APPENDIX E

GEOTECHNICAL REPORT

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

GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT

RIO HONDO PARK IMPROVEMENTS
8421 SAN LUIS POTOSI
CITY OF PICO RIVERA, LOS ANGELES COUNTY, CALIFORNIA

CONVERSE PROJECT NO. 22-31-304-01



Prepared For:

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Presented By:

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June 24, 2024

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

June 24, 2024

Mr. Doug Grove, RLA, ASLA, LEED AP
President
RHA Landscape Architects-Planners
6800 Indiana Avenue, Suite 245
Riverside, California 92506

Subject: **GEOTECHNICAL INVESTIGATION AND WATER PERCOLATION TEST REPORT**
Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
Converse Project No. 22-31-304-01

Dear Mr. Grove:

Converse Consultants (Converse) is pleased to submit this geotechnical investigation and percolation test report to assist with design and construction for the Rio Hondo Park Improvements project located at 8421 San Luis Potosi in the City of Pico Rivera, Los Angeles County, California. The report was prepared in accordance with our proposal dated October 25, 2022, and your Acceptance of Agreement and Authorization to Proceed and dated March 27, 2024.

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

We appreciate this opportunity to be of service to RHA Landscape Architects-Planners, Inc., and the City of Pico Rivera. If you should have any questions regarding this report, please do not hesitate to contact us at 909-474-2847.

CONVERSE CONSULTANTS

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

Dist.: 1/Addressee (e-mail)
AZ/SM/HSQ/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the following professionals whose seals and signatures appear hereon.

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

Aleksey Zhukov
Staff Engineer

Stephen McPherson, GIT
Staff Geologist



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

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation water percolation tests performed for the Rio Hondo Park Improvements project located at 8421 San Luis Potosi in the City of Pico Rivera, Los Angeles County, California. The approximate location of the site is shown on Figure No. 1, *Approximate Site Location Map*.

The purposes of this investigation were to determine the nature and engineering properties of the subsurface soils and to provide recommendations for site earthwork, and design and construction of the proposed improvements.

This report is prepared for the project described herein and is intended for use solely by RHA Landscape Architects-Planners, Inc., the City of Pico Rivera, and their authorized agents for design purposes. 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.

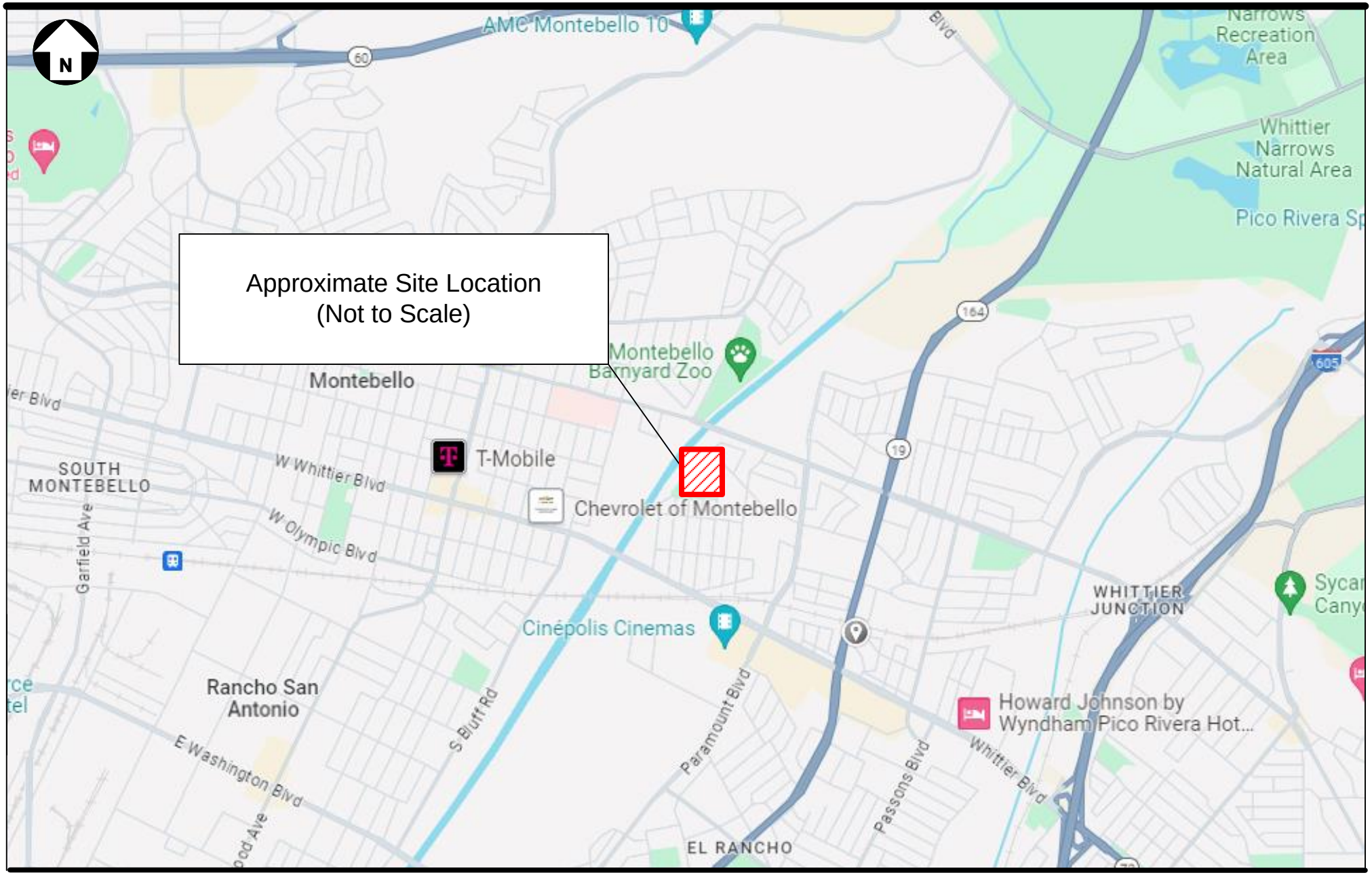
2.0 PROJECT DESCRIPTION

The following improvements are planned for the Rio Hondo Park Improvements project.

- Restroom with remote looking mechanism
- Exercise station
- Plaza and large shade sail for programmed recreation
- Game table and large shade sails
- Concrete cornhole
- Concrete ping pong tables
- Picnic Shelter
- D.G. walking path and information signage
- Rubberized walking patch with mileage sign
- Trash enclosure
- Bird and butterfly garden
- Parking
- Entry courtyard
- Bench
- Covered bleachers
- Synthetic soccer field with fencing

3.0 SITE DESCRIPTION

The existing Rio Honda Park is located at 8421 , City of Pico Rivera, Los Angeles County, California. The site is approximately 13-acres in size and has an irregular four-sided perimeter. The park is bounded to the northeast by Beverly Road, to the east by Fir Street,



Project: Rio Hondo Park Improvements

Location: 8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California

For: RHA Landscape Architects-Planners

Approximate Site Location Map

Project No.
22-31-304-01

Converse Consultants

Figure No.
1

to the south by San Luis Potosi, and to the northwest by the Rio Hondo Channel. The site is relatively flat and has approximate elevation that varies between 172 and 176 feet above mean sea level (AMSL). The coordinates for the project site are approximately 34.0115° north latitude and 118.0924 ° west longitude.

Currently, the site has existing park amenities, landscape, trees, and lighting. Photograph Nos. 1 through 3 depict the current site conditions.



Photograph No. 1: Current site conditions at BH-01 facing west.



Photograph No. 2: Current site conditions at the proposed bleachers facing northeast.



Photograph No. 3: Current conditions near the proposed mini pitch court Street facing southeast.

4.0 SCOPE OF WORK

The scope of this investigation included the following tasks.

4.1 Project Set-Up

As part of the project set-up, our staff performed the following tasks:

- Prepared the boring and percolation test locations map and submitted it for your review and approval.
- Staked and marked the boring locations in the field 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 conduct exploratory borings.

4.2 Subsurface Exploration

Three exploratory soil borings (BH-01 through BH-03) were advanced within the footprint of the proposed improvements on April 22, 2024. The planned depths of the borings were between 10.0 and 50.0 feet below ground surface (bgs). The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. The borings were visually logged by a Converse engineer and sampled at

regular intervals and at changes in subsurface soils. Borings details are presented in Table No. A-1, *Summary of Borings* and Drawings Nos. A-2 and A-7, *Logs of Borings.*

Three percolation test holes (PT-01 through PT-03) were drilled on April 22, 2024, and tested on April 24, 2024, within the project site to perform percolation testing. The test holes were drilled to depths between 4.5 and 5.0 feet bgs, the location PT-02 was drilled using a 5-inch diameter handheld auger. Details about the percolation tests are presented in Appendix D, *Percolation Testing.*

The approximate locations of the borings and percolation test holes is shown on Figure No. 2, *Approximate Borings and Percolation Tests Locations Map.* A detailed discussion of subsurface exploration is presented in Appendix A, *Field Exploration*.

4.3 Laboratory Testing

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

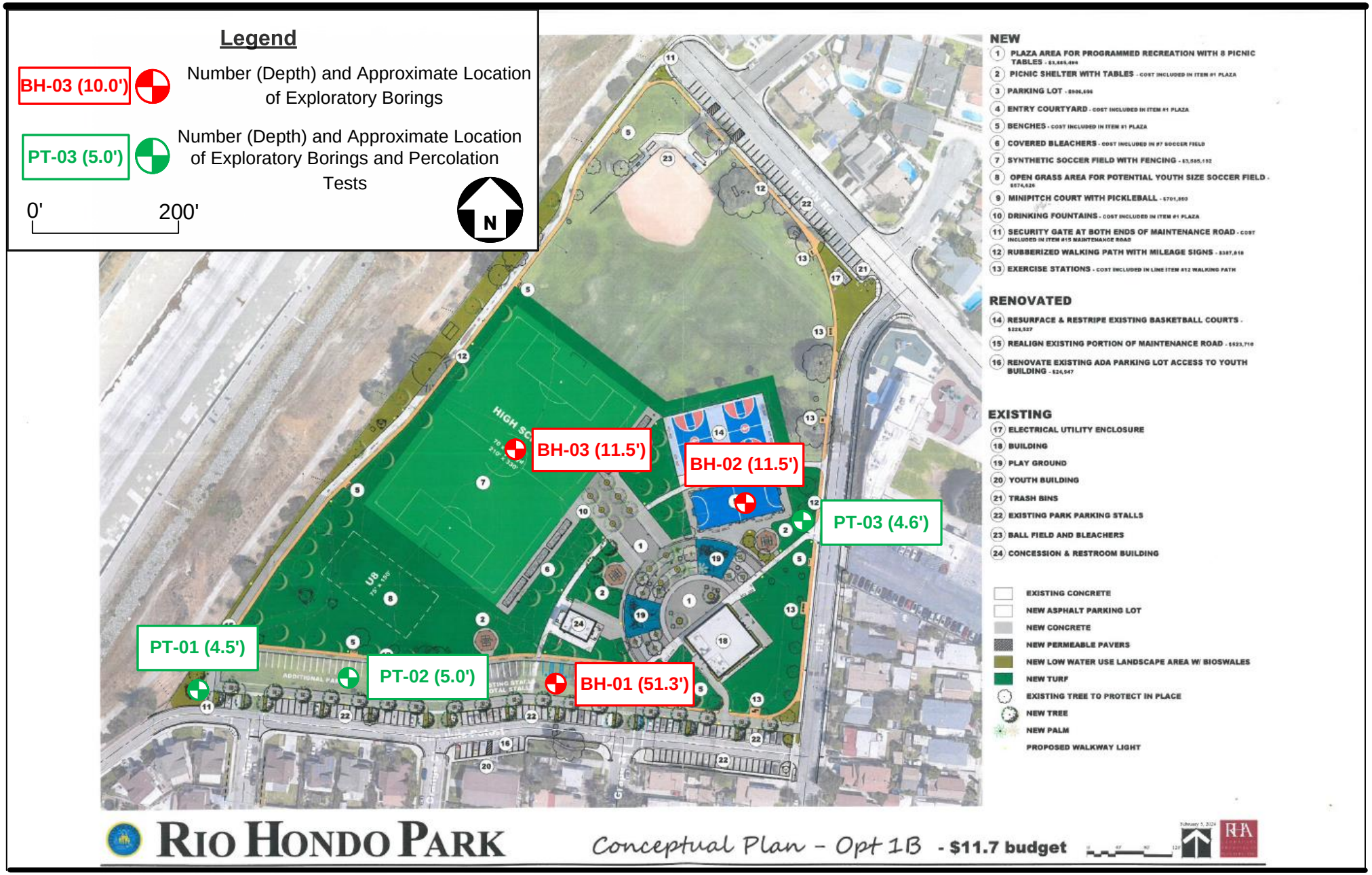
- *In-situ* moisture contents and dry density (ASTM D2216 and ASTM D2937)
- Expansion index (ASTM D4829)
- R-value (ASTM D2844)
- Soil corrosivity (California Tests 422, 417, and 643)
- Collapse potential (ASTM D4546)
- Percent Finer Than Sieve No. 200 (ASTM D1140)
- Grain size distribution (ASTM D6913)
- 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*.

Environmental Analytical Testing for Soil Sample and Waste Disposal

Soil samples from excess soil cuttings were collected during the field work and submitted to a California ELAP certified laboratory (Enthalpy, Orange) under chain-of-custody control. The samples were analyzed as follows.

- EPA 8015M – TPH carbon chain speciation
- EPA 8260 – VOCs and Oxygenates
- EPA 8270 – SVOCs
- EPA 8082 – PCBs
- EPA 8081A – OCPs
- EPA 6010B – Title 22 metals (TTLC)



Project: Rio Hondo Park Improvements

Location: 8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California

For: RHA Landscape Architects-Planners

Approximate Borings and Percolation Tests Locations Map

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Figure No.
2

Drums of excess soil cuttings are to be disposed by American Integrated Services, Inc.

4.4 Report Preparation

Data and information obtained from the document review, field exploration, and laboratory testing program were compiled and evaluated. Geotechnical analyses of the compiled data were performed, and this report was prepared to present our findings, conclusions, and recommendations for the proposed improvements.

5.0 SITE CONDITIONS

The subsurface conditions encountered at the site during our investigation are described in the following sections.

5.1 Subsurface Profile

Fill: Fill soils are likely associated with previous grading for the existing park structures and amenities. The fill soils were likely derived from on-site sources and are similar to the native alluvial soils in composition and density. Based on the exploratory borings and laboratory test results, the subsurface fill soils at the site consisted primarily of a mixture of sand, silt, and gravel. Cobbles up to 4-inch in maximum dimension was observed in PT-01.

Alluvium: Based on the exploratory borings and laboratory test results, the alluvium soils at the site consisted primarily of a mixture of sand, silt, and gravel. Gravel up to 2.5-inch in maximum dimension was observed in borings.

For a detailed description of the subsurface materials encountered in the exploratory borings, see Drawing Nos. A-2 through A-7, *Logs of Borings,* in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was encountered in the boring BH-01 at a depth of 25.0 feet bgs during the field investigation. The generalized coordinates of 34.0109 N, 118.0929 W, were used for the following groundwater comparisons.

For comparison, regional groundwater data from the GeoTracker database (SWRCB, 2024) was reviewed to evaluate the current and historical groundwater levels. Several sites with groundwater data were identified within a 1.0-mile radius of the project site and the results of that search are included below.

- MOBIL #18-EP2 (T0603703130), located approximately 3,400 feet east of the generalized coordinates, reported groundwater at depths between 27.5 and 49.4 feet bgs between 2001 and 2006.
- ERNIE HELM'S ARCO (T0603793062), located approximately 3,600 feet southeast of the generalized coordinates, reported groundwater at depths between 22.9 and 56.0 feet bgs between 2001 and 2008.
- 76 STATION #4867 (T10000001248), located approximately 4,300 feet southeast of the generalized coordinates, reported groundwater at depths between 41.2 and 50.3 feet bgs between 2011 and 2013.
- WINTERS INDUSTRIAL CLEANING (T0603776756), located approximately 4,000 feet south of the generalized coordinates, reported groundwater at depths between 22.2 and 65.2 feet bgs between 2001 and 2008.
- SHELL STATION (FORMER) (T0603717747), located approximately 4,500 feet northwest of the generalized coordinates, reported groundwater at depths between 59.0 and 65.9 feet bgs between 2004 and 2005.

The National Water Information System (USGS, 2024) was reviewed for current and historical 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

Site Number	Location	Groundwater Depth Range (ft. bgs)	Date Range
340108118050001	North of Friendship Avenue; approximately 3,950 feet northeast of generalized coordinates.	65.4	2018
340100118045701	East of South Bluff Road; approximately 3,500 feet northeast of generalized coordinates.	58.0	2018
340020118060101	East of South Bluff Road; approximately 3,400 feet southwest of generalized coordinates.	60.0	1989
340100118045701	East of North Montebello Blvd; approximately 4,300 feet southeast generalized coordinates.	172.2	2017
340047118054601	Between Maiden Lane and Madison Avenue; approximately 700 feet northwest of generalized coordinates.	117.1	2017

The California Department of Water Resources database (DWR, 2024) was reviewed for historical groundwater data from sites within a 1.0-mile radius of the generalized coordinates. No site with groundwater data was found within a 1.0-mile radius of the generalized coordinates.

Based on relevant available data, the historical high groundwater level reported at wells within approximately one mile of the site was deeper than approximately 22.2 feet bgs. Current groundwater is expected to be deeper than about 21.8 feet bgs. Groundwater may be expected to be encountered during the construction of the project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in the site vicinity. Shallow perched groundwater may be present locally, particularly following precipitation.

5.3 Collapse Potential

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

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

The degree of collapse of a soil can be defined by the collapse potential value, which is expressed as a percentage of collapse of the total sample using the Collapse Potential Test (ASTM D4546). According to the ASTM guideline, the severity of collapse potential is commonly evaluated by the following Table No. 2, *Collapse Potential Values*.

Table No. 2, Collapse Potential Values

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

Based on the laboratory test result (collapse potential of 0.4 and 1.3 percent), slight collapse potential is anticipated at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

5.4 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 result, the expansion index (EI) of the upper 5 feet of site soils were 0, corresponding to very low expansion potential.

5.5 Excavatability

The subsurface materials at the site are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment. However, excavation will be difficult if concentration of gravel and cobbles is encountered.

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. The selection of appropriate excavation equipment models should be done by an experienced earthwork contractor.

5.6 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the 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.

6.0 CBC 2022 SEISMIC DESIGN PARAMETERS

Seismic parameters based on the 2022 California Building Code (CBSC, 2022) and ASCE 7-16 are provided in the following table. These parameters were determined using the generalized coordinates for the location and the Seismic Design Maps ATC online tool.

Table No. 3, CBC Seismic Design Parameters

Parameter	Value
Site Coordinates	34.0105 N, 118.0927 W
Risk Category	II
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.886g
Mapped 1-second Spectral Response Acceleration, S_1	0.675g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.886g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.148g
Design Spectral Response Acceleration for short period S_{DS}	1.257g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.765g
Peak Ground Acceleration, PGA_M	0.895g

7.0 LABORATORY TEST RESULTS

Results of the various laboratory tests are presented in Appendix B, *Laboratory Testing Program,* except for the results of *in-situ* moisture and dry density tests which are presented on the Logs of Borings in Appendix A, *Field Exploration.* The results are also discussed below.

7.1 Physical Testing

The results of laboratory tests on samples obtained from the site are presented below.

- *In-situ* Moisture and Dry Density – *In-situ* dry density and moisture content of the alluvial soils were determined in accordance with ASTM Standard D2216 and D2937. The dry densities of the upper 10 feet from the site ranged from 96 to 110 pcf with moisture contents ranging from 2 to 7 percent. Results are presented in the *Logs of Borings* in Appendix A, *Field Exploration.*
- Expansion Index (EI) – One representative soil sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result showed an EI of 0, corresponding to a very low expansion potential.
- R-Value (R) – One representative bulk sample was tested in accordance with ASTM Standard D2844. The result of the R-value test was 36.
- Collapse Potential (CL) – The collapse potential of two relatively undisturbed samples were tested under a vertical stress of up to 2.0 kips per square foot (ksf) in accordance with the ASTM Standard D4546 test method. The test results showed a collapse potential of 0.4 and 1.3, indicating a slight collapse potential.

- Percent Finer than Sieve No. 200 (FC) tests were performed on 4 selected soil samples to aid in the classification of the on-site soils and to estimate other engineering parameters. Testing was performed in general accordance with the ASTM Standard D1140 test method. The test results are presented in the boring logs and in Appendix B *Laboratory Testing Program.*
- Grain Size Analysis (PA) – Two representative soil 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.*
- Maximum Dry Density and Optimum Moisture Content (CP) – The moisture-density relationship of one representative soil sample was tested in according to ASTM Standard D1557 and the result is presented in Drawing No. B-2, *Moisture-Density Relationship Result*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry density was 129.7 pounds per cubic feet (pcf) with optimum moisture content of 9.1 percent. With rock correction, the dry density was 133.1 pounds per cubic feet (pcf) with optimum moisture content of 8.1 percent.
- Direct Shear (DS) – One direct shear test was performed in accordance with ASTM Standard D3080 on relatively undisturbed ring samples. The result of the direct shear test is presented in Drawing No. B-3, *Direct Shear Test Result* in Appendix B, *Laboratory Testing Program*.

7.2 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 these tests was to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by Keegan Lab, LLC. (Pomona, CA) in accordance with California Tests 643, 422, and 417. The test results are presented in Appendix B, *Laboratory Testing Program* and summarized below.

- The pH measurement of the sample was 8.0.
- The soluble sulfate content of the sample was 43 ppm (0.0043 percent by weight).
- The chloride concentration of the sample was 31 ppm.
- The minimum electrical resistivity of the sample when saturated was 4,520 ohm-cm.

8.0 EARTHWORK RECOMMENDATIONS

Earthwork recommendations for the project are presented in the following sections.

8.1 General

This section contains our general recommendations regarding earthwork and grading for the proposed improvements. These recommendations are based on the results of our

field exploration, laboratory tests, 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 findings during the final investigation or observation of the actual field conditions during grading.

All existing underground utilities and appurtenances (if any) should be located at the 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, demolished material, and artificial fill (if any) and surficial soils containing roots and perishable materials should be stripped and removed from the 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.

8.2 Overexcavation/Removal

The footings and slabs-on-grade, and pavements should be uniformly supported by compacted fill. In order to provide uniform support, structural areas should be overexcavated, scarified, and recompacted as follows.

Table No. 4, Overexcavation Depths

Structure/Pavement	Minimum Excavation Depth
Footings (Restrooms, Shade structure)	18 inches below footing bottoms, or 3 feet below ground surface, whichever is deeper
Footings (Perimeter fencing, Security gates)	18 inches below footing bottoms, or 2 feet below ground surface, whichever is deeper
Slabs-on-grade	18 inches below slab bottom
Pavement	12 inches below existing grade

The depth of over excavation below the footings and slabs should be uniform. The over excavation should extend to at least 3 feet beyond the footprint of the footings and slabs and 1 foot beyond the edge of the pavement. The over excavation bottom should be scarified and compacted as described in Section 8.4, *Compacted Fill Placement*.

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. Consideration should be given to using slot cuts or other excavation methods which preserve lateral support during excavation operations near the existing structures.

8.3 Engineered/Structural Fill

No fill or aggregate base should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the site are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including cleaning 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 1 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 30 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-site soils may be suitable as structural/engineered fill materials provided the above criteria are meet with field testing.

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

8.4 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 12 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse grained soils and 0 to 2 percent above optimum moisture content for fine grained soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

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

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. At least the upper 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.

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.

8.5 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 on-site soils free of oversize particles and deleterious matter may be used to backfill the trench zone.

Trench backfill recommendations are presented 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.
- 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.

8.6 Shrinkage and Subsidence

The volume of excavated and recompacted soils may be expected to 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. For preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- An average shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) of 18 percent can be used for upper 6 feet of soils 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. For estimation purposes, ground subsidence may be taken as 0.1 to 0.15 feet.

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

8.7 Site Drainage

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. The building pad should have a gradient of at least 2 percent towards drainage facilities. The drainage gradient should be 1 percent for paved areas and 2 percent for landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

9.0 DESIGN RECOMMENDATIONS

Design recommendations are presented in the following sections.

9.1 General Evaluation

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the above earthwork recommendations will be implemented.

9.2 Shallow Foundation Design Parameters

The proposed improvements may be supported on continuous and/or isolated stiffened spread footings. The design of the shallow foundations should be based on the recommended parameters presented in the following table.

Table No. 5, Recommended Foundation Parameters

Parameter	Value
Minimum continuous footing width	18 inches
Minimum isolated footing width	18 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade	18 inches
Allowable net bearing capacity	2,500 psf

The allowable bearing capacity can be increased by 500 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.

9.3 Lateral Earth Pressures and Resistance to Lateral Loads

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

9.3.1 Active Earth Pressures

The active earth pressure behind any buried wall or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The recommended lateral earth pressures without surcharge for the site are presented in the table below.

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

Loading Conditions	Lateral Earth Pressure (psf/ft. depth)
Active earth conditions (wall is free to deflect at least 0.001 radian)	45
At-rest (wall is restrained)	65

These pressures assume a level ground surface around the structure for a distance greater than the structure height, no surcharge, and no hydrostatic pressure. If water pressure is allowed to build up behind the structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

9.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 between concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 250 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.

9.4 Drilled Pier Foundations

The light posts can be supported on drilled pier foundations deriving their support primarily through skin friction. The piers may be designed for compression using an allowable skin friction value of 110 psf for a minimum of 12 feet deep below the finished grade. This value may be increased by 33 percent for transient wind and seismic forces. For pier design in tension, 50 percent of the recommended allowable skin friction values in compression may be used. The drilled pier should have a minimum diameter of 24 inches. For design purposes, the upper 2 feet of the soil should be neglected in determining the skin friction and point of fixity can be considered in the toe of pier.

The equivalent lateral earth pressure equal to 250 pounds per square foot per foot of depth may be used for the design.

9.5 Drilled Pier Foundation Installation Recommendations

It is the responsibility of the contractor to select proper construction equipment and method to correctly install the piers based on his own interpretation of the information presented in this report.

Groundwater was encountered in the exploratory boring at the depth of 25.0 feet below existing ground surface and due to the presence of sandy soil with gravel, there is some possibility of caving. However, casing, or other methods approved by the project geotechnical consultant, may be used to support the sides of the excavation. Casing should be used at the discretion of the contractor. The casing should be advanced as drilling proceeds by drilling with a flight or bucket auger smaller in diameter than the inside of the casing. Occasional hammering may be required to advance the casing within the excavation. The casing, when used, should not be left in place as the pier designs are based on skin friction only. The casing should be pulled as the concrete is being poured, while always maintaining a head of concrete inside the casing. The contractor should have equipment on-site with sufficient pulling capacity to pull the casing at the proper time. The casing should have an outside diameter not less than the specified diameter of the pier.

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

Drilled pier installation shall be performed under continuous observation by the project geotechnical consultant to confirm that the subsurface soils are similar to the soils encountered during our field investigation, which have formed the basis of our pier design recommendations. The contractor shall provide access and necessary facilities, including droplights, at his expense, to accommodate pier observations.

Drilled pier installation shall be performed such that compliance with all safety rules and requirements is achieved.

9.6 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 8.4 *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 of the supporting materials and should be designed by a structural engineer. Slab should be monolithically constructed with the footings and grade beams.

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 pf sand, to minimize punctures and to aid in the concrete curing. At discretion of the structure engineer, the sand layer may be eliminated. Converse does not practice in the field of moisture vapor transmission evaluation/mitigation since this does not fall under the geotechnical disciplines. Therefore, we recommend that a qualified person, such as the flooring contractor, structural engineer, and/or architect be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction.

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.

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.

9.7 Settlement

The total settlement of shallow footings from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1 inch or less. The differential settlement resulting from static loads is anticipated to be 0.5 inches or less over a horizontal distance of 40 feet.

Our analysis of the potential dynamic settlement is presented in Appendix C, *Liquefaction and Settlement Analyses*. We estimate that the project site has the potential for up to 0.22 inches of dry seismic settlement and the potential for liquefaction settlement of 0.81 inches. The dynamic differential settlement of the site may be half of the total settlement over 40 horizontal feet.

Generally, the static and dynamic settlement do not occur at the same time. For design purposes, the structural engineer should decide whether static and dynamic settlement will be combined or not.

9.8 Soil Parameters for Pipe Design

Structural design requires proper evaluation of all possible loads acting on pipe. The stresses and strains induced on buried pipe 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 design are provided in the following table.

Table No. 7, Soil Parameters for Pipe Design

Soil Parameters	Value
Average compacted fill total unit weight (assuming 92% relative compaction), γ (pcf)	130.2
Angle of internal friction of soils, ϕ	30
Soil cohesion, c (psf)	50
Coefficient of friction between concrete and native soils, fs	0.35
Coefficient of friction between PVC pipe and native soils, fs	0.30
Bearing pressure against native soils (psf)	2,500
Coefficient of passive earth pressure, Kp	3.0
Coefficient of active earth pressure, Ka	0.33
Modulus of Soil Reaction E' (psi)	1,500

9.9 Bearing Pressure for Anchor and Thrust Blocks

An allowable net bearing pressure presented in Table No. 7, *Soil Parameters for Pipe Design* may be used for anchor and thrust block design against alluvial soils. Such thrust blocks should be at least 18 inches wide.

If normal code requirements are applied for design, the above recommended bearing capacity and passive resistances may be increased by 33 percent for short duration loading such as seismic or wind loading.

9.10 Soil Corrosivity

The results of chemical testing on one representative soil sample were evaluated for corrosivity with respect to common construction materials such as concrete and steel.

The test result is presented in Appendix B, *Laboratory Testing Program,* and general discussion pertaining to soil corrosivity is presented below.

The sulfate content of the sampled soil corresponds to American Concrete Institute (ACI) exposure category S0 for the sulfate concentration (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 will be exposed to moisture from precipitation and irrigation. Based on the project location and the results of chloride testing of the 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 compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

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

Table No. 8, 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 of the sample when saturated was 4,520 Ohm-cm. This indicates that the soil tested is moderately corrosive to ferrous metals in contact with the soils (Romanoff, 1957). Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site and site soils.

9.11 Flexible Pavement Recommendations

For pavement design, we have utilized an R-value of 36 and design Traffic Indices (TIs) ranging from 5 to 8.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2021), 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.

Table No. 9, Recommended Preliminary Pavement Sections

R-value	Traffic Index (TI)	Pavement Section		
		Option 1		Option 2
		Asphalt Concrete (inches)	Aggregate Base (inches)	Full AC Section (inches)
36	5	4.0	3.0	4.0
	6	4.5	4.0	5.0
	7	5.0	6.0	6.5
	8	5.5	8.0	7.5

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

Prior to placement of aggregate base, appropriate earthworks should be performed according to specifications provided in Section 8.0 *Earthwork Recommendations.*

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, 2021) 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.

9.12 Rigid Pavement Recommendations

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, Portland Cement Concrete Pavement (PCCP) for Light, Medium and Heavy Traffic Rigid Pavement. For pavement design, we have utilized a design subgrade R-value of 36 and design Traffic Indices (TIs) ranging from 5 to 8. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table.

Table No. 10, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
36	5	6.5
	6	7.0
	7	7.0
	8	7.5

The preceding pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,750 psi. The third point method of testing

beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are 3.0 inches to 1.0 inch, respectively.

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

Prior to placement of concrete, at least the upper 12 inches of subgrade soils below rigid pavement sections should be compacted to at least ninety-five percent (95%) relative compaction as defined by the ASTM D 1557 standard test method. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into pavement base and/or subgrade.

9.13 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, 2021).

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 within 3 percent of optimum moisture content for coarse-grained soils and 0 to 2 percent above optimum for fine-grained soils.

The cement concrete 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.

10.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

10.1 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 or pavement. Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing utilities or pavement may require vertical side wall 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.

10.2 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 in the following table.

Table No. 11, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal: Vertical) [1]
Sand with Silt (SP-SM), Silty Sand (SM), and Silty Sand with Gravel (SM)	C	0-4	vertical
	C	4-10	1.5:1

[1] Slope ratio is assumed to be constant from top to toe of slope, with level adjacent ground.

Shallow excavations up to 4 feet bgs can be vertical. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trenches should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring can be provided if necessary.

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.

11.0 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. Testing should be performed to determine density and moisture of the compacted soils. 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.

12.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by RHA Landscape Architects-Planners, Inc., the City of Pico Rivera, and their authorized agents, to assist in the design and construction 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 is 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, 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.

13.0 REFERENCES

AMERICAN CONCRETE INSTITUTE (ACI), 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, October 2014.

CALIFORNIA BUILDING STANDARDS COMMISSION (CBSC), 2022, California Building Code (CBC).

CALIFORNIA STATE WATER RESOURCES CONTROL BOARD (SWRCB), 2024, GeoTracker database (http://geotracker.waterboards.ca.gov/), accessed in June 2024.

CALIFORNIA DEPARTMENT OF WATER RESOURCES (DWR), 2024, Water Data Library (http://wdl.water.ca.gov/waterdatalibrary/), accessed June 2024.

COUNTY OF LOS ANGELES, 2013, Manual for the Preparation of Geotechnical Reports, July 1, 2013.

DAS, B.M., 2011, Principles of Foundation Engineering, Seventh Edition, published by Global Engineering, 2011.

Los Angeles County (2021), Administrative Manual--Guidelines for Design, Investigation, and Reporting Low Impact Development Storm Water Infiltration

PUBLIC WORKS STANDARDS, INC., 2021, Standard Specifications for Public Works Construction (“Greenbook”), 2021.

ROMANOFF, MELVIN, 1957, Underground Corrosion, National Bureau of Standards Circular 579, dated April 1957.

U.S. GEOLOGICAL SURVEY (USGS), 2024, National Water Information System: Web Interface (http://nwis.waterdata.usga.gov/nwis/gwlevels), accessed June 2024.

Appendix A

Field Exploration

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling of soil borings and conducting water percolation tests. During the site reconnaissance, the surface conditions were noted, and the borings were marked at locations approved by Mrs. Jennifer Salazar with RHA Landscape Architects-Planners. The approximate boring locations were established in the field with reference to local streets and other visible features. The locations should be considered accurate only to the degree implied by the method used.

Three exploratory soil borings (BH-01 through BH-03) were advanced within the footprint of the proposed improvements on April 22, 2024. The planned depths of the borings were between 10.0 and 50.0 feet below ground surface (bgs). The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. The borings were visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Borings details are presented in Table No. A-1, *Summary of Soil Borings and Percolation Test Holes* and Drawings Nos. A-2 through A-7, *Logs of Borings.*

Three percolation test holes (PT-01 through PT-03) were drilled on April 22, 2024, and tested on April 24, 2024, within the project site to perform percolation testing. The test holes were drilled to depths between 4.5 and 5.0 feet bgs, the location PT-02 was drilled using a 5-inch diameter handheld auger. Details about the percolation tests are presented in Appendix D, *Percolation Testing.*

Table No. A-1, Summary of Soil Borings and Percolation Test Holes

Boring No.	Boring Depth (ft, bgs)		Groundwater Depth (ft, bgs)	Date Completed
	Proposed	Completed		
BH-01	50.0	51.3	25.0	4/22/2024
BH-02	10.0	11.5	Not Encountered	4/22/2024
BH-03	10.0	11.5	Not Encountered	4/22/2024
PT-01	5.0	4.5	Not Encountered	4/24/2024
PT-02	5.0	5.0	Not Encountered	4/24/2024
PT-03	5.0	4.6	Not Encountered	4/24/2024

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. 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 in plastic bags.

Standard Penetration Testing (SPT) was also performed at BH-01 in accordance with the ASTM Standard D1586 test method 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 *Log of Boring*.

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 log at the top of the next drive sample.

Following the completion of logging and sampling, boring BH-01 were backfilled with cement grout. The locations BH-02 and BH-03 were backfilled with soil cuttings mixed with cement and compacted by pushing down with an auger using the drill rig weight. After completion of percolation testing, the pipes were removed from PT-01 through PT-03 and the locations were backfilled with soil cuttings and compacted. If construction is delayed, the surface of the borings may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that might occur or provide protection around the boring locations to prevent trip and fall injuries from occurring near the area of any potential settlement.

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)
WA	Passing No. 200 Sieve

BORING LOG SYMBOLS

SAMPLE TYPE

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

DRILLING METHOD SYMBOLS

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

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. **22-31-304-01**

Drawing **A-1a**

Project ID: 22-31-304-01.GPJ;Template: KEY

CONSISTENCY OF COHESIVE SOILS

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

APPARENT DENSITY OF COHESIONLESS SOILS

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

MOISTURE

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

PERCENT OF PROPORTION OF SOILS

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

SOIL PARTICLE SIZE

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

PLASTICITY OF FINE-GRAINED SOILS

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

CEMENTATION/ Induration

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

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

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Log of Boring No. BH-01

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 173 Depth to Water (ft, bgs): 25.0

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL: **SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in maximum dimension, rootlets, moist, brown.						R
5		**ALLUVIUM:** **SILTY SAND (SM):** fine to coarse-grained, scattered gravel up to 1" in maximum dimension, moist, yellow brown with hint of orange oxidation. **- 5':** medium dense. **- 7.5':** coarser sand, gravel up to 1.5" in maximum dimension, silt lenses, no oxidation, whitish yellow.	■	▨	8/13/13 8/9/15	4 7	102 110	DS PA
10		**SAND WITH SILT AND GRAVEL (SP-SM):** fine to coarse-grained, gravel up to 1.5" in maximum dimension, dense, moist, whitish yellow.	■		10/20/21	3	116	
15		**SAND WITH SILT (SP-SM):** fine to coarse-grained, few gravel up to 2" in maximum dimension, very dense, moist, yellow brown.	■		12/27/34	4	107	
20		**- 20':** fine to medium-grained, scattered gravel up to 0.5" in maximum dimension, dense.	☒		12/18/19	5		FC=7.4%
25		**- 25':** fine to coarse-grained, medium dense, becomes wet, groundwater.	☒		10/9/8	15		
30		**- 30':** few gravel up to 1.5" in maximum dimension.	☒		5/15/15	15		FC=6.2%

Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. **22-31-304-01**

Drawing No. **A-2a**

Log of Boring No. BH-01

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 173 Depth to Water (ft, bgs): 25.0

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		ALLUVIUM: **SAND WITH SILT (SP-SM):** fine to coarse-grained, few gravel up to 1.5" in maximum dimension, very dense, wet, yeellow brown.	■		45/50-2"	18	88	
40		**SAND WITH SILT AND GRAVEL (SP-SM):** fine to coarse-grained, gravel up to 1.5" in maximum dimension, very dense, wet, yellow brown.	☒		6/38/37	6		FC=5.7%
45		**SAND WITH GRAVEL (SP):** fine to coarse-grained, coarser sand, gravel up to 2.5" in maximum dimension, very dense, wet, yellow brown.	■		9/20/50-5"	9	130	
50		**- 50':** rock fragments.	☒		9/21/50-3"	6		FC=3.9%
		End of Boring at 51.3 feet bgs. Groundwater encountered at 25 feet bgs and stabilized at 21.8 feet bgs. Borehole backfilled with cement grout and capped with soil cuttings compacted with augers using the drill rig weight on 4/22/2024.						

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Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

Drawing No. A-2b

Log of Boring No. BH-02

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 174 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		FILL: **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1.5" in maximum dimension, moist, reddish brown.						CP, CR, EI
5		**ALLUVIUM:** **SAND WITH SILT (SP-SM):** fine to medium-grained, moist, whitish yellow. **- 5':** medium dense.			4/7/10	5	96	CL
		- 7.5': fine to medium-grained.			6/12/20	2	104	
10		**- 10':** scattered gravel 0.5" in maximum dimension, dense.			10/17/29	3	103	
		End of boring at 11.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 4/22/2024.						

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Rio Hondo Park Improvements
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City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

Drawing No. A-3

Log of Boring No. BH-03

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 172 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		FILL: **SILTY SAND (SM):** fine to coarse-grained, few gravel up to 1" in maximum dimension, moist, reddish brown. **- 2':** becomes brown.						PA
5		**ALLUVIUM:** **SAND WITH SILT (SP-SM):** fine to medium-grained, moist, grayish yellow. **- 5':** medium dense.	■		9/12/14	7	97	CL
		- 7.5': fine to coarse-grained, scattered gravel up to 1.5" in maximum dimension, whitish yellow with orange oxidation.	■		8/12/13	5	103	
10		**- 10':** fine to medium-grained, no gravel, yellow brown.	■		3/6/11	10	91	
		End of boring at 11.5 feet below ground surface. No groundwater was encountered. Boring was backfilled with soil cuttings mixed with cement and compacted with augers using the drill rig weight on 4/22/2024.						

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Rio Hondo Park Improvements
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City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

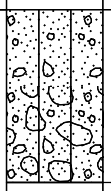
Drawing No. A-4

Log of Boring No. PT-01

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 172 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						

FILL:

SILTY SAND WITH GRAVEL (SM): fine to coarse-grained, gravel up to 2.5" in maximum dimension, moist, light brown..

- 2': increasing gravel and cobble content up to 4" in maximum dimension, dark brown.

End of boring at 4.5 feet bgs.
Groundwater was not encountered.
Borehole was used for percolation testing.
After completeion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 4/24/2024.

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Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

Drawing No. A-5

Log of Boring No. PT-02

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

Equipment: 5" DIAMETER HAND AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 173 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		FILL: **SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, gravel up to 3" in maximum dimension, moist, dark brown.						
5		**ALLUVIUM:** **SAND WITH SILT (SP-SM):** fine to medium-grained, silt lenses, moist, yellow brown.						
		End of boring at 5.0 feet bgs. Groundwater was not encountered. Borehole was used for percolation testing. After completeion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 4/24/2024.						

Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

Drawing No. A-6

Log of Boring No. PT-03

Date Drilled: 4/22/2024 Logged by: Aleksey Zhukov Checked By: Hashmi Quazi

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

Ground Surface Elevation (ft): 175 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.						
		FILL: **SILTY SAND WITH GRAVEL (SM):** fine to coarse-grained, gravel up to 2" in maximum dimension, moist, dark brown.						
		ALLUVIUM: **SAND WITH SILT (SP-SM):** fine to medium-grained, moist, whitish yellow.						
		End of boring at 4.6 feet bgs. Groundwater was not encountered. Borehole was used for percolation testing. After completeion of percolation testing, pipe was removed from borehole, backfilled with soil cuttings and compacted on 4/24/2024.						

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Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. 22-31-304-01

Drawing No. A-7

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 ASTM 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 (EI)

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

Table No. B-1, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Classification	Expansion Index	Expansion Potential
BH-02	0-5	Silty Sand (SM)	0	Very Low

R-Value (R)

One representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844 test method. The test provides a relative measure of soil strength for use in pavement design. The test result is summarized in the following table.

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

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-01	0-5	Silty Sand with Gravel (SM)	36

Soil Corrosivity Test (CR)

One representative soil sample was tested to determine minimum electrical resistivity (wet condition), 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 construction materials. The test was performed

by Keegan Lab, LLC. (Pomona, CA) in accordance with Caltrans Tests 643, 422 and 417. The test result is presented in the following table.

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

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-02	0-5	8.0	43	31	4,520

Collapse Potential (CL)

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, two collapse tests were performed on relatively undisturbed ring samples in accordance with the ASTM Standard D4546 laboratory procedure. The samples were loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test results are presented in the following table.

Table No. B-4, Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-02	5.0-6.5	Sand with Silt (SP-SM)	-1.3	Slight
BH-03	5.0-6.5	Sand with Silt (SP-SM)	-0.4	Slight

Percent Passing #200 Sieve (FC)

Four percent passing (#200 sieve) tests were performed on selected samples in accordance with ASTM Standard D1140. The results are presented in the following table.

Table No. B-5, Percentage Passing #200 Sieve Results

Boring No.	Depth (feet)	Soil Description	Passing #200 Sieve (%)
BH-01	20.0-21.5	Sand with Silt (SP-SM)	7.4
BH-01	30.0-31.5	Sand with Silt (SP-SM)	6.2
BH-01	40.0-41.5	Sand with Silt (SP-SM)	5.7
BH-01	50.0-51.3	Sand (SP)	3.9

Particle Size Analysis (PA)

To assist in classification of soils, mechanical grain-size analysis were performed on two select sample in accordance with the ASTM Standard D6913. The grain-size curves are shown in Drawing No. B-1, *Grain Size Distribution Results* and summarized in the following table.

Table No. B-6, Grain Size Distribution Test Results

Boring No.	Depth (feet)	Soil Description	% Gravel	% Sand	% Silt	% Clay
BH-01	5-10	Silty Sand (SM)	2.0	62.7	35.3	
BH-03	0-5	Silty Sand (SM)	6.0	67.7	26.3	

Maximum Density and Optimum Moisture Content (CP)

Laboratory maximum dry density-optimum moisture content relationship test was performed on one representative bulk soil sample. The test was conducted in accordance with the ASTM Standard D1557 test method. The test result is presented in Drawing No. B-2, *Moisture-Density Relationship Result,* and is summarized in the following table.

Table No. B-7, Summary of Moisture-Density Relationship Result

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-02	0-5	Silty Sand (SM), Reddish Brown	9.1 (8.1*)	129.7 (133.1*)

* Rock correction, BH-02 = 11.0%

Direct Shear (DS)

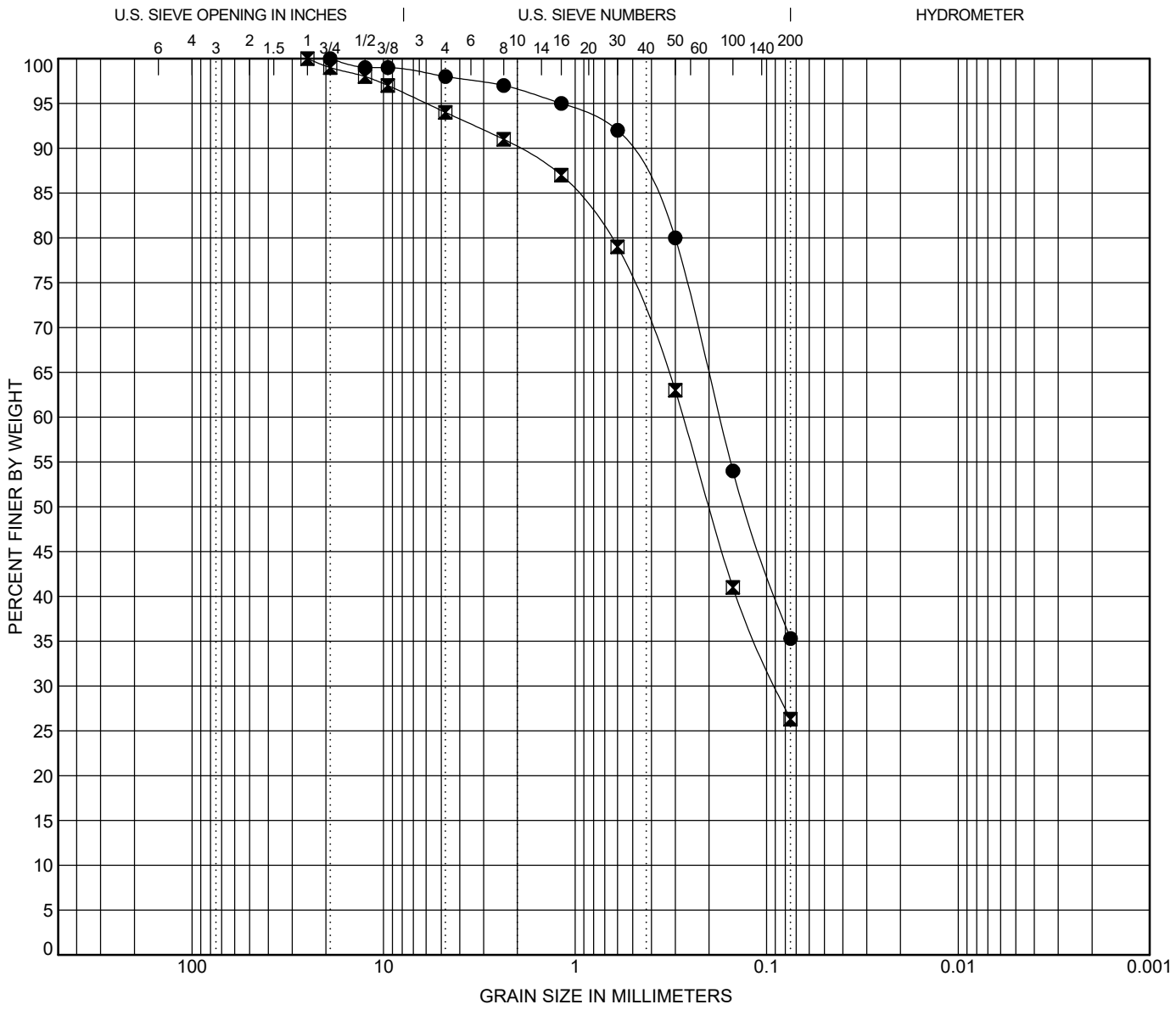
One direct shear test was performed on relatively undisturbed samples under soaked moisture condition in accordance with ASTM D3080. For the test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.015 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 Drawing No. B-3*, Direct Shear Test Result,* and the following table.

Table No. B-8, Summary of Direct Shear Test Result

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-01	5.0-6.5	Silty Sand (SM)	30	160

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-01**	**5-10**	**SILTY SAND (SM)**					
⊠	**BH-03**	**0-5**	**SILTY SAND (SM)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-01**	**5-10**	**19**	**0.176**			**2.0**	**62.7**	**35.3**	
⊠	**BH-03**	**0-5**	**25**	**0.273**	**0.089**		**6.0**	**67.7**	**26.3**	

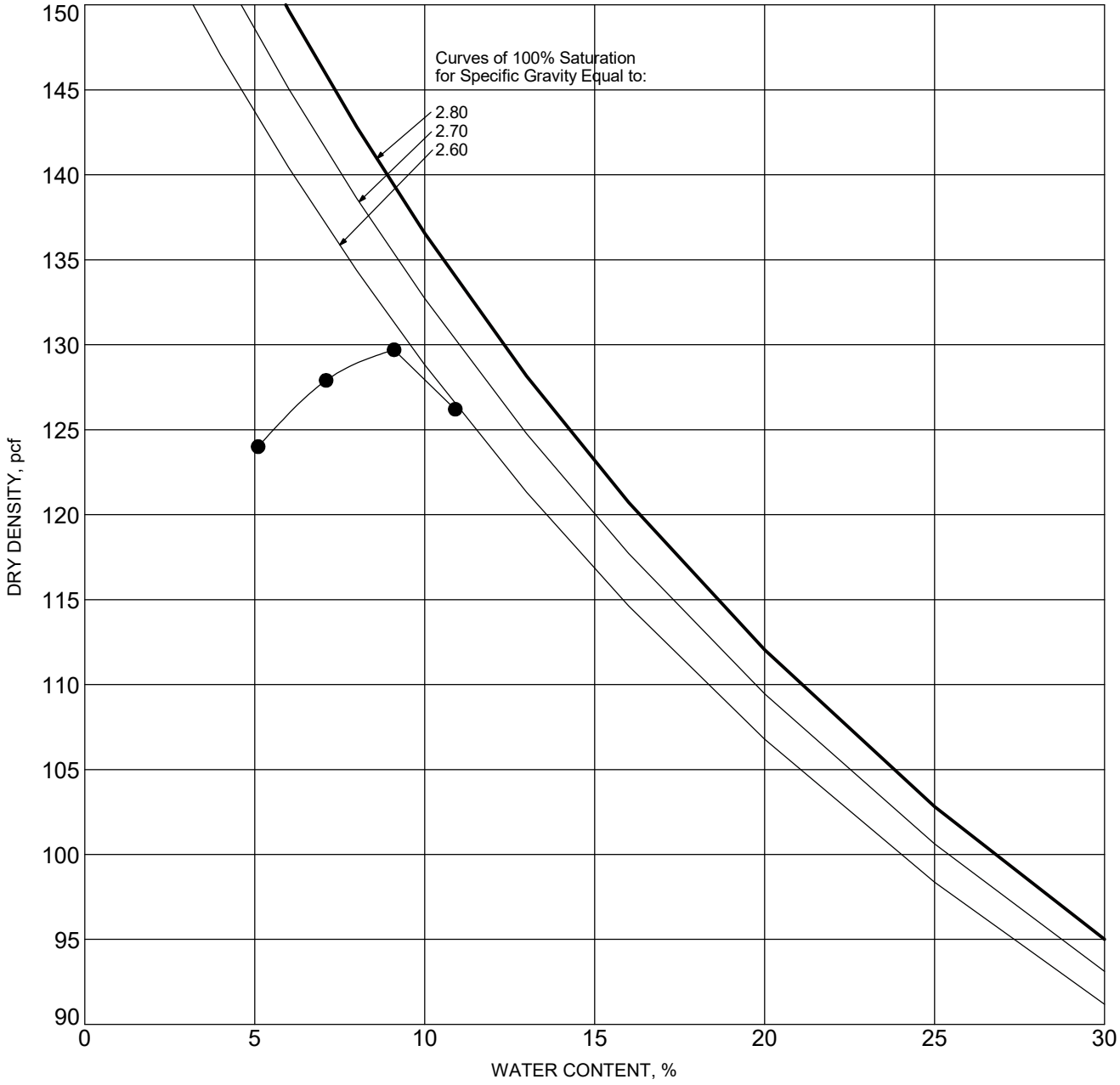
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. **22-31-304-01**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-02**	**0-5**	**SILTY SAND (SM), REDDISH BROWN**	**D1557 Method B**	**9.1 (*8.1)**	**129.7 (*133.1)**

(* Rock correction: BH-01 = 11.0%)

MOISTURE-DENSITY RELATIONSHIP RESULT

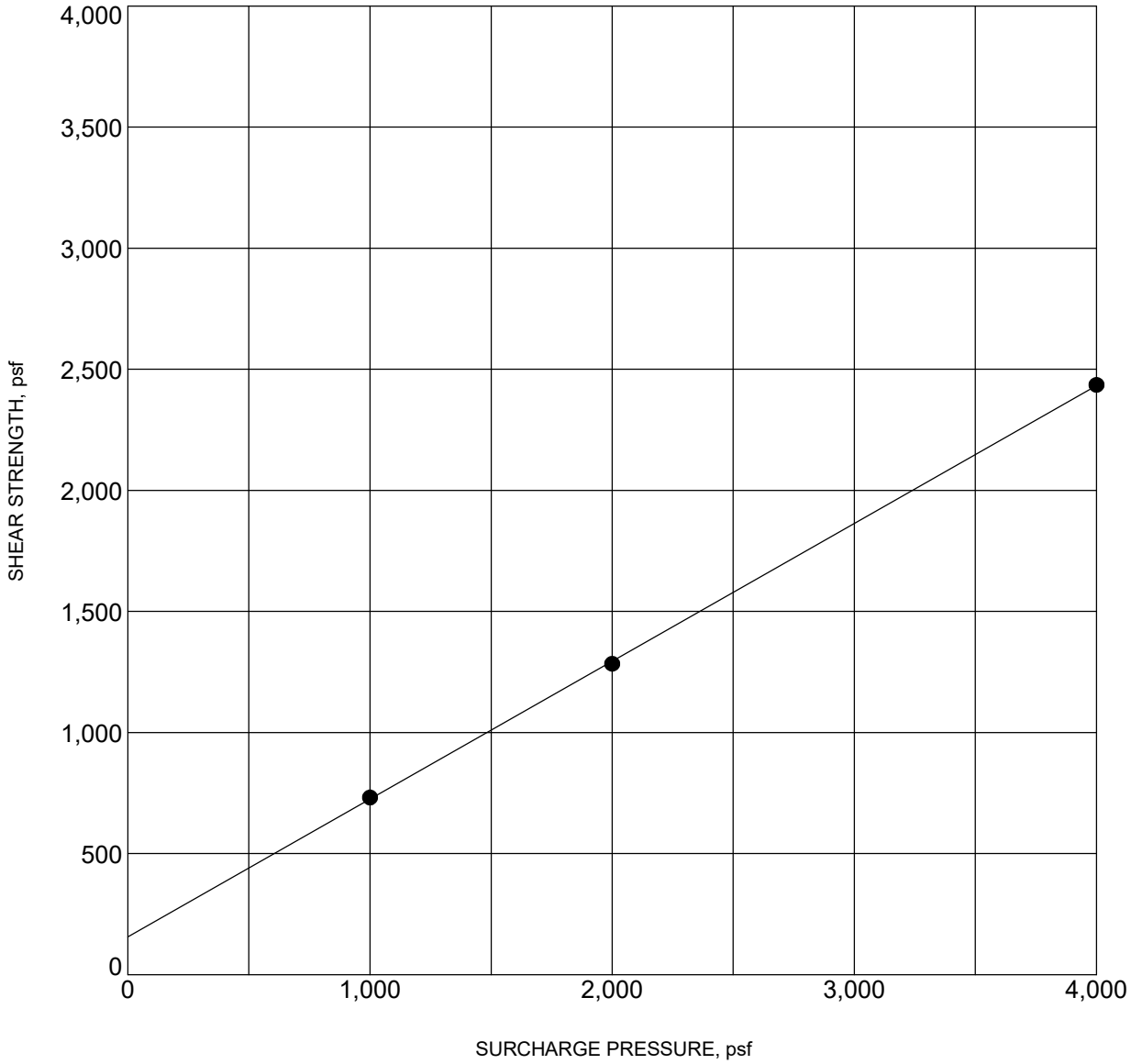
Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. **22-31-304-01**

Drawing No. **B-2**

Project ID: 23-31-168-01.GPJ; Template: COMPACTION



BORING NO.	:	**BH-01**	DEPTH (ft)	:	**5.0-6.5**
DESCRIPTION	:	**SILTY SAND (SM)**			
COHESION (psf)	:	**160**	FRICTION ANGLE (degrees):		**30**
MOISTURE CONTENT (%)	:	**4**	DRY DENSITY (pcf)	:	**102**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULT

Converse Consultants

Rio Hondo Park Improvements
8421 San Luis Potosi
City of Pico Rivera, Los Angeles County, California
For: RHA Landscape Architects-Planners

Project No. **22-31-304-01**

Drawing No. **B-3**

Appendix C

Liquefaction and Settlement Analyses

APPENDIX C

LIQUEFACTION AND SETTLEMENT ANALYSES

The subsurface data obtained from the boring BH-01 was used to evaluate the liquefaction potential and associated dry seismic settlement when subjected to ground shaking during earthquakes.

A simplified liquefaction hazard analysis was performed using the program SPTLIQ (InfraGEO Software, 2021) using the liquefaction triggering analysis method by Boulanger and Idriss (2014). A mean earthquake magnitude (M_w) of 7.30 was selected based on the results of seismic deaggregation analysis using the USGS interactive online tool (https://earthquake.usgs.gov/hazards/interactive/).

A peak ground acceleration (PGA_M) of 0.895g for the MCE design event, where g is the acceleration due to gravity, was selected for this analysis. The PGA_M was based on the CBC seismic design parameters presented in Section 6.0, *CBC 2022 Seismic Design Parameters.*

The result of our analysis is presented on Plates No. C-1 through C-3 and summarized in the following table.

Table C-1, Estimated Dynamic Settlements

Location	Groundwater Conditions	Groundwater Depth (feet bgs)	Dry Seismic Settlement (inches)	Liquefaction Induced Settlement (inches)
BH-01	Current	21.8	0.22	0.81
	Historical	22.5		

Based on our analysis, the project site has 0.22 inches of dry seismic settlement and the potential for liquefaction settlement of 0.81 inches. The dynamic differential settlement of the site may be half of the total settlement over 40 horizontal feet.

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

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PROJECT INFORMATION

Project Name	Rio Hondo Park Improvements
Project No.	22-31-304-01
Project Location	8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Analyzed By	Aleksey Zhukov
Reviewed By	Hashmi Quazi

SELECTED METHODS OF ANALYSIS

Analysis Description	
Triggering of Liquefaction	Boulanger-Idriss (2014)
Severity of Liquefaction	LPI: Liquefaction Potential Index based on Iwasaki et al. (1978)
Seismic Compression Settlement (Dry/Unsaturated Soil)	Pradel (1998)
Liquefaction-Induced Settlement (Saturated Soil)	Ishihara and Yoshimine (1992)
Liquefaction-Induced Lateral Spreading	Zhang et al. (2004)
Residual Shear Strength of Liquefied Soil	Idriss and Boulanger (2008)

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	7.30
Peak Ground Acceleration, A_{max}	0.90 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS

Boring No.	BH-01
Ground Surface Elevation	173.00 feet
Proposed Grade Elevation	173.00 feet
GWL Depth Measured During Test	21.80 feet
GWL Depth Used in Design	21.80 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER (%)	86.00 %
Hammer Distance to Ground Surface	5.00 feet
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)
- Ground Slope, S (%)	1.00 <<= Leave this blank
- Free Face Distance to Slope Height Ratio, (L/H)	1.00 <<= Leave this blank Set H to zero =>> feet

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Screening *Susceptible Soil? (Y, N)*	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	N	141.5	MCal	26.00	26.30
5.00	7.50	SM	Y	106.7	MCal	26.00	35.30
7.50	10.00	SM	Y	117.6	MCal	24.00	35.30
10.00	15.00	SP-SM	Y	119.1	MCal	41.00	11.00
15.00	20.00	SP-SM	Y	110.9	MCal	61.00	9.00
20.00	25.00	SP-SM	Y	110.0	SPT1	37.00	7.40
25.00	30.00	SP-SM	Y	100.0	SPT1	17.00	6.80
30.00	35.00	SP-SM	Y	100.0	SPT1	30.00	6.20
35.00	40.00	SP-SM	Y	103.9	MCal	100.00	5.95
40.00	45.00	SP-SM	Y	130.0	SPT1	75.00	5.70
45.00	50.00	SP	Y	142.0	MCal	70.00	2.80
50.00	51.50	SP	Y	142.0	SPT1	100.00	3.90

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

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PROJECT INFORMATION	
Project Name	Rio Hondo Park Improvements
Project No.	22-31-304-01
Project Location	8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Analyzed By	Aleksey Zhukov
Reviewed By	Hashmi Quazi

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	7.30
Peak Ground Acceleration, A_{max}	0.90 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS		
Boring No.	BH-01	
Ground Surface Elevation	173.00 feet	
Proposed Grade Elevation	173.00 feet	
GWL Depth Measured During Test	21.80 feet	
GWL Depth Used in Design	21.80 feet	
Borehole Diameter	8.00 inches	
Hammer Weight	140.00 pounds	
Hammer Drop	30.00 inches	
Hammer Energy Efficiency Ratio, ER	86.00 %	
Hammer Distance to Ground Surface	5.00 feet	
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)	
- Ground Slope, S	0.00 %	
- Free Face (L/H) Ratio	1.00	H = feet

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils: 5.00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 6.95 *** (High risk, with moderate liquefaction effects)

Seismic Ground Settlements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Seismic Compression Settlement:	Pradel (1998)	0.22 inches	0.22 inches	0.22 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0.81 inches	0.81 inches	0.81 inches	(Saturated Soils)
Total Seismic Settlement:		1.03 inches	1.03 inches	1.03 inches	

Seismic Lateral Displacements:	Analysis Method	Upper 30 feet	Upper 50 feet	Upper 65 feet	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.44 inches	0.44 inches	0.44 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs}$ = f{$(N_1)_{60}$, FC} where $(N_1)_{60} = N_{field}\ C_N\ C_E\ C_B\ C_R\ C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008).

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7.5}$ MSF K_σ K_α, MSF = Magnitude Scaling Factor, K_σ = f[$(N_1)_{60}$, σ'_{vo}], K_α =1.0, (level ground), CSR = Cyclic Stress Ratio = 0.65 A_{max} (σ_{vo}/σ'_{vo}) r_d, and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

INPUT SOIL PROFILE DATA								LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +																	
Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field SPT Blow Count N_{field} (blows/ft)	Fines Content FC (%)	Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. for Vert. Stress C_N	SPT Corr. for Hammer Energy C_E	SPT Corr. for Borehole Size C_B	SPT Corr. for Rod Length C_R	SPT Corr. for Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
0.00	5.00	SM	N	141.50	MCal	26.00	26.30	353.75	353.75									1.000		0.582							1.03	0.44	0.00
5.00	7.50	SM	Y	106.70	MCal	26.00	35.30	840.88	840.88	1.366	1.433	1.150	0.800	0.650	22.3	30.4	36.0	0.990	1.100	0.576							1.03	0.44	0.00
7.50	10.00	SM	Y	117.60	MCal	24.00	35.30	1,121.25	1,121.25	1.248	1.433	1.150	0.850	0.650	21.9	27.3	32.8	0.982	1.100	0.572							0.94	0.35	0.00
10.00	15.00	SP-SM	Y	119.10	MCal	41.00	11.00	1,566.00	1,566.00	1.075	1.433	1.150	0.850	0.650	37.3	40.2	41.8	0.970	1.073	0.564							0.81	0.23	0.00
15.00	20.00	SP-SM	Y	110.90	MCal	61.00	9.00	2,141.00	2,141.00	0.988	1.433	1.150	0.950	0.650	62.1	61.3	62.0	0.951	0.980	0.553							0.81	0.23	0.00
20.00	25.00	SP-SM	Y	110.00	SPT1	37.00	7.40	2,693.25	2,593.41	0.946	1.433	1.150	0.950	1.000	57.9	54.8	55.0	0.930	0.922	0.562							0.81	0.23	0.00
25.00	30.00	SP-SM	Y	100.00	SPT1	17.00	6.80	3,218.25	2,862.57	0.861	1.433	1.150	0.950	1.000	26.6	22.9	23.0	0.908	0.946	0.594	0.247	0.42	LIQUEFY	665.46	100.00	0.81	0.23	0.00
30.00	35.00	SP-SM	Y	100.00	SPT1	30.00	6.20	3,718.25	3,050.57	0.891	1.433	1.150	1.000	1.000	49.5	44.0	44.1	0.884	0.873	0.627						0.00	0.00	0.00
35.00	40.00	SP-SM	Y	103.90	MCal	100.00	5.95	4,228.00	3,248.32	1.007	1.433	1.150	1.000	0.650	107.1	107.8	107.9	0.860	0.855	0.651						0.00	0.00	0.00
40.00	45.00	SP-SM	Y	130.00	SPT1	75.00	5.70	4,812.75	3,521.07	1.054	1.433	1.150	1.000	1.000	123.6	130.3	130.3	0.835	0.830	0.664						0.00	0.00	0.00
45.00	50.00	SP	Y	142.00	MCal	70.00	2.80	5,492.75	3,889.07	0.904	1.433	1.150	1.000	0.650	75.0	67.8	67.8	0.810	0.800	0.666						0.00	0.00	0.00
50.00	51.50	SP	Y	142.00	SPT1	100.00	3.90	5,954.25	4,147.77	1.269	1.433	1.150	1.000	1.000	164.8	209.2	209.2	0.794	0.781	0.663						0.00	0.00	0.00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

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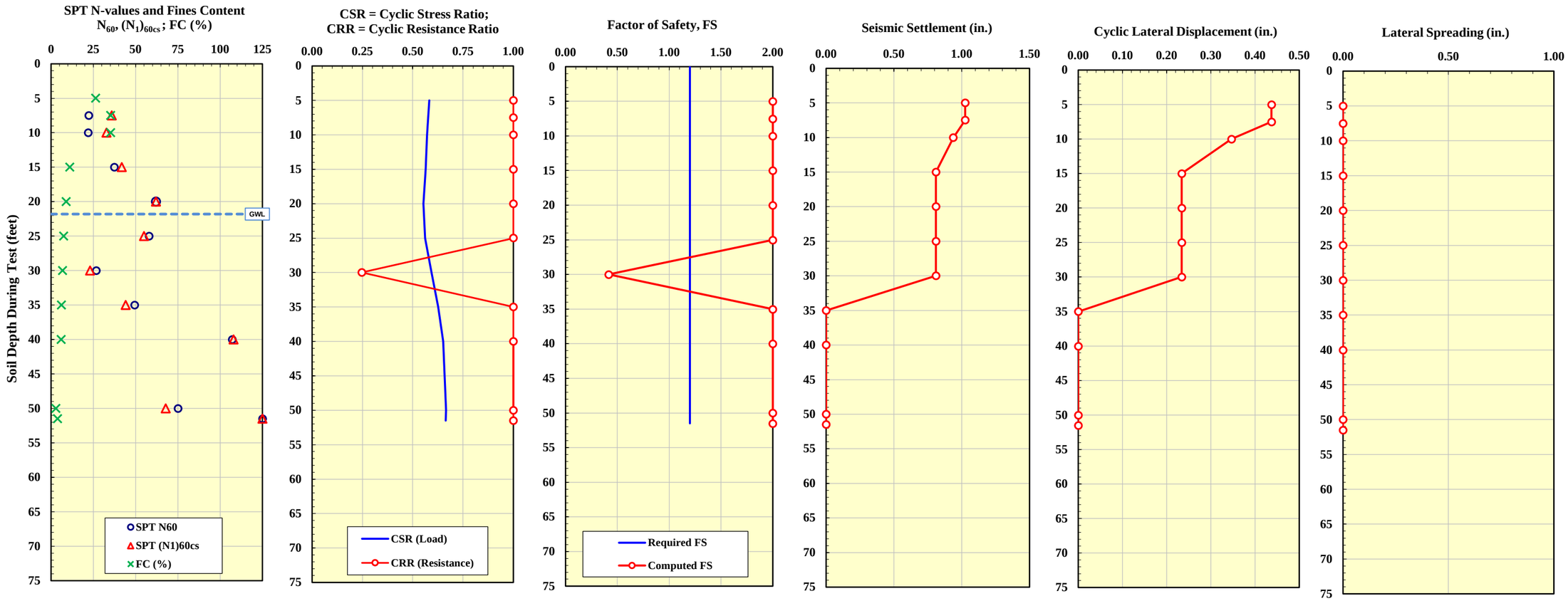
PROJECT INFORMATION	
Project Name	Rio Hondo Park Improvements
Project No.	22-31-304-01
Project Location	8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Analyzed By	Aleksey Zhukov
Reviewed By	Hashmi Quazi



Analysis Methods Used ==>>	Liquefaction Triggering:		Seismic Settlements:	Cyclic Lateral Displacements:	Lateral Spreading:
	Boulanger-Idriss (2014)	Above GWL:	Pradel (1998)	Pradel (1998)	Zhang et al. (2004)
		Below GWL:	Ishihara and Yoshimine (1992)	Tokimatsu and Asaka (1998)	

REFERENCES:

1. Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.
2. Bray, J.D., and Sancio, R.B. (2006). "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotech. and Geoenv. Engineering, ASCE 132 (9), 1165-1177.
3. Cetin, K.O. and Seed, R.B., et al. (2004), "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (12), 1314-1340.
4. Idriss, I.M. and Boulanger, R.W. (2008), "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute (EERI), Monograph MNO-12.
5. Ishihara, K. and Yoshimine, M. (1992), "Evaluation of settlements in sand deposits following liquefaction during earthquakes," Soils and Foundations, Japanese Geotechnical Society, 32 (1), 173-188.
6. Iwasaki, T., et al. (1978), "A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan," Proceedings Of 3rd International Conference of Microzonation, San Francisco, 885-896.
7. Olson, S.M. and Johnson, C.I. (2008), "Analyzing Liquefaction-Induced Lateral Spreads Using Strength Ratios," Journal of Geotech. and Geoenv. Engineering, ASCE 134 (8), 1035-1049.
8. Pradel, D. (1998), "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical Engineering, ASCE 124 (4), pp. 364-368.
9. Seed, R.B. and Harder, L.F. (1990), "SPT-based analysis of cyclic pore pressure generation and undrained residual strength, Proceedings Of Seed Memorial Symposium, Vancouver, B.C., 351-376.
10. Tokimatsu, K. and Seed, H.B. (1987), "Evaluation of settlements in sands due to earthquake shaking," Journal of Geotechnical Engineering, ASCE 113 (GT8), 861-878.
11. Tokimatsu, K. and Asaka, Y. (1998), "Effects of liquefaction-induced ground displacementson pile performance in the 1995 Hyogoken-Nambu Earthquake," Soils and Foundations, Special Issue, Japan Geotechnical Society, 163-177.
12. Toprak, S. and Holzer, T.L. (2003), "Liquefaction Potential Index: Field Assessment," Journal of Geotechnical and Geoenviromental Engineering, ASCE 129 (4), 315-322.
13. Youd, T.L, Idriss, I.M., et al. (2001), "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF Workshops", Journal of Geotech. and Geoenv. Engineering, ASCE 127 (10), 817-833.
14. Zhang, G, Robertson, P.K. and Brachman, R.W.I. (2004), "Estimating liquefaction-induced lateral displacement using the standard penetration test or cone penetration test," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (8), 861-871.

Appendix D

Percolation Testing



APPENDIX D

PERCOLATION TESTING

Percolation testing was performed utilizing test holes (PT-01 through PT-03) on April 24, 2024. The continuous pre-soak falling-head test method for water percolation testing was utilized to evaluate soil infiltration rates of the soils encountered between depths of 4.5 to 5.0 feet below the ground surface at the respective testing location in accordance with Los Angeles County (2021), Administrative Manual--Guidelines for Design, Investigation, and Reporting Low Impact Development Storm Water Infiltration. The test locations were prepared by placing a perforated 3-inch diameter PVC pipe surrounded by pea gravel after drilling. Water was filled to the ground surface elevation to pre-soak prior to testing.

The test holes were cased using a three-inch diameter perforated PVC casing. Water was added to the test hole until the water level was as near the ground surface as could be achieved and allowed to pre-soak, the pre-soaked hole drained entirely within thirty minutes after filling the hole two (2) consecutive times. The water level was measured to the nearest 1/8-inch. There were at least eight (8) readings that were taken or until a stabilized rate of drop was obtained, whichever occurred first. A stabilized rate is when the highest and lowest readings are within 10 percent of each other from three consecutive readings. Reduction factors RF (Based on the GS200.1 procedures), RF_t=1, RF_v=1 and RF_s=1 was used to calculate design infiltration rate. Design Infiltration rate was calculated as the average of the last three readings. Percolation test results can vary depending on location, depth, soil type and variability of the underlying alluvial sediments. The results of the percolation tests are presented on Plate Nos. D-1 through D-3 and are tabulated below.

Table No. D-1, Percolation Testing Results

Boring No.	Depth of Boring* (feet)	Predominant Soil Type (USCS)	Design Percolation Rate* (inches/hour)
PT-01	4.5	Silty Sand with Gravel (SM)	8.82**
PT-02	5.0	Silty Sand with Gravel (SM) and Sand with Silt (SP-SM)	0.27
PT-03	4.6	Silty Sand with Gravel (SM) and Sand with Silt (SP-SM)	0.50

*Design Percolation Rate is calculated as the average of the final three percolation test readings.
**Percolation rate is high due to the concentration of cobbles encountered, a rate of 0.50 should be applied at this location.

In accordance with County of Los Angeles requirements, the minimum percolation rate for design of infiltration systems for storm water management is 0.3 inches per hour. Design Engineer can use the field percolation readings attached in Appendix C and calculate the percolation rates. It should be noted that according to the Los Angeles County Low Impact Development, Best Management Practices Guidelines, any planned

infiltration systems should be at least 10 feet above historically highest groundwater levels. The project Civil Engineer shall review the percolation rates presented for design of the proposed infiltration system. Additional details about design and requirements can be found in the Low Impact Development Manual, County of Los Angeles Department of Public Works, latest edition. The infiltration system should be properly maintained periodically to minimize sedimentation in the infiltration system. The infiltration setback requirements per Los Angeles County is presented in the following table.

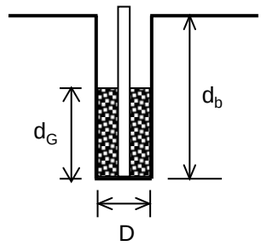
Table No. D-2, Infiltration Setback Requirements per Los Angeles County

Setback from	Distance
Property lines and public right of way	10 feet
Any foundation	15 feet or within 1:1 plane drawn up from the bottom of foundation, whichever greater
Face of any slope	H/2, 5 feet minimum (H is height of slope)
Water wells used for drinking water	100 feet
Historically highest groundwater levels	10 feet above

Percolation Testing

Job Name: Rio Hondo Park Improvements
Job No.: 22-31-304-01
Location: 8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Test Date: April 24, 2024

Test Boring No. **PT-01**
Depth of Boring (d_b): 4.5 feet
Diameter of Boring (D): 0.66 feet
Diameter of Casing (d_c): 0.25 feet
Percolation Test Zone: 1.8-4.5 feet bgs
Test Performer: A. Zhukov



Time of Testing			Water Level Measurement		Water Level Calculations				Infiltration Rate Calculations			
Initial Time	Final Time	Time Interval	Initial depth to water	Final depth to water	Initial Height of water column	Final Height of water column	Drop in Height	Average height of water column	Pre-adjusted Percolation Rate	Reduction Factor	Adjusted Percolation Rate	Infiltration Rate by Porchet Method*
T_i	T_f	ΔT	d_1	d_2	d_i	d_f	$\Delta d = d_i - d_f$	L_{ave}	$P_i = \Delta d / \Delta T$	R_f	$k = P_i / R_f$	$I = \Delta dD / [2\Delta T(D/2+2L_{ave})R_f]$
		(hr)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(inch/hr)		(inch/hr)	(inch/hr)
Percolation Test												
0.00	1.00	0.02	1.80	3.70	8.50	6.60	1.90	7.55	1368.00	3.0	456.00	9.75
1.00	2.03	0.02	1.97	3.70	8.33	6.60	1.73	7.47	1209.32	3.0	403.11	8.72
2.03	3.08	0.02	1.90	3.70	8.40	6.60	1.80	7.50	1234.29	3.0	411.43	8.86
3.08	4.05	0.02	2.05	3.70	8.25	6.60	1.65	7.43	1224.74	3.0	408.25	8.87

Reference: Adminstrative Manual - County of Los Angeles Public Works Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stomrwater Infiltration, dated 6/30/21

Reduction Factors:

RF_t (small diameter boring) = 1
RF_v (site variability) = 1
RF_s (siltation, plugging) = 1

Total Reduction Factor (RF) = RFt +RFv +RFs 3

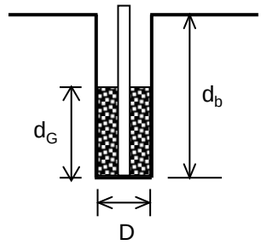
Design Infiltration Rate = 8.82 inch/hr

Note: Design Infiltration Rate = Average of last three adjusted precolation rate readings

Percolation Testing

Job Name: Rio Hondo Park Improvements
Job No.: 22-31-304-01
Location: 8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Test Date: April 24, 2024

Test Boring No. **PT-02**
Depth of Boring (d_b): 5.0 feet
Diameter of Boring (D): 0.42 feet
Diameter of Casing (d_c): 0.25 feet
Percolation Test Zone: 1.0-5.0 feet bgs
Test Performer: A. Zhukov



Time of Testing			Water Level Measurement		Water Level Calculations				Infiltration Rate Calculations			
Initial Time	Final Time	Time Interval	Initial depth to water	Final depth to water	Initial Height of water column	Final Height of water column	Drop in Height	Average height of water column	Pre-adjusted Percolation Rate	Reduction Factor	Adjusted Percolation Rate	Infiltration Rate by Porchet Method*
T_i	T_f	ΔT	d_1	d_2	d_i	d_f	$\Delta d = d_i - d_f$	L_{ave}	$P_i = \Delta d / \Delta T$	R_f	$k = P_i / R_f$	$I = \Delta dD / [2\Delta T(D/2+2L_{ave})R_f]$
		(hr)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(inch/hr)		(inch/hr)	(inch/hr)
Percolation Test												
0.00	10.00	0.17	1.03	2.16	9.27	8.14	1.13	8.70	81.36	3.0	27.12	0.32
10.00	20.00	0.17	1.69	2.66	8.61	7.64	0.97	8.12	69.84	3.0	23.28	0.30
20.00	30.00	0.17	2.67	3.42	7.63	6.88	0.75	7.25	54.00	3.0	18.00	0.26
30.00	40.00	0.17	2.17	3.04	8.13	7.26	0.87	7.69	62.64	3.0	20.88	0.28
40.00	50.00	0.17	2.13	2.99	8.17	7.31	0.86	7.74	61.92	3.0	20.64	0.28
50.00	60.00	0.17	2.27	3.08	8.03	7.22	0.81	7.62	58.32	3.0	19.44	0.26

Reference: Adminstrative Manual - County of Los Angeles Public Works Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stomrwater Infiltration, dated 6/30/21

Reduction Factors:

RF_t (small diameter boring) = 1
RF_v (site variability) = 1
RF_s (siltation, plugging) = 1

Total Reduction Factor (RF) = RFt +RFv +RFs 3

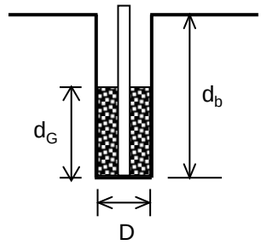
Design Infiltration Rate = 0.27 inch/hr

Note: Design Infiltration Rate = Average of last three adjusted precolation rate readings

Percolation Testing

Job Name: Rio Hondo Park Improvements
Job No.: 22-31-304-01
Location: 8421 San Luis Potosi, City of Pico Rivera, Los Angeles County, California
Test Date: April 24, 2024

Test Boring No. **PT-03**
Depth of Boring (d_b): 4.6 feet
Diameter of Boring (D): 0.66 feet
Diameter of Casing (d_c): 0.25 feet
Percolation Test Zone: 1.2-4.6 feet bgs
Test Performer: A. Zhukov



Time of Testing			Water Level Measurement		Water Level Calculations				Infiltration Rate Calculations			
Initial Time	Final Time	Time Interval	Initial depth to water	Final depth to water	Initial Height of water column	Final Height of water column	Drop in Height	Average height of water column	Pre-adjusted Percolation Rate	Reduction Factor	Adjusted Percolation Rate	Infiltration Rate by Porchet Method*
T_i	T_f	ΔT	d_1	d_2	d_i	d_f	$\Delta d = d_i - d_f$	L_{ave}	$P_i = \Delta d / \Delta T$	R_f	$k = P_i / R_f$	$I = \Delta dD / [2\Delta T(D/2+2L_{ave})R_f]$
		(hr)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(inch/hr)		(inch/hr)	(inch/hr)
Percolation Test												
0.00	10.00	0.17	1.17	3.10	9.13	7.20	1.93	8.17	138.96	3.0	46.32	0.92
10.00	20.00	0.17	1.80	3.14	8.50	7.16	1.35	7.83	96.96	3.0	32.32	0.67
20.00	30.00	0.17	1.93	3.04	8.37	7.26	1.12	7.82	80.40	3.0	26.80	0.55
30.00	40.00	0.17	1.98	2.98	8.32	7.32	1.01	7.82	72.48	3.0	24.16	0.50
40.00	50.00	0.17	1.66	2.73	8.64	7.57	1.08	8.11	77.52	3.0	25.84	0.52
50.00	60.00	0.17	1.73	2.74	8.57	7.56	1.01	8.07	72.96	3.0	24.32	0.49

Reference: Adminstrative Manual - County of Los Angeles Public Works Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stomrwater Infiltration, dated 6/30/21

Reduction Factors:

RF_t (small diameter boring) = 1
RF_v (site variability) = 1
RF_s (siltation, plugging) = 1

Total Reduction Factor (RF) = RFt +RFv +RFs 3

Design Infiltration Rate = 0.50 inch/hr

Note: Design Infiltration Rate = Average of last three adjusted precolation rate readings