no soil recovery because of the poor consolidation and large grain sizes.

Standard Penetration Testing (SPT) was also performed in borings BH-04 and BH-05 in accordance with the ASTM Standard D1586 test method at 10-foot intervals beginning at 20 feet in both boreholes 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.

Representative bulk samples were collected from selected depths and placed in large plastic bags for delivery to our laboratory.

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, borings BH-04 through BH-06 were backfilled with soil cuttings and compacted by pushing down with the augers using the drill rig weight. Following the completion of logging, sampling and percolation testing in borings BH-01/PT-01 through BH-03/PT-03, the perforated pipes were removed and then the holes were backfilled with soil cuttings and were tamped from the surface. 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.

For a key to soil symbols and terminology used in the boring logs, refer to Drawing Nos. A-1a andA-1b, *Unified Soil Classification and Key to Boring Log Symbols*. For logs of borings, see Drawings No. A-2 through A-7, *Logs of Borings*.

# SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

## 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

### DRILLING METHOD SYMBOLS

Auger Drilling | Mud Rotary Drilling | Dynamic Cone or Hand Driven | Diamond Core

### 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

## UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG AND TEST PIT SYMBOLS

Converse Consultants

Linden Bloomington Condos, Tentative Tract 20481
10598 Orchard Street
Bloomington Area, San Bernardino County,
California For: All-ERA Properties, LLC

Project No. 21-81-176-01

Drawing No. A-1a

## 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.

# UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG AND TEST PIT SYMBOLS

Converse Consultants

Linden Bloomington Condos, Tentative Tract 20481
10598 Orchard Street
Bloomington Area, San Bernardino County,
California For: All-ERA Properties, LLC

| Project No. | Drawing No. |
|---|---|
| 21-81-176-01 | A-1b |

# Log of Boring No. BH-01/PT-01

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1047 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SAND/SILTY SAND WITH GRAVEL (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, roots and rootlets, medium dense, dry, brown to grayish brown. | X | X | 5/7/12 | 1 | 122 | EI, R |
| 5 | | | | X | | | | |
| | | - @6.0': dense | X | X | 14/21/25 | 1 | 111 | |
| 10 | | - @9.0': very dense | X | X | 7/32/50-6" | 1 | 119 | |
| | | - @12.0': dense | X | | 17/21/23 | 2 | 116 | |
| 15 | | - @15.0': very dense | X | | 36/36/41 | 1 | 125 | |

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole was utilized for percolation testing.
Perforated tube was installed and hole was presoaked on 08/02/2021.
After completion of percolation testing, pipe was removed and borehole was backfilled with soil cuttings and hand-tamped on 08/03/2021.

Converse Consultants

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For: All-ERA Properties, LLC

Project No. 21-81-176-01

Drawing No. A-2

Project ID: 21-81-176-01.GPJ; Template: LOG

# Log of Boring No. BH-02/PT-02

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1048 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SAND/SILTY SAND WITH GRAVEL (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, roots and rootlets, medium dense, dry, brown to grayish brown. | | | 9/11/13 | 1 | 112 | |
| 5 | | - @4.0': dense | | | 12/16/20 | 1 | 114 | |
| | | | | | 14/25/25 | 1 | 127 | |
| 10 | | - @10.0': very dense | | | 22/36/40 | 2 | 117 | |
| | | | | | 50-6" | | | *no recovery* |
| | | End of boring at 14.5 feet bgs.<br>No groundwater encountered.<br>Borehole was utilized for percolation testing.<br>Perforated tube was installed and hole was presoaked on 08/02/2021.<br>After completion of percolation testing, pipe was removed and borehole was backfilled with soil cuttings and hand-tamped on 08/03/2021. | | | | | | |

# Log of Boring No. BH-03/PT-03

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1046 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SAND/SILTY SAND WITH GRAVEL (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, roots and rootlets, medium dense, dry, brown to grayish brown. | | | 11/12/12 | 1 | 117 | |
| 5 | | **SILTY SAND (SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, dense, moist, reddish brown. | | | 19/20/16 | 3 | 109 | |
| | | **SAND/SILTY SAND (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, roots and rootlets, very dense, dry, brown to grayish brown. | | | 18/33/50 | 1 | 116 | |
| 10 | | | | | 26/27/36 | 2 | 117 | |
| | | | | | 50-6" | | | *no recovery* |
| | | End of boring at 13.5 feet bgs.<br>No groundwater encountered.<br>Borehole was utilized for percolation testing.<br>Perforated tube was installed and hole was presoaked on 08/02/2021.<br>After completion of percolation testing, pipe was removed and borehole was backfilled with soil cuttings and hand-tamped on 08/03/2021. | | | | | | |

# Log of Boring No. BH-04

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1050 Depth to Water (ft, bgs): NOT ENCOUNTERED

**SUMMARY OF SUBSURFACE CONDITIONS**

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

| Depth (ft) | Summary of Subsurface Conditions | Samples: Drive | Samples: Bulk | Blows | Moisture (%) | Dry Unit Wt. (pcf) | Other |
|---|---|---|---|---|---|---|---|
| 0 | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, some gravel up to 3" maximum dimension, roots and rootlets, medium dense, dry, light brown to brown. | | | | | | |
| 1 | | Drive | Bulk | 5/8/9 | 1 | 129 | |
| 3 | | | Bulk | | | | R, PA |
| 4 | | Drive | Bulk | 9/11/16 | 1 | 128 | |
| 7 | | Drive | | 15/26/44 | 2 | 128 | |
| 7.5 | - @7.5': 6" thick layer of fine sand, gravel up to 1" maximum dimension, very dense, dark brown | | | | | | |
| 11 | | Drive | | 19/40/46 | 2 | 120 | |
| 16 | **SAND (SP):** fine to medium-grained, trace silt, dense, moist, grayish brown. | Drive | | 13/20/27 | | | PA |
| 20 | - @20.0': very dense | Drive (X) | | 8/13/50 | 2 | | |
| 25 | **SILTY SAND (SM):** fine to medium-grained, dense, moist, grayish brown. | Drive | | 10/18/38 | 11 | 105 | |
| 30 | | Drive (X) | | 9/11/13 | 2 | | |

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10598 Orchard Street
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For: All-ERA Properties, LLC

Project No. **21-81-176-01**

Drawing No. **A-5a**

# Log of Boring No. BH-04

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1050 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| | | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SILTY SAND/SANDY SILT (SM/ML):** fine to coarse-grained, dense/very stiff, moist, grayish brown. | | | 10/16/30 | | | PA |
| | | **OLD ALLUVIAL FAN DEPOSITS**<br>**SAND/SILTY SAND (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, moderately desiccated, very dense, moist, dark reddish brown to grayish brown. | | | | | | |
| 40 | | | | | 26/39/45 | 2 | | |
| 45 | | - @45.0': increased gravel and some cobbles | | | 50-6" | 2 | 99 | |
| 50 | | | | | 25/50-6" | 2 | | |
| | | End of boring at 51.0 feet bgs.<br>No groundwater encountered.<br>Borehole backfilled with soil cuttings and compacted by pushing down with the auger using the weight of the drill rig on 08/02/2021. | | | | | | |

# Log of Boring No. BH-05

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1058 Depth to Water (ft, bgs): 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 |
|---|---|---|---|---|---|---|---|
| 0 | **YOUNG ALLUVIAL FAN DEPOSITS** **SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, little gravel up to 2" maximum dimension, trace oxidation staining, dense, moist, light reddish brown. | | X (0–5) | | | | |
| 2.5 | | X | | 11/17/26 | 3 | 115 | EI, CR, CP, DS |
| 5 | - @5.0': medium dense | X | | 10/10/14 | 8 | 109 | |
| 7.5 | **SILTY SAND/SANDY SILT (SM/ML):** fine-grained, moderately desiccated, trace oxidation staining, medium dense/very stiff, moist, yellowish brown. | X | | 8/14/15 | 9 | 122 | |
| 10 | | X | | 11/14/16 | 9 | 123 | |
| 15 | **SAND/SILTY SAND (SP/SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, dry, brown to grayish brown. | X | | 15/25/38 | 1 | 108 | |
| 20 | **SANDY SILT (ML):** fine-grained sand, trace oxidation, stiff, moist, greenish brown. | X | | 6/9/13 | 11 | | |
| 25 | **SAND (SP):** fine to medium-grained, very dense, dry, light brown to grayish brown. | X | | 18/34/41 | 2 | 121 | |
| 30 | | X | | 13/20/26 | 2 | | |

End of boring at 31.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and compacted by pushing down with the auger using the weight of the drill rig on 08/02/2021.

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10598 Orchard Street
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Project No. 21-81-176-01

Drawing No. A-6

Project ID: 21-81-176-01.GPJ; Template: LOG

# Log of Boring No. BH-06

Dates Drilled: 8/2/2021 Logged by: Catherine Nelson Checked By: Robert Gregorek II

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

Ground Surface Elevation (ft): 1059 Depth to Water (ft, bgs): NOT ENCOUNTERED

| Depth (ft) | Graphic Log | SUMMARY OF SUBSURFACE CONDITIONS<br>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 |
|---|---|---|---|---|---|---|---|---|
| 0 | | **YOUNG ALLUVIAL FAN DEPOSITS**<br>**SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, little gravel up to 3" maximum dimension, trace oxidation, medium dense, dry, light grayish brown to brown. | | | | | | |
| | | | X | | 9/14/15 | 1 | 105 | |
| 5 | | | X | X | 12/19/16 | 1 | 121 | |
| | | | | | | | | CP |
| 9 | | **SANDY SILT (ML):** fine-grained sand, oxidation staining, slightly to moderately desiccated, stiff, moist, reddish brown. | | | | | | |
| 10 | | | X | | 5/9/13 | 9 | 113 | |
| 15 | | **SILTY SAND (SM):** fine-grained, slightly desiccated, oxidation staining, medium dense, dry, yellowish gray. | X | | 6/11/12 | 2 | 103 | |

End of boring at 16.5 feet bgs.
No groundwater encountered.
Borehole backfilled with soil cuttings and compacted by pushing down with the auger using the weight of the drill rig on 08/02/2021.

# Appendix B

## Laboratory Testing Program

# APPENDIX B

# LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their 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

Two representative bulk samples were tested to evaluate the expansion potential of materials encountered at the site in accordance with ASTM D4829 Standard. The test results are presented in the following table.

**Table No. B-1, Expansion Index Test Results**

| Boring No. | Depth (feet) | Soil Description | Expansion Index | Expansion Potential |
|---|---|---|---|---|
| BH-01 | 0-6 | Sand/Silty Sand with Gravel (SP/SM) | 0 | Very Low |
| BH-05 | 0-5 | Silty Sand with Gravel (SM) | 0 | Very Low |

## R-value

Two representative bulk soil samples were tested for resistance value (R-value) in accordance with California Test Method CT301. This test provides a relative measure of soil strength for use in pavement design. The test results are presented in the following table.

**Table No. B-2, R-Value Test Results**

| Boring No. | Depth (feet) | Soil Classification | Measured R-value |
|---|---|---|---|
| BH-01 | 0-6 | Sand/Silty Sand with Gravel (SP/SM) | 77 |
| BH-04 | 1-4 | Silty Sand with Gravel (SM) | 67 |

## 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-05 | 0-5 | 7.5 | 21 | 19 | 12,753 |

## Grain-Size Analyses

To assist in classification of soils, mechanical grain-size analyses were performed on three select samples in accordance with the ASTM Standard D6913 test method. Grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results*.

**Table No. B-4, Grain Size Distribution Test Results**

| Boring No. | Depth (ft) | Soil Classification | % Gravel | % Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|
| BH-04 | 1.0-4.0 | Silty Sand with Gravel (SM) | 29.0 | 53.9 | 17.1 | |
| BH-04 | 16.0-17.5 | Sand (SP) | 0.0 | 96.1 | 3.9 | |
| BH-04 | 35.0-36.5 | Silty Sand/Sandy Silt (SM/ML) | 0.0 | 48.9 | 51.1 | |

## 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-2, *Moisture-Density Relationship Results,* and is summarized in the following table.

**Table No B-5, Summary of Moisture-Density Relationship Results**

| Boring No. | Depth (feet) | Soil Description | Optimum Moisture (%) | Maximum Density (lb/cft) |
|---|---|---|---|---|
| BH-05 | 0-5 | Silty Sand, with Gravel (SM), Light Reddish Brown | 7.0 | 127.0 |
| BH-06 | 5-9 | Silty Sand with Gravel (SM), Light Grayish Brown | 5.5 | 132.0 |

## Direct Shear

One direct shear test was performed on samples remolded to 90% of the maximum dry density under soaked moisture conditions in accordance with ASTM D3080. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 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-3*, Direct Shear Test Results,* and the following table.

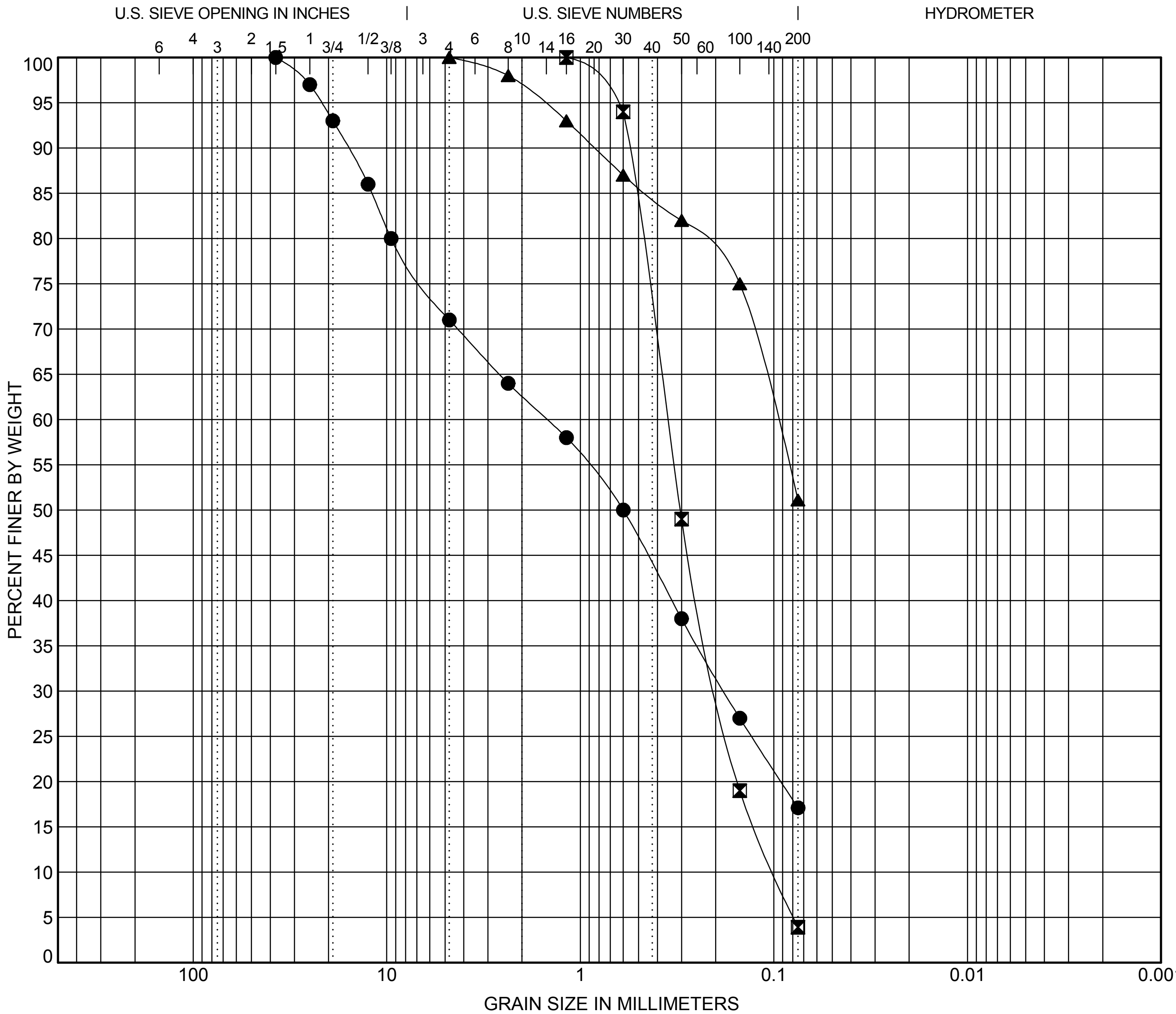
**Table No. B-6, Summary of Direct Shear Test Results**

| Boring No. | Depth (feet) | Soil Description | Peak Strength Parameters | |
|---|---|---|---|---|
| | | | Friction Angle (degrees) | Cohesion (psf) |
| BH-05* | 0-5 | Silty Sand, with Gravel (SM) | 32 | 70 |

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

## 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.




| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Boring No. | Depth (ft) | Description | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | **BH-04** | **1-4** | **SILTY SAND WITH GRAVEL (SM)** | | | | | |
| ⊠ | **BH-04** | **16.0-17.5** | **SAND (SP)** | | | | **1.06** | **3.58** |
| ▲ | **BH-04** | **35.0-36.5** | **SILTY SAND/SANDY SILT (SM/ML)** | | | | | |
| | | | | | | | | |
| | | | | | | | | |

| | Boring No. | Depth (ft) | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **BH-04** | **1-4** | **37.5** | **1.487** | **0.181** | | **29.0** | **53.9** | **17.1** | |
| ⊠ | **BH-04** | **16.0-17.5** | **1.18** | **0.355** | **0.193** | **0.099** | **0.0** | **96.1** | **3.9** | |
| ▲ | **BH-04** | **35.0-36.5** | **4.75** | **0.097** | | | **0.0** | **48.9** | **51.1** | |
| | | | | | | | | | | |
| | | | | | | | | | | |

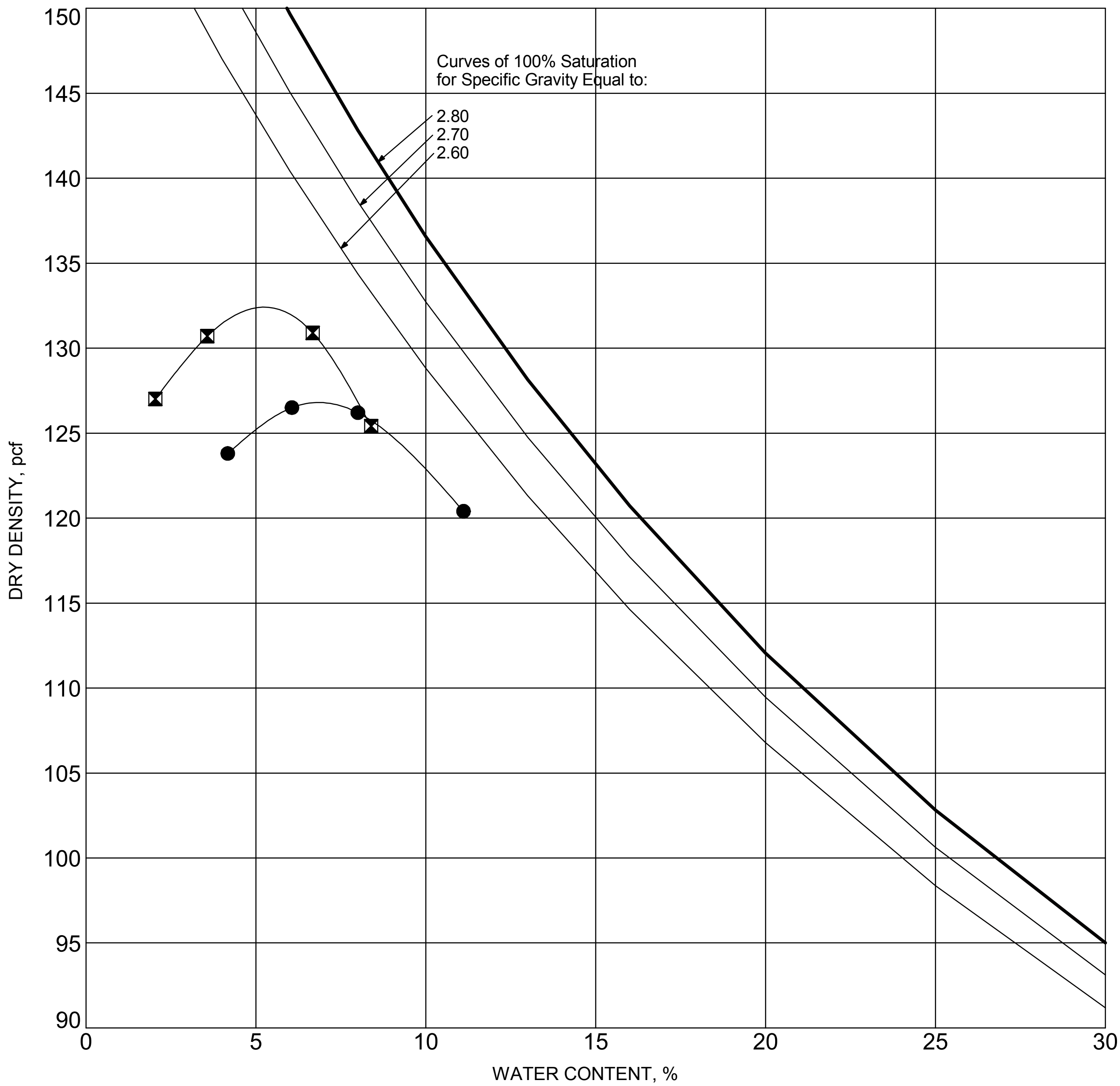
# GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Linden Bloomington Condos, Tentative Tract 20481
10598 Orchard Street
Bloomington Area, San Bernardino County, California
For: All-ERA Properties, LLC

Project No. **21-81-176-01**

Drawing No. **B-1**




| SYMBOL | BORING NO. | DEPTH (ft) | DESCRIPTION | ASTM TEST METHOD | OPTIMUM WATER, % | MAXIMUM DRY DENSITY, pcf |
|---|---|---|---|---|---|---|
| ● | **BH-05** | **0-5** | **SILTY SAND WITH GRAVEL (SM), Light Reddish Brown** | **D1557 - B** | **7.0** | **127.0** |
| ⧗ | **BH-06** | **5-9** | **SILTY SAND (SM), Light Grayish Brown** | **D1557 - B** | **5.5** | **132.0** |
| | | | | | | |
| | | | | | | |
| | | | | | | |

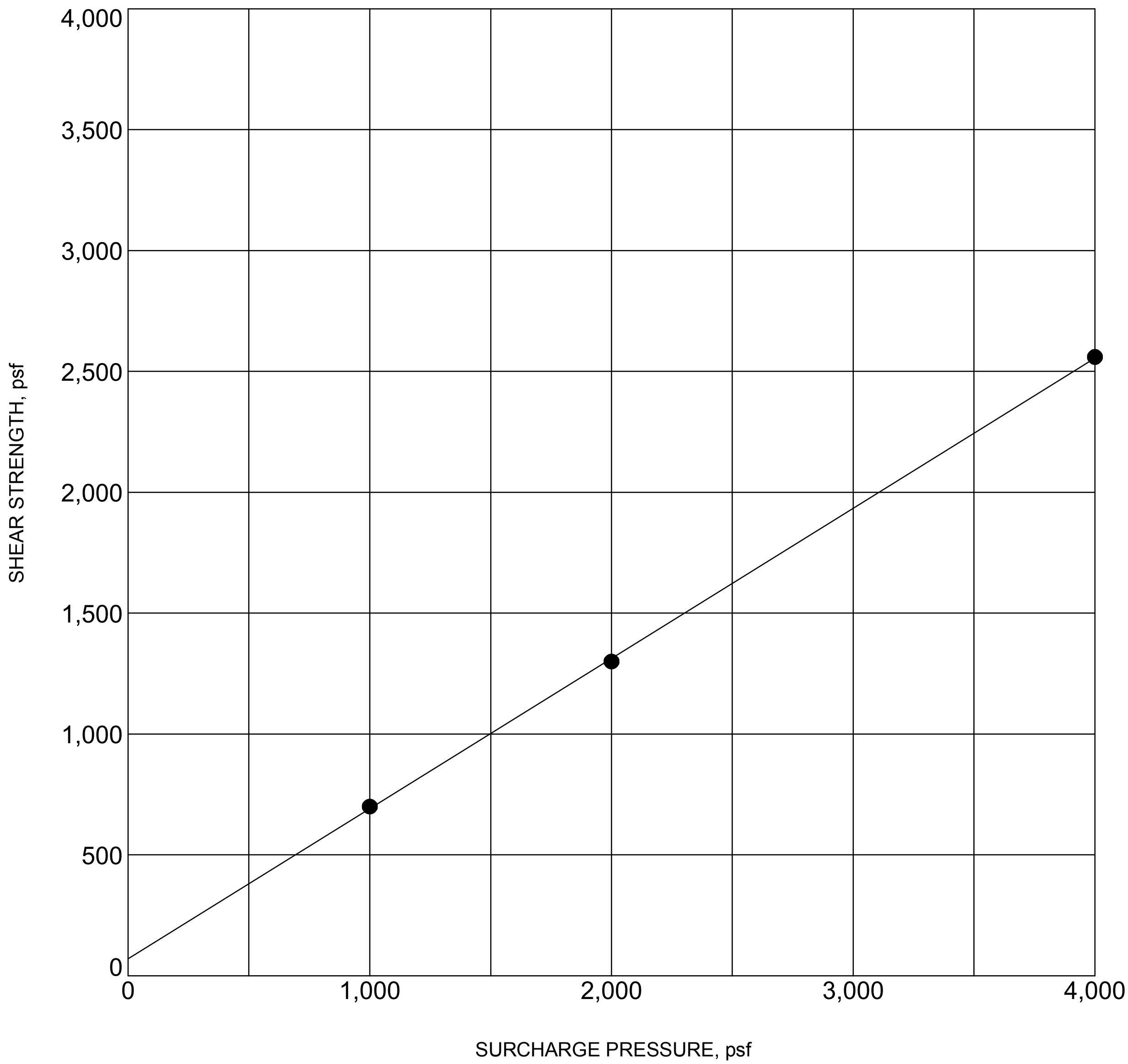
# MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Linden Bloomington Condos, Tentative Tract 20481
10598 Orchard Street
Bloomington Area, San Bernardino County, California
For: All-ERA Properties, LLC

Project No.
**21-81-176-01**

Drawing No.
**B-2**




| | | | | | |
|---|---|---|---|---|---|
| BORING NO. | : | **BH-05** | DEPTH (ft) | : | **0-5** |
| DESCRIPTION | : | **SILTY SAND WITH GRAVEL (SM)** | | | |
| COHESION (psf) | : | **70** | FRICTION ANGLE (degrees): | | **32** |
| MOISTURE CONTENT (%) | : | **7.0** | DRY DENSITY (pcf) | : | **114.5** |

NOTE: Ultimate Strength.

# DIRECT SHEAR TEST RESULTS

Converse Consultants

Linden Bloomington Condos, Tentative Tract 20481
10598 Orchard Street
Bloomington Area, San Bernardino County, California
For: All-ERA Properties, LLC

Project No. **21-81-176-01**

Drawing No. **B-3**

# Appendix C

Percolation Testing

# APPENDIX C

## PERCOLATION TESTING

Percolation testing was performed at three locations (PT-01 through PT-03) on August 03, 2021. The testing was in general accordance with the San Bernardino County Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013). 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 in all three borings, meeting the criteria for testing as “sandy soil”. Percolation testing was conducted within 26 hours of presoaking. During testing, the water level and total depth of the test hole were measured from the top of the pipe to a pre-determined height. During testing, the water level and total depth of the test holes were measured from the top of the pipe every *10* minutes for at least 1 hour. Following the completion of percolation testing, the pipe was removed, and the percolation test holes were backfilled with excavated soil and tamped.

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 San Bernardino County guidelines. A factor of safety of 2 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 Plate 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/hr) (FOS 2) |
|---|---|---|---|
| PT-01 | 15.1 | Sand/Silty Sand, with Gravel (SP/SM) | 11.62 |
| PT-02 | 13.1 | Sand/Silty Sand, with Gravel (SP/SM) | 11.53 |
| PT-03 | 13.9 | Sand/Silty Sand, with Gravel (SP/SM) | 11.57 |

Based on the calculated infiltration rate during the final respective intervals in each test, an average infiltration rate of 11.57 inches per hour can be utilized for design.

**Estimated Infiltration Rate from Percolation Test Data, PT-01**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-01 |
| Test Location | Roadway, Adj. Lot 53 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 181.2 |
| Inside Diameter of Pipe, I (inches) | 2.93 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| Factor of Safety (FOS), F | 2 |

| Interval No. | Time Interval, $\Delta t$ (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, $\Delta H$ (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 | 60 | 181.20 | 25.00 | 121.20 | 0.00 | 121.20 | 60.60 | 9.29 | | 4.65 |
| 2 | 25.00 | 60 | 181.20 | 50.00 | 121.20 | 0.00 | 121.20 | 60.60 | 9.29 | | 4.65 |
| 3 | 10.00 | 60 | 181.20 | 60.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 4 | 10.00 | 60 | 181.20 | 70.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 5 | 10.00 | 60 | 181.20 | 80.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 6 | 10.00 | 60 | 181.20 | 90.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 7 | 10.00 | 60 | 181.20 | 100.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 8 | 10.00 | 60 | 181.20 | 110.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 9 | 10.00 | 60 | 181.20 | 120.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| 10 | 10.00 | 60 | 181.20 | 130.00 | 121.20 | 0.00 | 121.20 | 60.60 | 23.23 | | 11.62 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **11.62** |

San Bernardino County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

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

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

**Plate No.**
1

**Infiltration Rate versus Time, PT-01**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-01 |
| Test Location | Roadway, Adj. Lot 53 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

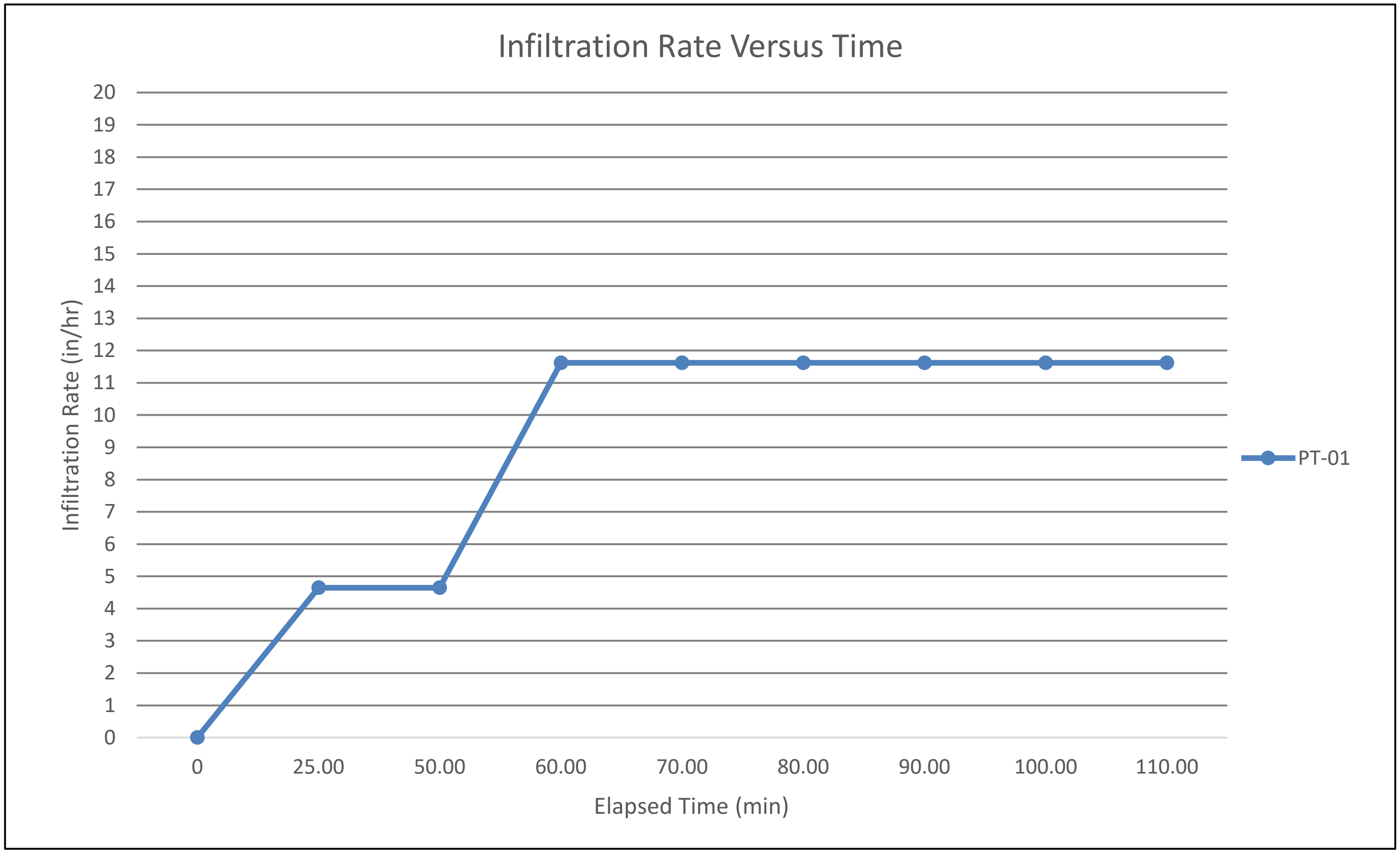



**Plate No.**
2

**Estimated Infiltration Rate from Percolation Test Data, PT-02**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-02 |
| Test Location | Roadway, Adj. Lot 49 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 157.2 |
| Inside Diameter of Pipe, I (inches) | 2.93 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 2 |

| Interval No. | Time Interval, $\Delta t$ (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, $\Delta H$ (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 | 60 | 157.20 | 25.00 | 97.20 | 0.00 | 97.20 | 48.60 | 9.22 | | 4.61 |
| 2 | 25.00 | 60 | 157.20 | 50.00 | 97.20 | 0.00 | 97.20 | 48.60 | 9.22 | | 4.61 |
| 3 | 10.00 | 60 | 157.20 | 60.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 4 | 10.00 | 60 | 157.20 | 70.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 5 | 10.00 | 60 | 157.20 | 80.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 6 | 10.00 | 60 | 157.20 | 90.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 7 | 10.00 | 60 | 157.20 | 100.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 8 | 10.00 | 60 | 157.20 | 110.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 9 | 10.00 | 60 | 157.20 | 120.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| 10 | 10.00 | 60 | 157.20 | 130.00 | 97.20 | 0.00 | 97.20 | 48.60 | 23.05 | | 11.53 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **11.53** |

San Bernardino County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

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

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

**Plate No.**
1

**Infiltration Rate versus Time, PT-02**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-02 |
| Test Location | Roadway, Adj. Lot 49 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

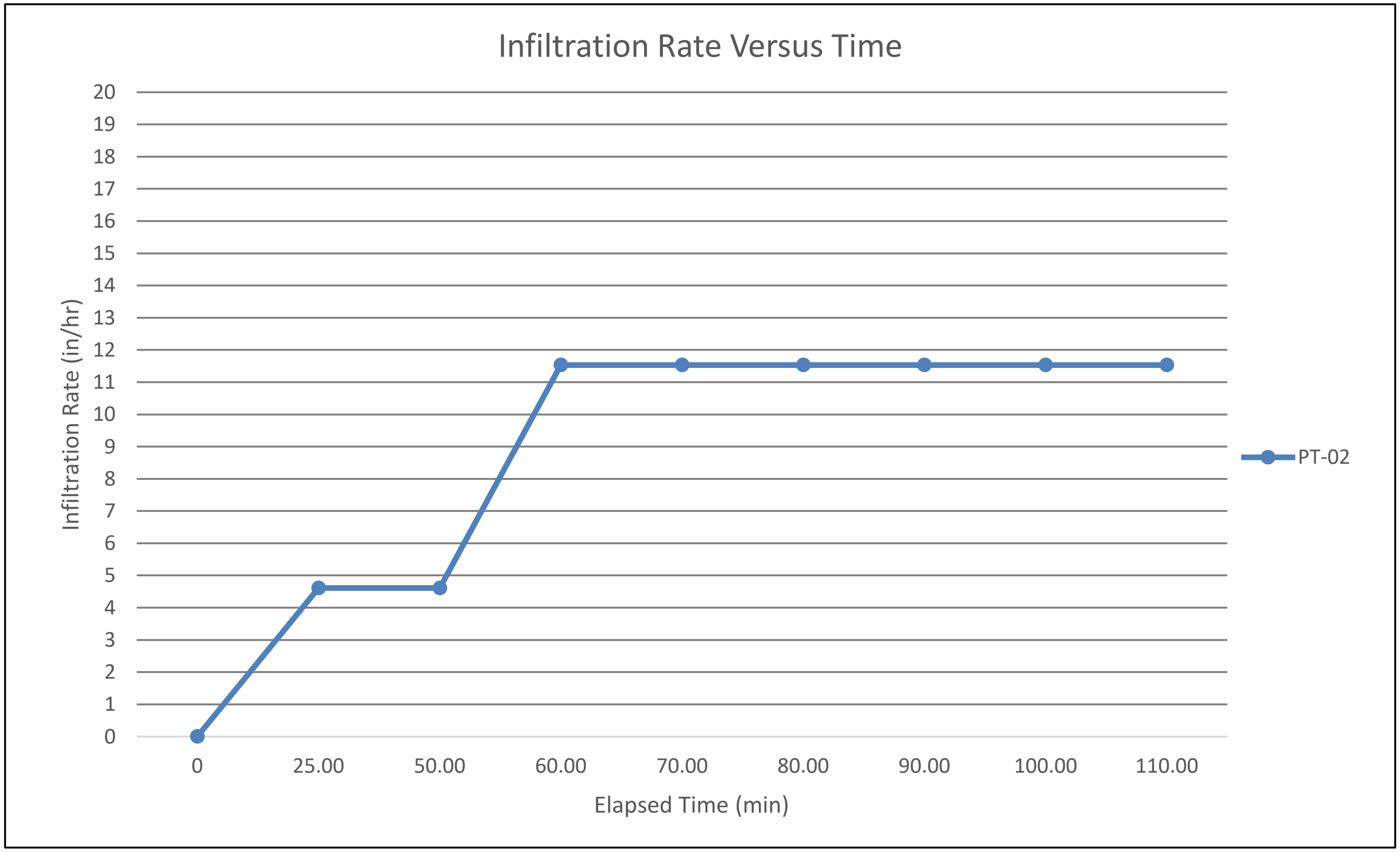



**Plate No.**
2

**Estimated Infiltration Rate from Percolation Test Data, PT-03**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-02 |
| Test Location | Roadway, Adj. Lot 59 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

Shaded cells contain calculated values.

| | |
|---|---|
| Test Hole Radius, r (inches) | 4 |
| Total Depth of Test hole, $D_T$ (inches) | 166.8 |
| Inside Diameter of Pipe, I (inches) | 2.93 |
| Outside Diameter of Pipe, O (inches) | 3.13 |
| | |
| | |
| Factor of Safety (FOS), F | 2 |

| Interval No. | Time Interval, $\Delta t$ (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, $\Delta H$ (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 | 60 | 166.80 | 25.00 | 106.80 | 0.00 | 106.80 | 53.40 | 9.25 | | 4.63 |
| 2 | 25.00 | 60 | 166.80 | 50.00 | 106.80 | 0.00 | 106.80 | 53.40 | 9.25 | | 4.63 |
| 3 | 10.00 | 60 | 166.80 | 60.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 4 | 10.00 | 60 | 166.80 | 70.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 5 | 10.00 | 60 | 166.80 | 80.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 6 | 10.00 | 60 | 166.80 | 90.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 7 | 10.00 | 60 | 166.80 | 100.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 8 | 10.00 | 60 | 166.80 | 110.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 9 | 10.00 | 60 | 166.80 | 120.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| 10 | 10.00 | 60 | 166.80 | 130.00 | 106.80 | 0.00 | 106.80 | 53.40 | 23.13 | | 11.57 |
| | | | | | | | | | | | |
| | | | | | | | | | | | |

| | |
|---|---|
| **Recommended Design Infiltration Rate (inches/hr)** | **11.57** |

San Bernardino County Technical Guidance Document for Water Quality Management Plans, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations (San Bernardino County, 2013)

$H_0 = D_T - D_0$

$H_f = D_T - D_f$

$\Delta H = H_0 - H_f$

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

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

**Plate No.**
1

**Infiltration Rate versus Time, PT-03**

| | |
|---|---|
| Project Name | Linden Bloomington Condos, Tentative Tract 20481 |
| Project Number | 21-81-176-01 |
| Test Number | PT-02 |
| Test Location | Roadway, Adj. Lot 59 |
| Personnel | Joseph Hyunh |
| Presoak Date | 8/2/2021 |
| Test Date | 8/3/2021 |

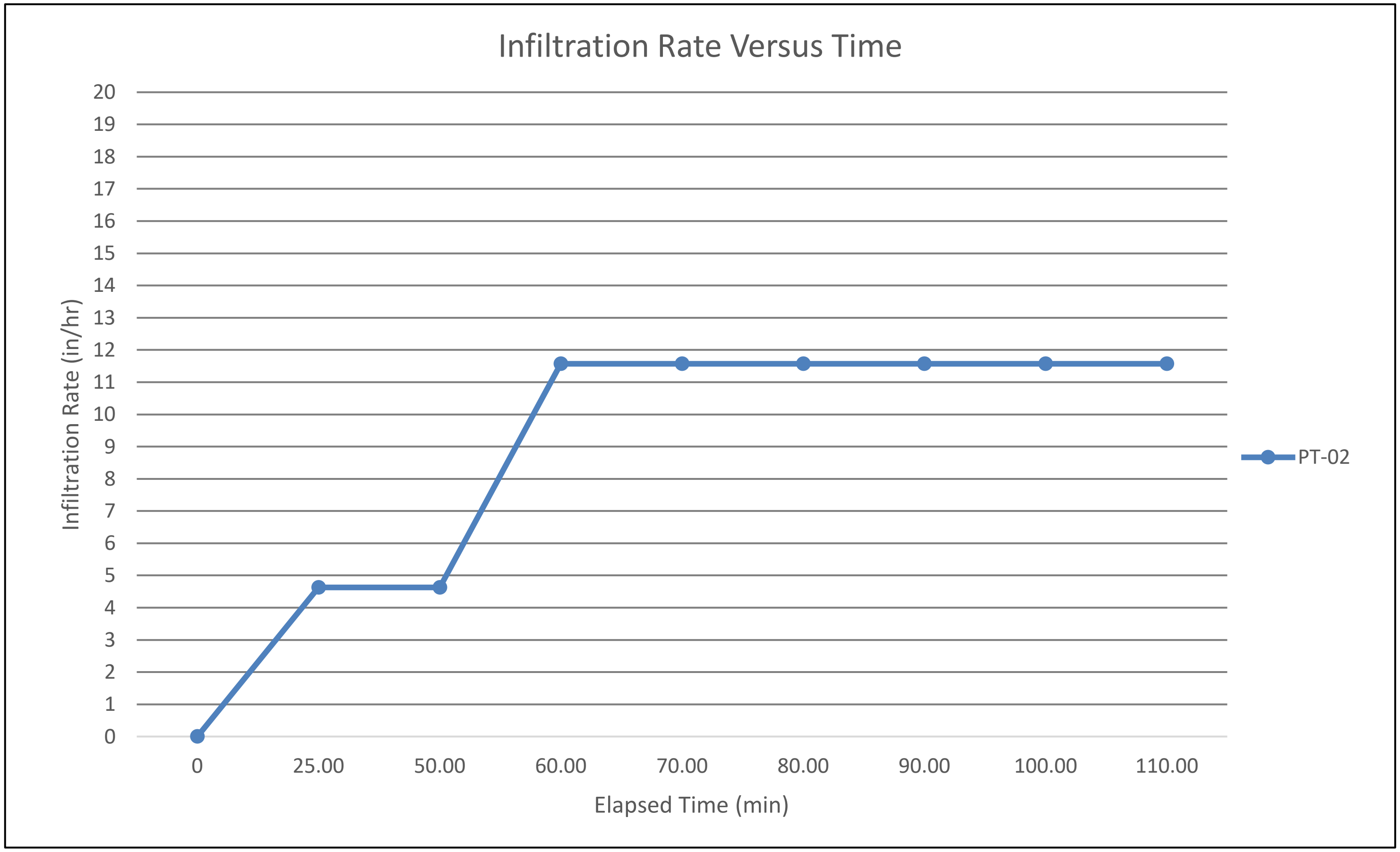



**Plate No.**
2

# Appendix D

## Earthwork Specifications

# APPENDIX D

# EARTHWORK SPECIFICATIONS

## D1.1 Scope of Work

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

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

## D1.2 Site Inspection

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

2. This Preliminary Geotechnical Investigation and Water Infiltration Testing Report by Converse Consultants, dated December 20, 2021, may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information should not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature

of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

## D1.3 Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.
2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.
3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

## D1.4 Site Clearing

1. Clearing and grubbing should consist of the removal from areas to be graded: all existing pavement, utilities, and vegetation.
2. Organic and inorganic materials resulting from the clearing and grubbing operations should be hauled away from the areas to be graded.

## D1.5 Excavations

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

## D1.6 Preparation of Fill Areas

1. All organic material, organic soils and debris should be removed from the proposed development areas.
2. After the required removals have been made, the exposed earth materials should be excavated to provide a zone of structural fill for the support of footings, slabs-on-grade, and exterior flatwork or other proposed improvements. All loose, soft or disturbed earth materials should be removed from the bottom of excavations before placing structural fill. All structures will require a minimum of 2.0 feet of compacted fill beneath building footings and 2.0 feet below any proposed wall footings .
3. The subgrade in all areas to receive fill should be scarified to a minimum depth of 6 inches. Scarification may be terminated on moderately hard to hard, cemented

earth materials with the approval of the Geotechnical Engineer. The soil moisture should be adjusted to at least 0 to 2 percent above optimum for fine-grained soils and within 3 percent of optimum moisture content for granular soils, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.

4. Compacted fill may be placed on native soils that have been properly scarified and recompacted as discussed above.
5. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

## D1.7 Placement and Compaction of Fill

1. Compacted fill placed for the construction of the embankment or for any planned structures will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.
2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer should be placed in layers on approved earth materials. Soils used as compacted structural fill should have the following characteristics:
   a. All fill soil particles should not exceed 8 inches in nominal size and should be free of organic matter and miscellaneous inorganic debris and inert rubble.
   b. Imported fill materials should have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) at about 0 to 2 percent above optimum moisture for fine-grained soils, and within 3 percent of optimum for granular soils.
3. Fill exceeding 5 feet in height should not be placed on native slopes that are steeper than 5:1 horizontal:vertical (H:V). Where native slopes are steeper than 5:1 H:V, and the height of the fill is greater than 5 feet, the fill should be benched into competent materials. The height and width of the benches should be at least 2 feet.
4. Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.
5. 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 Engineer approves the moisture and density conditions of the previously placed fill.

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

## D1.8 Fill Slope Construction

1. Fill slopes placed above existing surfaces or cut slopes should be constructed with keyways.
2. Where fill is placed against existing slopes steeper than 5:1 H:V, the new fill slopes should be keyed and benched to provide increased lateral support after removal of the unsuitable surficial soils, when present.

Keyways and benches should be constructed as indicated in Section 10.3 of this report.

## D1.9 Observation and Testing

1. During the progress of grading and trench backfill, the Geotechnical Engineer will provide observation of the fill placement operations.
2. Field density tests of all compacted fill will be made during grading and trench backfill to provide an opinion on the degree of compaction being obtained by the Contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content should be made as necessary, until the required degree of compaction is obtained.
3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.