Exhibit C

MOORE TWINING ASSOCIATES, INC.

**DRAFT**
**GEOTECHNICAL ENGINEERING AND**
**GEOLOGIC/SEISMIC HAZARD INVESTIGATION**
**PROPOSED REPLACEMENT HEADQUARTERS BUILDING**
**RIVERSIDE POLICE DEPARTMENT**
**4102 ORANGE STREET**
**RIVERSIDE, CALIFORNIA**

**Project Number: H01602.01**

For:

City of Riverside
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December 01, 2025

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MOORE TWINING ASSOCIATES, INC.

December 01, 2025 H01602.01

City of Riverside
8095 Lincoln Avenue
Riverside, CA 95204

Attention: Mr. Joseph Lopez, Capital Projects Manager

Subject: Geotechnical Engineering and Geologic/Seismic Hazard Investigation
Proposed Replacement Headquarters Building
Riverside Police Department
4102 Orange Street
Riverside, California

Dear Mr. Lopez:

We are pleased to submit this geotechnical engineering and geologic/seismic hazard investigation report conducted for the replacement headquarters building for the City of Riverside Police Department located at 4102 Orange Street in Riverside, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluations, conclusions, and recommendations.

We recommend that Moore Twining Associates, Inc. (Moore Twining) be retained to review those portions of the plans and specifications that pertain to earthwork, and foundations to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement, however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

We appreciate the opportunity to be of service to the City of Riverside. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
**MOORE TWINING ASSOCIATES, INC.**
Geotechnical Engineering Division

DRAFT

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# TABLE OF CONTENTS

**Page**

**1.0 INTRODUCTION** . . . . . 1

**2.0 PURPOSE AND SCOPE OF INVESTIGATION** . . . . . 1

2.1 Purpose . . . . . 1
2.2 Scope . . . . . 2

**3.0 BACKGROUND INFORMATION** . . . . . 3

3.1 Site History and Previous Studies . . . . . 3
3.2 Site Description . . . . . 3
3.3 Anticipated Construction . . . . . 4

**4.0 INVESTIGATIVE PROCEDURES** . . . . . 4

4.1 Field Exploration . . . . . 4
4.1.1 Site Reconnaissance . . . . . 4
4.1.2 Drilling Test Borings . . . . . 4
4.1.3 Soil Sampling . . . . . 5
4.1.4 Percolation Testing . . . . . 5
4.2 Laboratory Testing . . . . . 5

**5.0 FINDINGS AND RESULTS** . . . . . 6
5.1 Surface Conditions and Asphaltic Concrete Pavement Sections . . . . . 6
5.2 Soil Profile . . . . . 7
5.3 Soil Engineering Properties . . . . . 7
5.4 Results of Percolation Testing . . . . . 9
5.5 Groundwater Conditions . . . . . 10

**6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION** . . . . . 11
6.1 Geomorphic and Geologic Setting . . . . . 11
6.2 Tectonics and Seismicity . . . . . 12
6.3 Geologic Hazards . . . . . 13
6.3.1 Landslides . . . . . 13
6.3.2 Conditional Geologic Hazards . . . . . 13
6.3.2.1 Hazardous Materials . . . . . 13
6.3.2.2 Volcanic Activity . . . . . 14
6.3.2.3 Flooding and Dam Inundation . . . . . 14
6.3.2.4 Seiches and Tsunamis . . . . . 14
6.3.2.5 Radon Gas . . . . . 14
6.3.2.6 Naturally Occurring Asbestos . . . . . 14
6.3.2.7 Hydrocollapse . . . . . 15
6.3.2.8 Regional Subsidence . . . . . 15
6.3.2.9 Clays and Cyclic Softening . . . . . 15
6.4 Seismic Hazards . . . . . 15
6.4.1 Active Faulting and Surface Fault Rupture . . . . . 15
6.4.2 Groundshaking . . . . . 16
6.4.2.1 Historic Seismic Activity . . . . . 16
6.4.2.2 Design Seismic Ground Motion Parameters and Site Class . . . . . 16
6.4.3 Liquefaction and Seismic Settlement . . . . . 17

**TABLE OF CONTENTS (Cont'd)**

**7.0 GEOTECHNICAL EVALUATIONS** . . . . . 18
7.1 Existing Site Features and Surface Conditions . . . . . 18
7.2 Expansive Soils . . . . . 18
7.3 Static Settlement and Bearing Capacity of Shallow Foundations . . . . . 19
7.4 Mitigation of Bearing Support Failure due to Dry Seismic Settlement . . . . . 19
7.5 Temporary Excavations and Foundation Monitoring . . . . . 20
7.6 Asphalt Concrete (AC) Pavements . . . . . 20
7.7 Portland Cement Concrete (PCC) Pavements . . . . . 20
7.8 Soil Corrosion . . . . . 21
7.9 Sulfate Attack of Concrete . . . . . 22

**8.0 CONCLUSIONS** . . . . . 23

**9.0 RECOMMENDATIONS** . . . . . 24
9.1 General . . . . . 24
9.2 Site Grading and Drainage . . . . . 25
9.3 Storm Water Retention and Infiltration System(s) . . . . . 26
9.4 Deep Ground Modification for Dry Seismic Settlement Mitigation . . . . . 27
9.5 Site Demolition and Preparation . . . . . 29
9.6 Engineered Fill . . . . . 34
9.7 Shallow Spread Foundations . . . . . 37
9.8 Frictional Coefficients and Earth Pressures . . . . . 39
9.9 Retaining Walls and Site Walls . . . . . 41
9.10 Basement Slabs-on-Grade . . . . . 43
9.11 Exterior Slabs-on-Grade . . . . . 45
9.12 Asphalt Concrete (AC) Pavements . . . . . 46
9.13 Portland Cement Concrete (PCC) Pavements . . . . . .47
9.14 Slopes, Shoring and Temporary Excavations . . . . . 49
9.15 Utility Trenches . . . . . 50
9.16 Corrosion Potential . . . . . 52

**10.0 DESIGN CONSULTATION** . . . . . 53

**11.0 CONSTRUCTION MONITORING** . . . . . 53

**12.0 NOTIFICATION AND LIMITATIONS** . . . . . 53

**13.0 CLOSING** . . . . . 55

APPENDICES

APPENDIX A - Drawings . . . . . A-1

Drawing No. 1 - Site Location Map
Drawing No. 2 - Test Boring Location Map
Drawing No. 3 - Regional Geologic Map
Drawing No. 4 - Map of Major Active and Potentially Active Faults Relative to Site
Drawing No. 5 - Historical Earthquake Epicenter Map
Drawing No. 6 - Site Geologic Map
Drawing No. 7 - Geologic Soil Profile Cross Section A-A'
Drawing No. 8 - Geologic Soil Profile Cross Section B-B'

APPENDIX B - Logs of Test Borings . . . . . B-1

APPENDIX C - Results of Laboratory Tests . . . . . C-1

APPENDIX D - Results of Percolation Tests . . . . . D-1

APPENDIX E - Seismic Settlement Graphical Analysis . . . . . E-1

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**GEOTECHNICAL ENGINEERING AND**
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**RIVERSIDE, CALIFORNIA**

**Project Number: H01602.01**

## 1.0 INTRODUCTION

We are pleased to submit this geotechnical engineering and geologic/seismic hazard investigation report conducted for replacement of the City of Riverside Police Department Headquarters building at 4102 Orange Street in Riverside, California. Moore Twining Associates, Inc. (Moore Twining) was authorized to conduct this investigation by the City of Riverside.

The contents of this report include the purpose of the investigation and the scope of services provided. The existing site features and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, this report provides a summary of geotechnical engineering and geohazard evaluations, general conclusions, and related recommendations. The report appendices contain the drawings (Appendix A); the logs of borings (Appendix B); the results of laboratory tests (Appendix C); the results of the percolation testing (Appendix D), the graphical results of liquefaction and seismic settlement analysis (Appendix E).

## 2.0 PURPOSE AND SCOPE OF INVESTIGATION

**2.1 Purpose:** The purpose of this investigation was to conduct a field exploration and laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 Geotechnical engineering parameters for use in design of foundations;

2.1.2 Recommendations for site preparation, including placement, moisture conditioning, and compaction of engineered fill soils;

2.1.3 Recommendations for surface drainage;

2.1.4 Recommendations for slab-on-grade floors and exterior concrete flatwork;

2.1.5 Recommendations for asphalt concrete and Portland cement concrete pavement thickness;

2.1.6 Results of percolation tests and recommendations for storm water infiltration;

2.1.7 Descriptions of potential geologic / seismic hazards in accordance with the requirements of the current California Building Code and California Geological Survey – Note No. 48;

2.1.8 A soil profile type, site coefficients and adjusted Maximum Considered Earthquake spectral response acceleration parameters in accordance with the 2025 California Building Code; and

2.1.9 Recommendations for temporary excavations, trench excavation, trench backfill, and excavation stability;

2.1.10 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed improvements described in the Anticipated Construction section of this report. The purpose of our investigation was to provide geotechnical engineering parameters for use in design of foundations, slabs-on-grade, pavements, infiltration systems, and preparation of related construction documents. The intent of the investigation was also to evaluate potential geologic/seismic hazards. This investigation did not include in-place density tests, an environmental investigation, or an environmental audit.

This investigation also did not include conducting shear wave velocity measurements to a depth of 100 feet below site grade (BSG) for determination of the Site Class in accordance with the 2025 California Building Code and ASCE 7-22. A fee estimate was provided for conducting field shear wave velocity measurements to a depth of 100 feet. However, this supplemental work was not authorized during our investigation and prior to the preparation of this report. If these services are authorized, a supplemental report and revised recommendations will be presented after completion of the supplemental testing.

**2.2 Scope:** Our proposal, dated January 9, 2025, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows:

2.2.1 The Informal Bid Specification and Requirements request for Geotechnical Services and Testing, prepared by the City of Riverside (received on December 16, 2024), was reviewed.

2.2.2 A "Shaded Site Plan - OPT B", dated August 10, 2024, was provided via email correspondence on January 8, 2025, and was reviewed for project information. In addition, an As-Built Site Plan Schematic showing two (2) locations for percolation testing within landscape planters in the existing parking lot to remain was also reviewed.

2.2.3 A topographic survey, labeled "Riverside Police Admin Building", plot dated June 04, 2024, was reviewed for project information regarding as-built utility alignments and spot elevation.

2.2.4 A site reconnaissance and subsurface exploration were conducted.

2.2.5 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils encountered.

2.2.6 Mr. Joseph Lopez (Capital Projects Manager with the City of Riverside) and Mr. Matt Acton (Holt Architecture) were consulted during the investigation.

2.2.7 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and the engineering properties of the subsurface soils encountered.

2.2.8 This report was prepared.

## 3.0 BACKGROUND INFORMATION

The site history and previous studies, site description and the anticipated construction are summarized in the following subsections.

**3.1 Site History and Previous Studies:** Based on review of online historical aerial images for various years between 1948 and 2025, the 1948 and 1959 historical aerial images of the site indicate that the site was previously within a residential neighborhood. The City of Riverside Police Department Headquarters building first appears on a 1966 historical aerial image. Based on information obtained from the City of Riverside Department of Building and Safety, Records Division, a building permit was issued for a new Police Building at the site, dated May 1963. The historical aerial images and satellite images of the site through 2025 indicate that the site has not had any significant changes since the Police Department Headquarters building was constructed. Moore Twining requested the original building plans for the subject building, but these were not available at the time of the preparation of this investigation. Consequently, the existing foundation type and depth of foundation supporting the existing structure are not known.

No other previous reports of geotechnical engineering investigations, compaction testing or environmental studies conducted for this site were provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

**3.2 Site Description:** The existing City of Riverside Police Department Headquarters building is located at 4102 Orange Street in Riverside, California. The site is bounded to the northwest by Orange Street, to the northeast by the Riverside County Sheriff's Department building, to the southeast by Lemon Street, and to the southwest by the Riverside Court Hall of Justice parking structure. The existing Police Department Headquarters building and the limits of the replacement building are indicated on Drawing No. 2 in Appendix A.

The existing Police Department Headquarters building is a 3-story concrete building with brick siding, including a daylight basement level adjacent to an alleyway with edge parking. The floor elevation of the basement generally matches the pavement surface elevation of the adjacent alleyway on the south/southeast side of the building. Parking spaces are also along the south side of the building adjacent to the alleyway. In addition, a parking lot is located on the east side of the building adjacent to Lemon Street. The parking lot on the east side of the building is at a similar elevation to Lemon Street. The south side of the parking lot borders a retaining wall which includes a metal fence above the wall. The retaining wall has a maximum exposed height of about 11 feet on the southwest corner of the parking lot and tapers to match the driveway entrance at Lemon Street. The alleyway on the south side of the parking lot trends downward adjacent to the retaining wall to allow vehicle access into the basement level of the Police Department Headquarters building.

The north side of the building is bordered by a pedestrian plaza/corridor including concrete walkways and landscaped areas with large mature trees and tables with seating. The north side of the building also includes a concrete ramp and six-step stairway for entrance to the building. The west side of the building includes a grass lawn area that slopes gently down toward Orange Street and include a large mature tree and a transformer. A landscape wall extends off of the southwest corner of the building toward Orange Street and separates the grass lawn area on the west side of the building from a concrete walkway leading into the site from Orange Street. This concrete walkway leads to a stairway that descends to an access door at the basement level of the building. The landscape retaining wall on the west side of the building retains as much as about 2½ feet of soil. Although not observed, it is reported that a tunnel connects to the structure at the northwest corner of the building, which reportedly crosses below Orange Street. The depth and width of the tunnel is unknown.

The existing asphalt concrete parking lot adjacent to Lemon Street exhibited a high amount of cracking including block and longitudinal cracking and also localized areas of alligator cracking. The existing asphalt concrete pavement in the alleyway exhibited block cracking, raveling and localized areas of alligator cracking.

According to the USGS 7½-Minute Series Topographic Map (Riverside East Quadrangle), produced by the United States Geological Survey (USGS), the elevation of the site is about 850 feet above mean sea level (AMSL). The site is located at 33.978100 degrees latitude and -117.374999 degrees longitude.

**3.3 Anticipated Construction:** Based on the information provided, it is our understanding the project will include replacing the existing Riverside Police Department Headquarters building and the retaining wall bordering the south side of the existing parking lot. In addition, the existing asphalt concrete parking lot is planned to reconstructed in its current configuration. It is our understanding that the new structure will include a total of three (3)-stories (including a basement level for parking). The basement walls are anticipated to consist of reinforced concrete, and the above ground structure is anticipated to consist of a steel framed building. The building footprint is anticipated to be about 16,500 square feet.

It is further reported that the connection from the existing building to the below-grade pedestrian tunnel crossing Orange Street, located at the northwest portion of the existing building, will be abandoned/demolished.

Structural loads were not provided during preparation of this report. For the purpose of the report, we assumed structural loads of up to 150 kips for dead-plus-live column loads and about 5 kips per linear foot for dead-plus-live wall loads.

It is our understanding stormwater retention and/or infiltration systems may be considered. In addition, appurtenant construction is anticipated to include exterior hardscaping and landscape improvements.

## 4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections. The field exploration, number of test borings, sampling, and laboratory testing were performed in accordance with our proposal, dated January 9, 2025.

**4.1 Field Exploration:** The field exploration included a site reconnaissance, drilling test borings, conducting standard penetration tests, soil sampling and conducting percolation tests.

**4.1.1 Site Reconnaissance:** The site reconnaissance consisted of walking the site around the exterior of the existing building and noting visible surface features. The site reconnaissance was conducted by a Moore Twining staff engineer on August 20 and 21, 2025. The features noted are described in the "Background Information" section of this report.

**4.1.2 Drilling Test Borings:** Between August 20 and 21, 2025, eight (8) test borings were advanced at the site ranging in depth from about 5 to 57 feet below site grade (BSG), at which auger refusal was encountered due to very dense conditions. The borings included a hand auger boring HA-1 on the grass lawn area to a depth of 5 feet BSG on the west side of the building. The drill rig could not access the west lawn area due to wet soil conditions from irrigation and concerns with rutting and damage to the lawn. Borings B-1 through B-6 were drilled in the parking lot area on the east side of the building and also in the parking spaces adjacent to the alleyway on the south side of the building. Boring B-6 was used to conduct a percolation test (P-1). A second percolation test (P-2) was also drilled to a depth of 10 feet BSG in the parking lot area on the east side of the building. Other than the hand auger boring HA-1, the test borings were drilled using a CME-75 truck mounted drill rig equipped with 6-5/8 inch diameter hollow stem augers.

The approximate test boring locations are shown on Drawing No. 2 in Appendix A. The soils encountered in the test borings were logged during drilling by a representative of Moore Twining. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features. Soil samples were collected and returned to our laboratory for classification and soil mechanics testing. The presence and elevation of free water, if any, in the test borings were noted and recorded during the drilling.

Test boring locations were determined by field measurement with reference to existing site features. Elevations of the borings were not surveyed as a part of the investigation. The test borings were backfilled with soil cuttings and capped with asphaltic concrete cold patch where drilled in pavement areas, thus some settlement should be anticipated. Elevations of the ground surface at the boring locations are included on the boring logs in Appendix B based on a topographic survey, labeled "Riverside Police Admin Building", plot dated June 04, 2024.

**4.1.3 Soil Sampling:** Standard penetration tests were conducted, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler (SPT) has a 2 inch O.D. and a 1-⅜ inch inside diameter (I.D.). The sampler is driven by a 140 pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by driving a California modified split barrel sampler into the soil. The soil was retained in stainless steel rings, 2.5 inches O.D. and 1 inch in height. The lower 6 inch portion of the samples were placed in close-fitting, plastic, air-tight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

**4.1.4 Percolation Tests:** Two (2) falling head percolation tests were conducted on August 22nd, 2025 in areas planned for stormwater improvements at locations coordinated with Mr. Matt Acton (Holt Architecture). The percolation tests were conducted in accordance with Riverside County guidelines.

The test holes were cylindrical with a diameter of about 8 inches. Gravel packing was used to protect the sidewalls of the holes from washout during refilling. A 2-inch diameter screened/perforated PVC pipe was placed in the boreholes and used to transmit poured water to the bottom of the holes. On the day prior to the percolation testing, percolation test holes were presoaked with a minimum of about 5 gallons of water. The percolation tests were then conducted the following day by adding water to the test holes periodically and measuring the drop in water level over time. Measurements of water levels and the time of each reading were recorded on the field percolation test logs. The rate of water level decline near the end of the test period (generally stabilized) was used to estimate the average stabilized percolation rate of the soils tested. The head of the water in the test holes during the percolation testing was generally about 1 to 2 feet.

**4.2 Laboratory Testing:** The laboratory testing was programmed to determine selected physical and engineering properties of the samples tested. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface material encountered.

The results of laboratory tests are described in Section 5.3 of this report and the laboratory report forms are included in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

## 5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

**5.1 Surface Conditions and Existing Asphalt Concrete Pavement Sections:** The pavement section thicknesses encountered in the borings drilled in pavement areas are described below.

At the time of the field exploration, borings B-2, B-5, B-6/P-1, and P-2 were located within the existing asphalt concrete parking lot east of the existing building. These borings generally encountered an asphalt concrete section with a thickness of 5 to 6 inches overlying subgrade soils. A pavement fabric was encountered in Boring B-6/P-1.

Borings B-3 and B-4 were drilled within the parking stalls adjacent to the alleyway on the south side of the existing building. These borings encountered an asphalt concrete section with a thickness of 4 to 4½ inches overlying subgrade soils. A thin sheet of what was queried to be an aluminum foil type material was observed within boring B-3. No other locations encountered the aluminum layer and the age of placement is unknown.

Numerous existing underground utilities were noted within the asphalt concrete pavement areas and the landscape areas adjacent to the subject building.

**Table No. 1**
**Asphalt Concrete Pavement Section Thicknesses Encountered**

| Boring Number | Asphalt Concrete Thickness[1] (inches) | Aggregate Base Thickness[2] (inches) | Subgrade Soil |
|---|---|---|---|
| B-2 | 5.0 | N/A | Sandy Silt |
| B-3 | 4.5[(3)] | N/A | Clayey Sand |
| B-4 | 4.0 | N/A | Silty Sand |
| B-5 | 5.0 | N/A | Silty Sand |
| B-6/P-1 | 6.0[(4)] | N/A | Silty Sand |
| P-2 | 6.0 | N/A | Silty Sand |

[1] - The asphalt concrete thickness was measured on four sides to the nearest ¼ inch and averaged to the nearest inch.
[2] - Aggregate base thickness was not encountered at the boring locations.
[3] - Layer of suspected aluminum (foil) observed below a 2-inch overlay.
[4] - Pavement fabric encountered below a 2-inch overlay.

**5.2 Soil Profile:** The subsurface conditions encountered in the borings drilled for this investigation are described below.

Hand auger HA-1 was advanced within the grass area within the western side of the project site and generally encountered a silty sand fill soil extending to a depth of about 2-feet below site grade (BSG), which was underlain by native sandy lean clay to a depth of about 4-feet BSG. The native sandy lean clay was underlain by native silty sand extending to the maximum depth of exploration, 5-feet BSG.

Boring B-2, B-5, B-6/P-1, and P-2 were advanced below the existing asphalt concrete paved parking lot located to the east of the existing building. A sandy silt fill and silty sand fill was encountered extending to depths ranging from about 4 to about 18½ feet BSG within borings B-2 and B-5, respectively. The sandy silt fill encountered within boring B-2 to a depth of about 15-feet BSG was underlain by native, medium dense silty sand extending to 57-feet BSG, with exception of an interbedded lens of sandy silt from about 25-feet BSG to a depth of about 30-feet BSG. The silty sand fill encountered within boring B-5 extending to a depth of about 18½-feet BSG and was underlain by native clayey sand extending to a depth of about 20-feet BSG, the depth of exploration within boring B-5. Boring B-6/P-1 encountered a silty sand fill extending to a depth of about 4-feet BSG, which was underlain by native silty sand extending to a depth of about 8½ feet BSG, further underlain by sandy silt extending to about 10-feet BSG, the maximum depth of the boring. The boring advanced for percolation test P-2 encountered silty sand fill extending to 10-feet BSG, the maximum depth of exploration.

Borings B-3 and B-4 were advanced below the existing asphalt concrete paved alleyway located south of the existing building, which is at an elevation about 11-feet below the existing parking lot area elevation. Boring B-3 encountered a clayey sand fill extending to a depth of about 2-feet BSG, underlain by native silty sand extending to a depth of 10½ feet BSG. The silty sand was underlain by a sandy lean clay extending to a depth of 33½ feet BSG, further underlain by clayey sand extending to a depth of 35 feet BSG, the depth of exploration at this location. Below the asphalt pavement at boring B-4, a silty sand fill was encountered extended to a depth of about 2-feet BSG. The silty sand fill was underlain by native sandy silt extending to a depth of 7-feet BSG, which was further underlain by alternating layers of silty sand and sandy lean clay, extending to a depth of 30-feet BSG, the maximum depth of exploration at this location.

**5.3 Soil Engineering Properties:** The engineering properties of the subsurface soils encountered during this investigation are summarized below.

**FILL Silty Sands:** The silty sand fills encountered were described as very loose to loose, as determined by standard penetration resistance, N-values, ranging from 4 to 8 blows per foot, and a SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) ranging from 3 to 5 blows per foot. The moisture contents of the samples tested ranged from 5.4 to 10.5 percent. Two (2) relatively undisturbed samples revealed dry densities of 101.8 and 105.2 pounds per cubic foot.

**FILL Clayey Sands:** The clayey sand fills encountered were described as loose, as determined by standard penetration resistance, (SPT) equivalent N-value (estimated by driving a California Modified split barrel sampler) of 9 blows per foot. The moisture contents of the samples tested at 4.7 percent. The results of one (1) Atterberg limits test indicated a plasticity index of 13, with a corresponding liquid limit of 21.

**FILL Sandy Silt:** The sandy silt fills encountered were described as soft to stiff, as determined by standard penetration resistance, N-value of 3 blows per foot, and a SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 6 to 10 blows per foot. The moisture contents of the samples tested ranged from 5.5 to 6.7 percent. Three (3) relatively undisturbed samples revealed dry densities of 101.9, 105.2, and 117.2 pounds per cubic foot.

**Native Silty Sands:** The native silty sands encountered were described as medium dense to very dense, as determined by standard penetration resistance, N-values, ranging from 11 to greater than 50 blows per foot and a SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 5 to 26 blows per foot. The moisture contents of the samples tested ranged from 5.3 to 12.8 percent. Six (6) relatively undisturbed samples revealed dry densities of 100.2, 108.0, 108.4, 110.6, 115.7, and 122.5 pounds per cubic foot. The results of an Atterberg limits test collected from boring B-4 at a depth of 7 to 8½ feet BSG indicated a non-plastic plasticity index and a non-viscous liquid limit. The results of one (1) direct shear test conducted on a sample collected from boring B-2 indicated an internal angle of friction of 36 degrees with a cohesion value of 560 pounds per square foot. The results of one (1) direct shear test conducted on a sample collected from boring 4 indicated an internal angle of friction of 40 degrees with a cohesion value of 130 pounds per square foot. A consolidation test conducted on a sample collected from depths of 20 to 21½ feet BSG from boring B-2 indicated 5 percent consolidation under a load of 16 kips per square foot. Upon inundation, the sample tested exhibited a slight collapse under a load of 2.0 kip per square foot. Another consolidation test conducted on a sample collected from depths of 8½ to 10 feet BSG from boring B-3 indicated 5 percent consolidation under a load of 16 kips per square foot. Upon inundation, the sample tested exhibited an 0.6 percent collapse under a load of 2.0 kip per square foot.

**Native Clayey Sands:** The clayey sand soils encountered were described as loose to dense, as determined by standard penetration test (SPT), N-values, ranging from 8 to 35 blows per foot, and a SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 12 to 23 blows per foot. The moisture contents of the samples tested ranged from 7.5 to 13 percent. Four (4) relatively undisturbed samples revealed dry densities ranging from 110.4, 114.0, 116.2, and 125.7 pounds per cubic foot. A consolidation test conducted on a sample collected from depths of 1 to 2½ feet BSG from boring B-3 indicated 3¾ percent consolidation under a load of 16 kips per square foot. Upon inundation, the samples tested exhibited about ⅓ percent collapse under a load of 2.0 kip per square foot. Another consolidation test conducted on a sample collected from depths of 18½ to 20 feet BSG from boring B-5 indicated 6¼ percent consolidation under a load of 16 kips per square foot. Upon inundation, the samples tested exhibited about ½ percent collapse under a load of 2.0 kip per square foot. The results of an Atterberg limits test collected from boring B-5 at a depth of 18½ to 20 feet BSG indicated a plasticity index of 10 and a corresponding liquid limit of 21.

**Native Sandy Lean Clays:** The native lean clay soils encountered were described as stiff to very stiff, as determined by standard penetration test (SPT), N-values, of 11 to 19 blows per foot, and a SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 9 to 23 blows per foot. The moisture contents of the samples tested ranged from 10.6 to 17.3 percent. Three (3) relatively undisturbed samples revealed dry densities ranging from 106.9, 110.6 and 114.2 pounds per cubic foot. A consolidation test conducted on a sample collected from depths of 18½ to 20 feet BSG from boring B-3 indicated about 6¾ percent consolidation under a load of 16 kips per square foot. Upon inundation, the sample tested exhibited about ½ percent collapse under a load of 2.0 kip per square foot. The results of an Atterberg limits test collected from hand auger boring HA-1 at a depth of 2 to 2½ feet BSG indicated a plasticity index of 13, with corresponding liquid limit of 21. The results of an Atterberg limits test collected from boring B-3 at a depth of 13½ to 15 feet BSG indicated a plasticity index of 10, with corresponding liquid limit of 23.

**Native Sandy Silts:** The native sandy silts encountered were described as soft to very stiff, as determined by standard penetration resistance, N-values, ranging from 5 to 22 blows per foot. The moisture contents of the samples tested ranged from 5.0 to 10.5 percent. One (1) relatively undisturbed samples revealed dry densities of 109.2 pounds per cubic foot. A consolidation test conducted on a sample collected from depths of 1 to 2½ feet BSG from boring B-4 indicated 4½ percent consolidation under a load of 16 kips per square foot. Upon inundation, the sample tested exhibited a swell of about ¼ percent under a load of 2.0 kip per square foot.

**Maximum Density / Optimum Moisture Tests:** One (1) Optimum Moisture / Maximum Density test was conducted on a near surface sample collected from depths of about 1 to 5 feet BSG from boring B-5. The test results indicated a maximum dry density of 133.6 pounds per cubic foot at an optimum moisture content of 8.3 percent.

**Expansion Index Testing**: One (1) Expansion Index Test was conducted on a near surface sample collected from boring B-3 at a depth of about 1 to 5 feet BSG indicated an expansion index of 0.

**R-Value Testing:** One (1) R-value test was conducted on a near surface sample collected from a depth of about 1 to 5 feet BSG from boring B-6. The test result indicated an R-value of 45.

**Chemical Analysis:** Chemical tests were performed on one (1) soil sample collected within the upper 5 feet BSG to evaluate the corrosion potential. The results of the chemical testing performed are summarized in Table No. 2 below. The laboratory test sheets are included in Appendix C.

**Table No. 2 - Corrosion Potential Test Results**

| Location / Parking Lot | Depth (Feet) | pH | Sulfate (Percent by Weight) | Chloride (Percent by Weight) | Minimum Resistivity (ohm-cm) |
|---|---|---|---|---|---|
| B-3 | 1.0-5.0 | 8.1 | 0.0011 | ND | 9,300 |

ND - Not Detected (reporting limit of 0.0006 percent)

**5.4 Results of Percolation Testing:** The infiltration rates estimated from the percolation test data are summarized in Table No. 3 below. The percolation test data are included in Appendix D.

**Table No. 3**

| **Results of Percolation Tests and Summary of Estimated Unfactored Percolation and Infiltration Rates** | | | | |
|---|---|---|---|---|
| Test Location | Total Depth of Test Hole BSG (feet) | Approximate Average Unfactored Percolation Rate Measured [1] (Minutes/Inch) | Estimated Average Unfactored Infiltration Rate [1] (Inches per Hour) | Soil Type |
| P-1 | 10.33 | 12.5 [2] | 0.8 [2] | (ML) Sandy Silt |
| P-2 [3] | 9.96 | 6.9 [2] | 1.2 [2] | (SM) Silty Sand Fill |

1. - Includes no factor of safety (see discussion below).
2. - The values listed include the gravel correction factor.
3. - Percolation Test P-2 was installed to a depth that adjacent borings indicate is within fill.

BSG - Below site grade

The unfactored estimated percolation and infiltration rates above do not take into account the long term effects of subgrade saturation, silt accumulation, groundwater influence, nor densification as a result of the construction process. In general, the percolation/absorption rate of the soils will decrease when the soils are saturated and the absorption rate is further reduced the longer the soils are saturated. Published studies indicate short term field infiltration rates can significantly overestimate the saturated permeability. In addition, soil bed consolidation, sediment, suspended soils, etc. in the discharge water can result in clogging of the pore spaces in the soil. This clogging effect can also reduce the long term infiltration rate. Numerous other factors, such as variations in soil type and soil density across the entire area of the system can influence the percolation/infiltration rate, both short and long term. Based on these factors, infiltration systems have a limited life span.

Since the field testing is a small-scale test method and does not take into account the long term effects of subgrade saturation, silt/fines accumulation, infrequent maintenance, or mechanical densification of the soils as a result of the construction process, etc., appropriate safety factors should be applied to the estimated infiltration rates for design. Depending on the design methodology used, a safety factor ranging from 3 to 10 is generally recommended for infiltration system applications; however, the final safety factor should meet the requirements of the governing agency and should consider such factors as the sediment load of the stormwater, whether pretreatment of the stormwater is planned, the consequences of failure, the degree of maintenance that can be relied upon and the uncertainties in the estimated inflow volume. In addition, stormwater infiltration and retention systems should incorporate outfall of stormwater as part of the design for redundancy.

It should be further noted that the proposed stormwater structures will need to be setback a sufficient horizontal distance from retaining walls and basement walls to reduce potential impacts from seepage water and hydrostatic pressures on subsurface walls. This is explained in further detail within section 9.3.

**5.5 Groundwater Conditions:** Groundwater was not encountered at the time of the field investigation (August 2025) within the borings advanced to the maximum depth explored of 57 feet BSG.

Based on our review of water well data from the Department of Water Resources website, a well located about ½ mile north of the site indicated a high groundwater elevation of about 769 feet above mean sea level (AMSL), taken in March 2016, and a low groundwater elevation of about 763 feet AMSL taken in October, 2014. The subject well has been monitored from October 2011 to March of 2016. Based on the elevation of the subject site of about 840 to 850 feet AMSL and the 843 feet AMSL elevation of the subject monitoring well, these elevations would indicate an approximate high groundwater depth of about 70 to 80 feet BSG at the subject site.

Based on review of files on the Department of Water Resources and California State Water Resources Control Board (SWRCB) Geotracker website, a groundwater monitoring report for a site located approximately 550 feet southwest of the subject site was reviewed. Data from this closed case extended as far back as 2008. The data indicated that the shallowest groundwater recorded was about 70 feet BSG for measurements made in 2008.

It should be recognized, however, that groundwater is dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. Actual groundwater levels during construction could be higher or lower than indicated in this report. The evaluation of such factors was beyond the scope of this investigation and report.

## 6.0 GEOLOGIC AND SEISMIC HAZARDS EVALUATION

**6.1 GEOMORPHIC AND GEOLOGIC SETTING**: The City of Riverside is situated within the Peninsular Ranges geomorphic province, which features broad inland valleys separated by groups of rolling hills and rocky knolls. The Peninsular Ranges geomorphic province is surrounded by the Santa Ana Mountains to the southwest, the San Jacinto Mountains to the southeast, and the convergence of the San Gabriel and San Bernardino Mountain Ranges to the north.

In regards to physiography, the following is summarized from the City of Riverside Geologic and Seismic Technical Background Report, 2004:

> *"The City is controlled by the distribution and character of geologic units, by fault movements, and by climate and erosion. Riverside is within the northern end of the Peninsular Ranges physiographic province, approximately 12 miles south of its intersection with the Transverse Ranges physiographic province. The northwest to southeast trending Santa Ana Mountains (related to the Whittier-Elsinore fault zone and possibly an undocumented Santa Ana Mountains blind thrust) are approximately 15 miles south and southwest of the City, while northwest to southeast trending San Jacinto Mountains (associated with the San Jacinto fault zone) are about 10 miles east and northeast of Riverside.*
>
> *A series of hills and small mountains, comprised primarily of Cretaceous age crystalline rocks, surrounds the City. These hills and mountains are between the two dominant mountain ranges (San Jacinto and Santa Ana). They include La Loma Hills, Jurupa Mountains, Pedley Hills, La Sierra Hills and others. Within the City, surface elevations range from about 700 feet above mean sea level (AMSL) near the Santa Ana River to over 1,400 feet AMSL west of La Sierra. The highest point in the immediate vicinity is Arlington Mountain, standing 1,853 feet AMSL approximately 1-1/2 miles northwest of Lake Mathews.*
>
> *Lying between the Santa Ana and San Jacinto Mountain ranges are a series of small valleys separated by small mountains and hills. Generally within the City, ground surfaces slope northwest and have accumulated sediments shed from the mountains along streams and across alluvial fans. The City of Riverside lies along the southern edge of the Santa Ana River valley, just north of the foothills where Lake Mathews is located. This is the point where the Santa Ana River transitions from its north-south path, flowing generally from east to west across the City's northern boundary, before turning southward toward Prado Basin. The Santa Ana River borders the City on the north and northeast, with several small southeast-to-northwest trending tributaries flowing through the City."*

In regards to surficial deposits, the City of Riverside Geologic and Seismic Technical Background Report, 2004, indicated the following:

> *"Surficial deposits consist of relatively recent (geologically young) sediments formed by alluvial processes in streams, on alluvial fans/aprons, and primarily by the Santa Ana River and its tributaries. In general, past studies assumed that the uppermost surficial sediments are Holocene (less than 10,000-12,000 years old) to late Quaternary (less than 100,000 years old) in age, although few deposits have been age-dated by absolute methods (e.g., radiocarbon)."*

Based on the Geologic Map of the Riverside East 7.5-Minute Quadrangle, Riverside County, California, prepared by Douglas M. Morton and Brett Cox, dated 2001, the area of the proposed Police Department Headquarter Building is underlain by late to middle Pleistocene Epoch Quaternary sediments comprised of old alluvial fan deposits (Qof) consisting of *"indurated sandy alluvial fan deposits developed extensively in western part of quadrangle. Most of unit is slightly to moderately dissected and reddish-brown. Some Qof includes thin, discontinuous surface layer of Holocene alluvial fan material."*

According to the USGS 7½-Minute Series Topographic Map (Riverside East, California), produced by the United States Geological Survey (USGS), the area of the site with the Riverside Police Department building is at an elevation of about 850 feet above mean sea level (AMSL).

A regional geologic map is included as Drawing No. 3 in Appendix A of this report, and a site geologic map is included in Appendix A as Drawing No. 6. Geologic (soil profile) cross sections are included as Drawing Nos. 7 and 8 in Appendix A of this report.

**6.2 <u>TECTONICS AND SEISMICITY</u>**: An "active fault" is defined, for the purpose of this evaluation, as a fault that has had surface displacement within the Holocene Epoch (about the last 11,700 years). Potentially active faults also contribute to the seismicity of the site. A widely accepted definition of "potentially active" is a fault that does not show evidence of displacement during the Holocene, but shows evidence of displacement occurring less than 1.6 million years ago (after the beginning of the Pleistocene Epoch). Since the Quaternary Period includes the Holocene and Pleistocene Epochs, Quaternary Faults are those that have documented latest fault displacement within the Holocene or Pleistocene Epochs, undifferentiated. Faults showing evidence of displacement older than 1.6 million years are usually classified as "Pre-Quaternary" and "inactive." Pre-Quaternary faults do not impact the site seismicity.

The Fault Activity Map of California indicates that the nearest active fault to the site with surface rupture is the San Jacinto Fault, which is located nearly 7¼ miles northeast of the site. The map also indicates the active San Andreas Fault (South Branch) is located about 15 miles northeast of the site, and the Glen Ivy North Fault of the Elsinore Fault Zone is about 15½ miles southwest of the site.

Based on the results of deaggregation of the 2,475 year return period event (2 percent probability of exceedance in 50 years) for probabilistic analysis using the United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) Dynamic Conterminous U.S. 2014 (update), the numerous gridded fault model point sources (general seismicity not related directly to specific known seismic source) are the predominant contributing source of seismicity for the site area. The San Jacinto fault (San Bernardino segment) is estimated to be located about 7¼ miles horizontally from the site and also contributes significantly to the site seismicity. According to the deaggegation results, the San Andreas fault (San Bernardino North segment) contributes a lower percentage of the total seismicity of the site area. The Glen Ivy segment of the Elsinore fault zone and San Andreas (North Branch Mill Creek segment) fault contribute successively lower percentages of the total seismicity and account for the remaining seismicity of the site area. Attributes of the primary faults and fault zones contributing to the site seismicity are provided in the following Table No. 4.

**Table No. 4**
**List of Quaternary Faults Contributing to the Seismicity at the Site**

| Fault (Rupture Segment) | Predominant Moment Magnitude(s) | Horizontal Distance to Site (Miles) from United States Geological Survey deaggregation website |
|---|---|---|
| San Jacinto fault (San Bernardino segment) | 8.0 | 7.8 |
| San Andreas fault (San Bernardino North segment) | 7.9 | 14.7 |
| Elsinore fault (Glen Ivy segment) | 7.5 | 15.3 |
| San Andreas fault (North Branch Mill Creek segment) | 8.0 | 14.3 |

Deterministic analysis using the Building Seismic Safety Council 2014 (BSSC2014) Scenario Catalog from the USGS website for the Earthquake Hazards Program (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/) indicates that a magnitude 8.2 is assigned to the cascading segments of the South San Andreas fault (listed as S. San Andreas: PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO and also listed as S. San Andreas: CH+CC+BB+NM+SM+NSB+SSB+BG+CO).

The site location relative to known nearby fault systems within 60 miles of the site is depicted on Drawing No. 5 (Map of Major Active and Potentially Active Faults Relative to Site) in Appendix A of this report.

**6.3 GEOLOGIC HAZARDS**: The potential geologic hazards of flooding, landslides, and volcanic activity are described in the following subsections.

**6.3.1 Landslides**: The project site(s) have not been mapped by CGS for geologic hazards such as seismic landslide hazards. No significant slopes are located within or bordering the site. Thus, the potential for slope instability is not considered a potential impact for the proposed project.

**6.3.2 CONDITIONAL GEOLOGIC HAZARDS:** Conditional geologic hazards, as identified in California Geological Survey Note 48, are discussed in the following subsections.

**6.3.2.1 Hazardous Materials:** According to the Division of Oil and Gas, and Geothermal Resources GIS Well Finder website (https://maps.conservation.ca.gov/doggr/wellfinder/ and using site coordinates 33.978100, -117.37499), the subject site is not located in an area of oil or gas production. Hazardous materials such as methane gas, hydrogen-sulfide gas and tar seeps are not known to be present in the project area and are not considered to be a concern at the subject site.

**6.3.2.2 Volcanic Activity:** California includes six regions with a history of late Pleistocene volcanic eruptions that are subject to hazards from future eruptions (Miller, 1989). Of these six regions, one area is located within 100 miles of the site. The volcanic region of the Rodman Mountains area within the Amber Crater-Lavic Lake Area is the nearest volcanic region to the site, located about 65 miles northeast of the site.

Beyond the aforementioned Amber-Crater-Lavic Lake Area, no additional known areas of areas of known volcanism are present within 100 miles of the site. Based on the distance between the site and known volcanic areas, the prospect for basaltic lava flows or tephra hazards associated with basalt or basaltic-andesite vents at the site during the design life of the project is considered low.

**6.3.2.3 Flooding and Dam Inundation**: The Flood Insurance Rate Map, Community Panel number 06065C0726H, distributed by the Federal Emergency Management Agency, dated September 12, 2024, indicates the site is located in Zone X, which is defined as an "area of minimal flood hazard".

Based on review of the Draft Public Safety Element of the City of Riverside Technical Background Report (2021-2029), Safety Element Figure PS-4, entitled "Flood Hazard Areas", and based on our review of the Division of Safety of Dams, California Dam Breach Inundation Maps (GIS website: https://fmds.water.ca.gov/maps/damim/), failure of the Box Springs Dam is capable of inundating the south side of the site (parking spaces and alleyway), which would result in inundation of the basement level of the building.

**6.3.2.4 Seiches and Tsunamis**: A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. Lake Evans is the closest large body of water, located about 1⅛ miles northwest and about 100 feet lower in elevation compared to the subject site. Thus, seiches are not considered a significant hazard at the site.

Tsunamis are waves generated in oceans from seismic activity. The site is not located in a tsunami hazard zone. Based on our review of California Tsunami Hazard Area Maps and Data on the California Geological Survey website (https://www.conservation.ca.gov/cgs/tsunami/maps), due to the elevation of the site and distance to the ocean (Pacific Ocean is about 38½ miles southwest of the site), tsunamis are not considered a significant hazard for the site.

**6.3.2.5 Radon Gas:** The California Department of Public Health Services (CDPH) on the California Geologic Survey (CGS) website was also reviewed for California Indoor Radon Levels Sorted by Zip Code, last updated February 2017. Based on the zip code for the site (92501), the CGS website indicates only 1 of the 12 measurements were 4 picocuries per liter (approximately 8.3 percent) or higher.

In addition, based on our review of Special Report 247 entitled, "California Indoor Radon Potential," dated 2022, prepared by CGS, based on the zip code for the site (92501), the special report indicates non of the two (2) measurements were 4 picocuries per liter (approximately 8.3 percent) or higher.

Based on the information reviewed, the potential for indoor radon gas is considered low.

**6.3.2.6 Naturally Occurring Asbestos:** Asbestos occurs in soil and rock naturally in certain geologic settings in California. It has been documented that inhalation of asbestos fibers may cause negative health effects. Most commonly, naturally occurring asbestos is associated with serpentinite and partially serpentized ultramafic rocks. Ultramafic rocks are scattered throughout much of the Sierra Nevada mountain and the Coast Range regions. Open-File Report

2000-19, titled *A General Location Guide for Ultramafic Rocks in California - Areas More Likely to Contain Naturally Occurring Asbestos,* prepared by the State of California Department of Conservation, Division of Mines and Geology, dated August, 2000, does not identify ultramafic rock in the site vicinity. The referenced report indicates the closest surficial occurrence of ultramafic rocks are located in Santa Barbara County, about 140 miles northwest of the site.

Ultramafic rocks were not encountered at the site during our investigation and are not common to the geologic environment of the site. Based on the cited literature and our site observations, it is our opinion that the potential to encounter near surface naturally occurring asbestos containing rock at the site is very low.

**6.3.2.7 Hydrocollapse:** Collapsible soils typically consist of loose, dry, low-density soils that exhibit significant consolidation with the addition of water. Collapsible soils can be found in many areas of the southwestern United States, specifically in areas of young alluvial fans, debris flow sediments, and wind-blown deposits. Collapsible soils can possess significant strength in a dry state; however, the saturation of collapsible soils can break the clay/cement bonds holding the soil grains together and subsequent collapse of the soils can cause damage to overlying structures.

The results of laboratory consolidation tests conducted as part of this investigation indicated that the soils tested had insignificant collapse when wetted under a load of 2,000 pounds per square foot. Considering the soils below the site are mapped as older alluvium, the in-place dry densities of the near surface soils encountered (average in-place dry density of 109.6 pounds per cubic foot), and the results of the consolidation tests discussed above, collapsible soils are not anticipated to be a concern for the project.

**6.3.2.8 Regional Subsidence:** Based on the interactive USGS mapping "Areas of Subsidence" (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html), the subject project site is not located within a mapped area of subsidence due to groundwater pumping, peat loss, or oil extraction. Thus, regional subsidence is not considered a concern for the subject project.

**6.3.2.9 Clays and Cyclic Softening:** Saturated, low plasticity clays were not encountered during this investigation such as below a static groundwater condition. Accordingly, cycling softening of clays is not considered an impact to the project.

**6.4 SEISMIC HAZARDS**: The potential for fault hazard ground rupture, seismic groundshaking and seismic coefficients/earthquake spectral response acceleration design values, seiche, tsunamis and liquefaction and seismic settlement are described in the following subsections.

**6.4.1 Active Faulting and Surface Fault Rupture**: Earthquakes are caused by the sudden displacement of earth along faults with a consequent release of stored strain energy. The fault slippage can often extend to the ground surface where it manifests in abrupt relative ground displacement across a rupture surface. Damage resulting directly from fault rupture ground displacement occurs only where structures are located astride a rupture surface with relative displacement.

The project site is not located in an Alquist-Priolo Earthquake Fault Zone or a known local earthquake fault rupture hazard zone. The locations of active faults were evaluated using the 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6, prepared by Jennings and Bryant. A map depicting the major active faults in the vicinity of the site is included on Drawing No. 4 in Appendix A of this report.

The Fault Activity Map of California indicates the active fault with surface rupture nearest to the site is the San Jacinto fault (San Bernardino segment), which is located about 7¼ miles northeast of the site. Considering the distance to the nearest known active fault with surface rupture, the potential for surface fault rupture due to a known active fault is considered low.

**6.4.2 Groundshaking**: For any given earthquake, the rock in the immediate vicinity will respond with a certain maximum acceleration and with a predominant period that depends on the nature of the rock and the source mechanism. Away from the focus of the earthquake, the ground motions begin to attenuate. The way in which the earthquake wave is altered depends to a great degree on source characteristics and to a lesser degree on the travel path.

A summary of our review of historic seismic activity relative to the site is included below.

**6.4.2.1 Historic Seismic Activity**: The general site region has experienced recurring seismic activity. Based on historical earthquake data obtained from the U.S. Geological Survey's earthquake database system, approximately 1,018 historical earthquakes with magnitude 4.0 or greater have been recorded from 1900 through August 18, 2025, within about 100 miles of the site. A map showing the location of the project site with relation to the approximate historical earthquake epicenter locations and magnitude category is presented on Drawing No. 5 in Appendix A of this report.

The nearest recent earthquake event with registered intensity was identified as a 4.6 magnitude earthquake which occurred on May 22, 1907, approximately 3.3 miles northwest of the site. This event produced an estimated peak ground acceleration at the subject site of 0.188g. The largest magnitude earthquake identified in the 100 mile search radius was the 7.3 magnitude Landers Earthquake which occurred on June 28, 1992, approximately 56 miles northeast of the site. This event is estimated to have caused a peak ground acceleration at the subject site of 0.118g. The highest estimated historic ground motion at the subject site was 0.261g from the 6.25 magnitude San Jacinto earthquake that occurred on July 23, 1923, about 7.3 miles northeast of the site. This analysis was performed using the USGS on-line United States Geological Survey, Search Earthquake Catalog, (https://earthquake.usgs.gov/earthquakes/search/). Ground motion estimates for the subject site was obtained based on EQSEARCH: a computer program by Blake, T.F., 2000a, for the estimation of peak acceleration from digitized California historical earthquake catalogs.

**6.4.2.2 Design Seismic Ground Motion Parameters and Site Class:**
Seismic coefficients and spectral response acceleration values were developed for design of the building in accordance with the 2025 California Building Code (CBC). The CBC methodology for determining design ground motion values is based on U.S Geological Survey seismic hazard maps, which incorporate both probabilistic and deterministic seismic ground motion.

A table providing the seismic coefficient and earthquake spectral response acceleration values that will need to be applied for the project site is included in the "Shallow Spread Foundations" recommendations section of this report. As indicated in this report, different site classes were applied in accordance with Section 20.1 of ASCE 7-22 based on the Standard Penetration Test (SPT) N-values from the borings.

**6.4.3 Liquefaction and Seismic Settlement:** Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction.

Based upon the Riverside County Liquefaction Susceptibility mapping (https://koordinates.com/layer/96846-riverside-county-ca-liquefaction/), the site is mapped in a low liquefaction hazard area. The liquefaction hazards have not been mapped by CGS in the site area.

Due to the depth of groundwater being greater than 50 feet below site grade, the potential for liquefaction to occur is low. However, there is a potential for dry seismic settlement to occur above the design groundwater depth. Accordingly, dry seismic settlement estimates were estimated.

Dry seismic settlement analysis was conducted using the computer program LIQUEFYPRO by Civiltech, and the methodology of Tokimatsu and Seed, 1987. The analysis utilized soil parameters, such as unit weight, N-value, and fines content for the soil conditions encountered throughout the depths explored (see test boring logs, Appendix B). A hammer energy correction factor of 1.5 was applied to the field N-value results considering previous testing of the energy delivery for the auto hammer.

Based on the 2025 CBC, a Site Class D was used to estimate the seismic ground acceleration for estimation of seismic settlement potential. Accordingly, a Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects ($PGA_M$) of 0.62g was applied based on the Ground Motion Parameter Calculator provided by the Structural Engineers Association of California website (https://seismicmaps.org/). An earthquake magnitude of 8.2 was applied in the analysis based on the highest earthquake magnitude determined from probabilistic analysis (hazard deaggregation analysis for the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/), and deterministic analysis using the Building Seismic Safety Council 2014 (BSSC2014) Scenario Catalog from the USGS website for the Earthquake Hazards Program (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/).

The worst-case condition was identified using the SPT N-value profile from Boring B-2, which indicated about 4¾ inches of dry seismic settlement. The majority of this settlement (about 3½ inches) occurs in the soft to medium stiff sandy silt fill soils in the upper ~15 feet BSG. However, Boring B-2 was advanced in the asphalt pavement area at an elevation of about 11-feet above the basement elevation of the proposed building. Thus, the seismic settlement estimated from the Boring B-2 profile represents the conditions in the proposed parking lot. Analysis using the SPT N-value data collected from Borings B-3 and B-4 indicated about 2¼ inches of dry seismic settlement. The majority of this settlement (about 1¼ inches) occurs in the soft sandy silt and loose silty sand native soils in the upper ~10 feet BSG. Refer to Section 7.4 of this report regarding recommended mitigation measures due to the magnitude of estimated dry seismic settlement. Graphical results of the seismic settlement analyses are included in Appendix E.

## 7.0 GEOTECHNICAL ENGINEERING EVALUATIONS

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions determined from the field exploration and laboratory testing program and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report

**7.1 Existing Site Features and Surface Conditions:** At the time of our August 2025 field investigation, the project site was developed with the active City of Riverside Police Department Headquarters. Improvements include the existing Police Department Building which includes a basement level, an at-grade asphalt paved parking area, an asphalt concrete paved alleyway and a site retaining wall. The proposed building is to be located within the footprint of the existing building. Various underground utilities were marked in the area of the borings and proposed improvements.

As part of the site preparation, existing surface improvements, foundations and subsurface structures, utilities and other improvements will need to be removed. In addition, all soils which are disturbed from the demolition activity will need to be excavated in order to expose undisturbed soils. All excavations resulting from the demolition should be backfilled as engineered fill. It should be noted that the depth and type of the existing building foundation is unknown. If the existing building is supported on pile foundations, the tops of the pile foundations will need to be removed to a sufficient depth in order to prepare the subgrade soils for the overlying improvements. In addition, if pile foundations are present, could create conflicts with planned improvements.

The western and northern portion of the site included landscaped areas with mature trees. Thus, vegetation, root systems and organics should be removed in these areas as part of the subgrade preparation.

**7.2 Expansive Soils:** One of the potential geotechnical hazards evaluated at this site is the expansion potential of the near surface soils. Over time, expansive soils will experience cyclic drying and wetting as the dry and wet seasons pass. Expansive soils experience volumetric changes (shrink/swell) as the moisture content of the clayey soils fluctuate. These shrink/swell cycles can impact foundations and lightly loaded slabs-on-grade when not designed for the anticipated expansive soil pressures. Expansive soils cause more damage to structures, particularly light buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). Expansion potential may not manifest itself until months or years after construction. The potential for damage to slabs-on-grade and foundations supported on expansive soils can be reduced by placing non-expansive fill underlying foundations and slabs-on-grade.

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on a representative sample of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The expansion index testing was performed in accordance with ASTM D4829 and the results are included in Appendix C of this report. The results of the expansion index testing indicated that the near surface soils have a very low expansion potential (EI=0). In addition, laboratory plasticity index testing the soils possess a low plasticity. Thus, the potential for expansive soils impacts are considered low. Although the potential for expansive soils movement is low, for constructability, this report recommends that the equipment slab-on-grade be underlain by at least 6 inches of aggregate base.

**7.3 Static Settlement and Bearing Capacity of Shallow Foundations:** The potential for excessive total and differential static settlement of foundations and slabs-on-grade is a geotechnical concern that was evaluated for this project. The increases in effective stress to underlying soils which can occur from new foundations and structures, placement of fill, withdrawal of groundwater, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structures and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

In order to reduce the potential for excessive dry seismic settlement, excavation of the subsurface soils to a depth of at least 7 feet below the planned basement elevation is recommended in this report. As a result of the excavation and compaction of engineered fill, foundations will be supported on engineered fill. This recommended over-excavation will also reduce the total and differential static settlement of foundations to 1-inch total and ½ inch differential in 30 feet.

Based on our analysis, the subgrade preparation will provide a net allowable soil bearing pressure of 4,000 pounds per square foot for design of spread foundations. The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and weight of the footing may be neglected. The net allowable soil bearing pressure presented was selected using the Terzaghi bearing capacity equations for foundations considering a minimum factor of safety of 3.0 and based on the anticipated static settlements noted in this report.

A structural engineer experienced in foundation and slab-on-grade design should determine the thickness, reinforcement, design details and concrete specifications for the proposed building foundations and slabs-on-grade based on the anticipated settlements estimated in this report.

**7.4 Mitigation of Dry Seismic Settlement for Building Support:** As indicated in Section 6.4.3 of this report, analysis of the SPT N-values obtained from Borings B-3 and B-4, indicated a potential for excessive dry sand settlement. Based on the potential impacts of seismic dry settlement, this report provides recommendations for over-excavation to a depth of 7-feet below the basement elevation to reduce the estimated total seismic settlement to 1½ inches. If the estimated dry seismic settlement needs to be further reduced, additional excavation and compaction could be conducted; however, considering the potential constraints to deep excavations (i.e, engineered shoring) as discussed in Section 7.5, as an alternative to excavation and compaction as a means to mitigate the dry seismic settlement, the foundations could be supported on deep ground modification elements. Due to the relatively close proximity of existing building structures to the subject site, deep ground improvement methods that result in excessive ground vibrations are not recommended. From a geotechnical engineering standpoint, deep ground modification such as rigid inclusions could be used to mitigate the effects of dry sand settlement as well as to provide structural support of the proposed building and floor slab loads for the structure. These ground modification methods are typically designed and constructed by specialty ground modification contractors. In order to use the deep ground modification elements for support of the structure and floor slab loads, the foundation design will need to be coordinated with the design of the deep ground modification system. The building foundation system and slab on grade may need to be stiffened in order to span between ground modification elements. Deep ground modification recommendations are included in Section 9.4 of this report.

Other approaches such as a mat foundation which may allow for a higher magnitude of seismic settlement and therefore less excavation may also be an option. Alternatively, a deep foundation system could also be considered, but would likely be more costly than the deep ground modification approach. If a deep foundation system is considered, additional geotechnical investigation should be conducted to provide deeper subsurface soil data for assessment of potential gravel and cobble in order to reduce potential change order claims.

**7.5 Temporary Excavations and Foundation Monitoring**: As indicated in this report, one of the options for support of the proposed foundations would be to use a spread foundation system supported on engineered fill achieved by over-excavation to a depth of about 7 feet below the proposed basement. However, existing adjacent improvements scheduled to remain will need to be protected during excavation. Excavations adjacent to existing slabs and foundations could undermine support and damage the existing adjacent improvements. Unsupported excavations should not be conducted below a 1.5 horizontal to 1 vertical plane from the top of existing foundations, or below utilities. Thus, it is anticipated that temporary excavations extending to a depth of 7 feet below the basement level would require much more engineered shoring as compared with the depth of excavation that would be required for support of the structure on deep ground modification elements. As a minimum, due to the exi engineered shoring would be anticipated on the side of the excavation adjacent to Orange Street as well as the Pedestrian Mall to the north.

In addition, appropriate construction methods and equipment, such as low vibration equipment, should be used for earthwork activities adjacent to the existing improvements so as not to damage existing improvements which are to remain. Temporary excavations should be monitored for movement during construction.

**7.6 Asphalt Concrete (AC) Pavements:** Recommendations for new asphalt concrete pavement structural sections are presented in the "Recommendations" section of this report. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highway Design Manual. The analysis was based on traffic index values ranging from 5.0 to 9.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

It should be noted that if pavements are constructed prior to the construction of the structures, the additional construction truck traffic should be considered in the selection of the traffic index value. If more frequent or heavier traffic is anticipated and higher Traffic Index values are needed, Moore Twining should be contacted to provide additional pavement section designs.

Based on the results of the testing included in this report, an R-value of 45 was used to provide the pavement section thickness recommendations.

**7.7 Portland Cement Concrete (PCC) Pavements:** Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the characteristics of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL) from the loading of heavy trucks called a traffic index (T.I).

The recommendations provided in this report for PCC pavement thicknesses are based on truck loading and the design procedures using Streetpave at www.pavementdesigner.org.

The pavement sections were prepared based on traffic loading which is equivalent to traffic indexes ranging from about 6 to 9. A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 6-inch layer of aggregate base material (minimum R-value of 78), of 180 psi/in at the top of the aggregate base was used for pavement design.

**7.8 Soil Corrosion:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the corrosion severity ratings listed in the Table No. 5 below.

**Table No. 5**
**Soil Resistivity and Corrosion Potential Ratings**

| Soil Resistivity (ohm cm) | Corrosion Potential Rating |
|---|---|
| >20,000 | Essentially non-corrosive |
| 10,000 - 20,000 | Mildly corrosive |
| 5,000 - 10,000 | Moderately corrosive |
| 3,000 - 5,000 | Corrosive |
| 1,000 - 3,000 | Highly corrosive |
| <1,000 | Extremely corrosive |

The results of soil sample analyses indicate that the near-surface soils exhibit a “moderately corrosive” corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the “moderately corrosive” corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion

protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

**7.9 Sulfate Attack of Concrete:** Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil, groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with structural concrete is to perform laboratory testing to determine the concentrations of sulfates present in the soils. The test results are then compared with the exposure classes in Table 19.3.1.1 of ACI 318 to provide guidelines for concrete exposed to soils containing sulfates. It should be noted that other exposure conditions such as the presence of seawater, groundwater with elevated concentrations of dissolved sulfates, or materials other than soils can result in sulfate exposure categories to concrete that are higher than the concentrations of sulfate in soil. The design engineer will need to determine whether other potential sources of sulfate exposure need to be considered other than exposure to sulfates in soil. The sulfate exposure classes for soils from Table 19.3.1.1 are summarized in the below table.

**Table No. 6**
**ACI Exposure Categories for Water Soluble Sulfate in Soils**

| Sulfate Exposure Class (per ACI 318) | Water Soluble Sulfate in Soil (Percent by Mass) |
|---|---|
| S0 | Less than 0.10 Percent |
| S1 | 0.10 to Less than 0.20 Percent |
| S2 | 0.20 to Less than or Equal to 2.00 Percent |
| S3 | Greater than 2.00 Percent |

Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The laboratory test results for sulfates are included in Appendix C of this

report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

## 8.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigation, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following conclusions are presented.

8.1 The site is considered suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and observation of foundations and earthwork activities by Moore Twining are integral to this conclusion.

8.2 The Geologic Map of the Riverside East 7.5-Minute Quadrangle, Riverside County, California, prepared by Douglas M. Morton and Brett Cox, dated 2001, indicates the area of the proposed building is underlain by late to middle Pleistocene Epoch Quaternary sediments comprised of old alluvial fan deposits (Qof).

8.3 Based on the subsurface soils encountered in borings B-2, B-3 and B-4, the planned basement area is anticipated to be underlain by about 2 feet of fill soils over native soils which generally consisted soft sandy silt and loose silty sands to a depth of about 10 feet below the proposed basement floor elevation, which were further underlain by medium dense to dense silty sand, very stiff sandy silt, and stiff to very stiff sandy lean clay to the maximum depth explored, about 47 feet below the basement floor elevation in Boring B-2.

8.4 The soils encountered in Borings B-2 and B-5 below the asphalt pavement in the existing parking lot generally consisted of soft to medium stiff sandy silt and loose silty sand, overlying native medium dense clayey sand, and medium dense to dense silty sand to the maximum depth explored, about 57 feet below the top of the existing pavement.

8.5 Groundwater was not encountered at the time of the field investigation (August 2025) within the borings advanced to a maximum depth of 57 feet BSG. Based on the groundwater depths from wells near the site, a high groundwater depth of 70 feet (exceeding 50 feet BSG) was used in liquefaction/dry seismic analysis.

8.6 Due to the depth of groundwater, the potential for liquefaction to occur is low. However, dry seismic settlement of up to about 2¼ inches was estimated below the proposed basement level of the structure. In order to reduce the dry seismic settlement to 1½ inches, an excavation depth and compaction of engineered fill to about 7 feet below the basement would be required. Due to the potential constraints to excavation to a depth of 7 feet below the basement, deep ground modification such as rigid inclusions could be used as a cost effective option to support a spread foundation system. Mat foundations or deep foundation systems could also be considered.

8.7 The results of soil sample analyses indicate that the near-surface soils exhibit a "moderately corrosive" corrosion potential to buried metal objects.

8.8 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318, the sulfate concentration from chemical testing of a soil sample fall in the S0 classification (less than 0.10 percent by weight) for concrete.

8.9 The subject police department building and development is located in flood Zone X, which is defined as "area of minimal flood hazard." The site is located within a dam inundation area. Refer to the discussion in Section 6.3.2.3 of this report.

8.10 The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Based on our review of the 2010 Fault Activity Map of California, the nearest known active fault with surface rupture is the San Jacinto Fault (San Bernardino segment), which located nearly 7¼ miles northeast of the site. Accordingly, the potential for surface fault rupture at the site is considered low.

## 9.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings and conclusions should be considered. The recommended design consultation and construction monitoring by Moore Twining are integral to the proper application of the recommendations.

Where the requirements of a governing agency, utility agency or product manufacturer differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

### 9.1 General

9.1.1 If the planned construction is different from that described in the Anticipated Construction section of this report, the recommendations in this report may not be appropriate. Moore Twining should be notified and requested to provide supplemental recommendations for the proposed construction if changes are planned.

9.1.2 Requests for the original plans for design and construction of the existing building were requested, but were not available at the time of the preparation of this investigation. Consequently, the existing foundation type and geometry are not known. If the plans are not available, a supplemental investigation could be conducted to expose the existing footings to determine whether shallow or deep foundation systems are present.

9.1.3 A preconstruction meeting including, as a minimum, the City of Riverside, the architect, earthwork contractor, and Moore Twining, should be scheduled by the general contractor at least one week prior to the start of construction. The purpose of the meeting should be to discuss project requirements and scheduling.

9.1.4 If wet soil or unstable conditions are experienced, methods such as aeration, mixing wet soils with drier soils, chemical (i.e., cement) treatment of the soil, or the use of aggregate base and a geotextile stabilization fabric such as Mirafi 600X or equivalent, may be required to achieve a stable condition.

9.1.5 It is recommended a preconstruction survey be conducted by a qualified consultant working for the City of Riverside to document the conditions of adjacent improvements prior to the start of construction. The preconstruction survey would serve as the baseline condition before construction and could be used to evaluate potential damage from construction work such as ground vibration impacts, etc.

9.1.6 Existing foundations within influence of the work should be surveyed prior to construction, protected, and monitored by the Contractor for movement by level surveys during construction.

9.1.7 Appropriate construction methods and equipment, such as low vibration equipment, should be used adjacent to the existing improvements so as not to damage existing improvements which are to remain.

9.1.8 Contractor(s) bidding on this project should determine if the data are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor should identify to the City in writing what information is needed for an accurate bid.

9.1.9 All wells encountered (if any) should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner upon completion.

9.1.10 A demolition plan should be prepared to identify existing improvements such as landscaped islands with and without trees, underground utilities, concrete walkways, etc. that are to be demolished and/or removed as part of the project.

## 9.2 Site Grading and Drainage

9.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades which are to be modified should be sloped a minimum of five (5) percent for a distance of at least ten (10) feet away from the structures, or as necessary to preclude ponding of water adjacent to foundations, whichever is more stringent. Adjacent exterior grades which are paved should be sloped at least 2 percent away from the foundations.

9.2.2 Landscaping after construction (if any) should direct rainfall away from foundations and to prevent ponding of water and erosion.

9.2.3 It is not recommended to place landscape or planted areas directly adjacent to the improvements. Trees should be setback from the proposed structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip line diameter of 30 feet, the tree should be at least 15 feet away (radius) from the proposed foundations.

9.2.4 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with minimal water requirements are recommended.

### 9.3 Storm Water Retention and Infiltration System(s)

9.3.1 **Due to the planned basement for the structure and the retaining wall on the south side of the parking lot, stormwater infiltration systems are generally not recommended at this site due to the potential for subsurface seepage to migrate behind earth retaining structures.** In the event subsurface storm water infiltration systems, bioswales or similar designs are required, the proposed locations and details of these features should be provided to Moore Twining for review and comment. On-site stormwater infiltration features should be setback as far as possible from the proposed structures and retaining walls (minimum 20 feet). Pavements, curbs, utilities, etc. that are supported over, or are located adjacent to storm water infiltration systems should be expected to require added maintenance and repairs due to settlement of the pavements, etc.

9.3.2 The lowest infiltration rate estimated from the two (2) percolation tests indicated an estimated infiltration rate of 0.8 inches per hour at a depth of about 10⅓ feet BSG in the existing parking lot area. The test data is only applicable for the depth and location where the test was conducted. Since field percolation testing is a small-scale test method and does not take into account the long term effects of subgrade saturation, silt/fines accumulation, or mechanical densification of the soils as a result of the construction process, etc., a safety factor should be applied to the recommended infiltration rate for design of the basin. A safety factor ranging from 3 to 10 is generally recommended for soils with high permeability such as the site soils; however, the final safety factor should also meet the requirements of the governing agency and should consider such factors as the sediment load of the stormwater, whether pretreatment of stormwater is planned, the consequences of failure, the degree of maintenance that can be relied upon and the uncertainties in the estimated inflow volume.

9.3.3 If an increased risk of settlement below site improvements such as pavements is not acceptable, stormwater infiltration systems should not be used below or near settlement sensitive features.

9.3.4 Although not anticipated, stormwater infiltration design should consider appropriate safety factors determined by the designer to account for the consequences of exceeding the system capacity, regulatory agency requirements, uncertainty in the inflow rate calculations, and such factors as reduction in infiltration rate due to siltation.

9.3.5 The bottom of excavations for on-site storm water infiltration system should be excavated to a neat, undisturbed condition prior to placement of fill. The contractor shall use such equipment as necessary to achieve a smooth, undisturbed surface at the bottom of the excavation. Compactive effort should not be applied to the soils at the bottom of the infiltration system.

9.3.6 If an open graded material such as crushed rock is required around the storm water pipes, a crushed rock may be used as bedding, haunching and to 12 inches above the pipe, provided these materials are fully encapsulated in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Open-graded rock, such as gravel, should be placed in thin horizontal lifts (6 to 8 inch lift thickness) and compacted with vibratory equipment. A sufficient space should be provided beyond the storm drain pipes to allow for proper placement and compaction of the haunching and initial fill materials. Native on-site soils or import soils,

may be used for the final fill from 12 inches above the pipe to final design grades. Where infiltration systems are buried below pavements, a layer of Mirafi 600X should be placed below the aggregate base layer over the top of the entire storm water infiltration system and extending a minimum horizontal distance of 10 feet beyond the outside edge of the storm water infiltration system.

9.3.7 The Contractor shall schedule Moore Twining to observe the bottom of the excavation for the subsurface storm water infiltration system to observe the exposed soil conditions at the bottom of the excavation for consistency with the conditions anticipated based on this investigation.

9.3.8 After installation, the stormwater infiltration system should be flooded with a six (6) inch height of water at least two (2) times to induce settlement prior to construction of the pavements. The objective is for the soils below the proposed infiltration system to receive sufficient water that is evenly distributed to saturate the entire bottom of the excavation prior to construction of the finished surface improvements. The contractor should conduct the flooding under the observation of Moore Twining. These requirements should be specified on the plans.

9.3.9 The Contractor should be responsible for arranging for the manufacturer of the storm drain pipes to certify in writing that the pipes have been installed in accordance with their standards. The Contractor is responsible to have the manufacturer conduct sufficient site visits and have the manufacturer to verify that the pipes were installed in accordance with the minimum requirements of the manufacturer. These requirements should be specified on the plans.

## 9.4 Deep Ground Modification for Dry Seismic Settlement Mitigation

9.4.1 The following general recommendations were prepared for design and construction of deep ground modification to mitigate dry seismic settlement and to provide direct support of the foundation and floor slab loads for the structure. On a preliminary basis, this report provides recommendations regarding the use of rigid inclusions. If other methods of deep ground modification are proposed, the proposed method should be provided to Moore Twining for review and consideration. Ground modification methods that induce ground vibration should not be allowed due to the proximity of adjacent improvements.

9.4.2 The design-build ground modification contractor shall provide a deep ground modification design submittal a minimum of 10 days prior to start of construction to the owner, architect and Moore Twining for review. The submittal shall include an independent geotechnical engineering analysis using the data provided in this report, and any additional geotechnical data deemed necessary by the design build engineer, indicating that the proposed ground modification will achieve the required mitigation of dry seismic settlement, support capacity for the proposed foundations, static settlements for design, etc. This assessment should be prepared and stamped by a professional engineer registered in the State of California with a minimum of 5 years of experience in deep ground modification design. The submittal shall include the estimated static and seismic settlements, the deep ground modification procedures, a plan showing the deep ground modification design layout in relation to the proposed improvements (reference the project drawings for the minimum limits of ground modification) including utilities, how the work will be coordinated to prevent damage to both existing and

proposed utilities, the design/specifications for all required aggregate cushion, load transfer platform materials, etc., the requirements for preparing/restoring the subgrade, the procedures that will be used to verify the performance requirements of the deep ground modification design, and the quality control methods to be employed by the contractor during the work.

9.4.3 Ground modification along with over-excavation and compaction of the subgrade soils will be required to reduce the estimated total and differential static settlement to a maximum of 1 inch total and ½ inch differential and to reduce the total seismic settlement to a maximum of ¾ inch and ½ inch differential in 30 feet, or other limits as specified by the project structural engineer based on the details of the proposed structure. The deep ground modification activity shall be conducted in accordance with this report, the plans, the specifications, and the City of Riverside requirements, whichever is the most stringent. In the event a select fill material is required for load transfer for support of the slab on grade, the load transfer platform design requirements should be detailed by the design build contractor.

9.4.4 The deep ground modification will need to extend to an appropriate elevation such that the deep ground modification elements will be suitable for direct support of the proposed foundations as well as to address the effects of dry seismic settlement for the building foundations and slabs on grade.

9.4.5 Due to soil disturbance caused by the installation process, upon successful completion of the ground modification, the existing surface soils should be over-excavated in the treatment area as recommended in the Site Preparation section of this report to establish a compacted subgrade condition.

9.4.6 The minimum extent of the deep ground modification treatment should be specified on the project plans. In general, deep ground modification will be required to mitigate dry seismic settlement and to provide direct support of the building foundations, and interior and exterior slabs on grade. Other improvements which are sensitive to functionality of the project or are otherwise sensitive to differential movement should also be supported on deep ground modification.

9.4.7 The sequencing of the work, bulking of soils resulting from the work, construction of a stable, working platform and any requirements of the site preparation necessary to provide a level work area and equipment access, etc. should be coordinated with the general and earthwork contractors and included in the scope of work.

9.4.8 In addition to dry seismic settlement mitigation, the deep ground modification elements will be required to provide direct support of all new foundations, the interior slab on grade and all walkways adjacent to the building. The design shall achieve a minimum allowable bearing capacity of 4,000 pounds per square foot. The final extent of the treatment areas shall be determined by the design-build contractor in accordance with the recommendations of this report. It is recommended the area required for ground modification include the building and all foundation areas, and any other area deemed to be sensitive to seismic settlement as specified by the design professional for post-earthquake functionality, etc.

9.4.9 The deep ground modification program should include post ground modification testing to verify the soil densification and composite ground modification element/section stiffness that was assumed for design is actually achieved. The methods of verification of the post ground modification condition (i.e., load/modulus testing, cone penetration tests, soil borings, etc.) shall be developed by the design-build engineer. The design-build contractor shall submit a final report signed and stamped by the design build engineer at the conclusion of the work including an analysis of the post ground modification geotechnical data and analysis to validate that the ground modification achieved the specified performance criteria. At the completion of work, a final report shall be prepared by the design build engineer to conclude that the deep ground modification work is suitable for support of the planned improvements and provide any supplemental recommendations that are deemed necessary by the design build engineer.

9.4.10 At the conclusion of the installation of deep ground modification work, the Contractor shall prepare an as-built plan showing the locations, diameters, depths, etc. for all the deep ground modification elements.

9.4.11 The design build contractor will be responsible for quality control of the installation of the deep ground modification program. Moore Twining should be contracted by the owner to observe the contractor's load tests, observe the ground modification installation, and conduct sampling and testing of the materials for quality assurance purposes.

## 9.5 Site Demolition and Preparation

9.5.1 All surface topsoil, vegetation and organics identified should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent by weight should be removed and not used as engineered fill. Stripping should be reviewed by Moore Twining at the time of construction and should extend laterally a minimum of 5 feet outside areas of planned excavation and outside the limits of the new improvements (i.e., proposed building and any attached concrete walkways). Deeper stripping may be required in localized areas. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner.

9.5.2 Excavation of tree roots should be conducted as necessary to remove all root systems greater than about ¼ inch in diameter from the areas of proposed improvements. Soils containing organic matter such as root clumps, roots exceeding ¼ inch in diameter, with organic contents above 3 percent should be completely removed and not used as engineered fill. After removal of the roots and organics, the bottom of the excavation should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations. Areas of depressions should be excavated and backfilled with engineered fill under the observation of Moore Twining.

9.5.3 As part of the site preparation, existing surface and subsurface improvements such as foundations, slabs on grade, basement, pavements, curb and gutter, stormwater collection structures, hardscaping, underground utilities, buried structures (including the reported tunnel entrance to be abandoned), and associated backfill will need to be removed. As part of the removal, all trench backfill and basement retaining wall backfill should be removed to expose undisturbed, native soils. During removal of underground utilities and subsurface structures, the backfill soils should also be removed. The utility pipe or conduits to be abandoned should be completely removed and disposed of off-site. Care should be taken to over-excavate all soils which are disturbed from the demolition activities prior to backfilling the excavations with engineered fill. All excavations should be backfilled with engineered fill in accordance with the recommendations of this report, under the observation and testing of Moore Twining. In addition, all undocumented fill soils should be removed from the areas of the proposed improvements. The resulting excavations should be cleaned of all backfill and organic material, the exposed undisturbed native soils should then be scarified to a depth of 8 inches, moisture conditioned, then compacted as engineered fill, and the excavation backfilled with engineered fill.

9.5.4 If existing pile foundations are encountered, the upper portions of existing pile foundations should be removed to a minimum depth of about 8 feet below finished grade in areas where new buildings are planned, or to a minimum depth of 5 feet below the bottom of any foundation, whichever is greater. Removals of the piles in planned paved areas may be reduced to a minimum depth of 5 feet below finished grade. Deeper removal may be required for installation of underground utilities or other purposes. Pile removal should include cutting of steel reinforcement flush with the concrete, which is anticipated to require hand tools for cutting. The bottoms of the excavations should be scarified to a depth of 8 inches, moisture conditioned, and compacted to at least 92 percent relative compaction (ASTM D15557). The excavations should be backfilled in accordance with the recommendations of this report. Contractors should also anticipate that compaction of soils above the cut-off piles may require the use of hand equipment directly above and to some distance above the top of the pile in order to reduce the potential for damage to larger grading equipment.

9.5.5 Following stripping and removal of existing surface and subsurface improvements, two options are provided to mitigate the potential for static and seismic settlement.

Option 1 - over-excavate and place engineered fill to support foundations and reduce seismic settlement: The building pad area should be over-excavated to a depth of 7 feet below the basement finish floor elevation, to at least 1 foot below subsurface structures to be removed, to at least 2 feet below the bottom of foundations, and the depth required to remove all soils which are disturbed from demolition work, whichever is greater.

Option 2 - use of deep ground modification to mitigate seismic settlement and for direct support spread foundations: The over-excavation should extend to the depth required for placement of load transfer platform materials, as required for the deep ground modification design, to at least 1 foot below subsurface structures to be removed, to the depth required to remove all soils which are disturbed from demolition work, and to the depth required to excavate all undocumented fill, whichever is greater.

As a minimum, the horizontal limit of the excavation for preparation of the building pad subgrade should include the entire footprint of the building, all foundations, adjacent walkways, to a minimum of 5 feet horizontally beyond the foundations, and to the horizontal extent of the deep ground modification limits, whichever is greater. Upon approval of the excavation by Moore Twining based on visual observations and the survey data provided by the contractor, the bottom of the excavation should be scarified to a depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to a minimum of 92 percent relative compaction prior to placement of fill soils. It should be noted that the above recommendations are not intended for preparation of a working surface to support any specific construction equipment such as may be used for ground modification work. The contractor should determine the preparation and fill material requirements for support of construction equipment. The materials proposed for fill should be submitted to Moore Twining for review and approval prior to use.

If optioned, due to the soil disturbance caused by the deep ground modification installation process, after the deep ground modification work, the subgrade soils should be re-prepared by compacting the soils disturbed from the deep ground modification work. The recompaction work and final documentation/verification of the compaction of the building pad area should be verified by Moore Twining by observation and testing.

9.5.6 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from Moore Twining or excavating for foundations.

9.5.7 For areas to receive fill soils outside the building pad preparation limits, after site stripping, removal of root systems and removal of existing surface and subsurface improvements, the areas should be over-excavated to a minimum of 6 inches below preconstruction site grade, to a minimum of 6 inches below the improvements to be removed, and to the depth required to remove existing undocumented fills (if any), whichever is greater. The exposed bottom of the excavation should be scarified to a minimum of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill to a stable condition.

9.5.8 Miscellaneous lightly loaded foundations (such as retaining walls, sound walls, trash enclosures, screen walls, monument signs, etc.) should be evaluated on a case by case basis to develop supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, following stripping and removal of existing surface and subsurface improvements, miscellaneous foundations should be over-excavated to a minimum of 6 inches below the bottom of the foundations, to a minimum depth to remove undocumented fill, and to at least 12 inches below the existing improvement to be removed, whichever is greater. The zone of over-excavation should extend at least 5 feet beyond the edge of foundations on all sides. Upon approval of the over-excavation limits by Moore Twining, the soils at the bottom of the excavation should be scarified to a minimum depth of 12 inches, moisture conditioned to slightly above optimum moisture content and compacted as engineered fill to a minimum of 92 percent of the maximum dry density determined in accordance with ASTM D1557. Areas of instability should be stabilized by use of chemical soil stabilization or bridging materials such as noted in Section 9.1.4 of this report.

9.5.9 The retaining wall to be removed and replaced between the parking lot and the alleyway should be prepared by over-excavation to a minimum of 18 inches below the bottom of the foundation, to a minimum depth to remove undocumented fill and all disturbed soils, and to at least 12 inches below improvements to be removed, whichever is greater. The zone of over-excavation should extend at least 5 feet beyond the edge of foundations on all sides and sloped or shored per Sections 7.5 and 9.14 of this report. Upon approval of the over-excavation limits by Moore Twining, the soils at the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to slightly above optimum moisture content and compacted as engineered fill to a minimum of 92 percent of the maximum dry density determined in accordance with ASTM D1557. Areas of instability should be stabilized by use of chemical soil stabilization or bridging materials such as noted in Section 9.1.4 of this report.

9.5.10 Following stripping and removal of existing surface and subsurface improvements, areas of new/replacement asphalt concrete pavements (if any) should be over-excavated to the bottom of the new aggregate base section. The zone of over-excavation should extend laterally a minimum of 3 feet outside the perimeters of new pavements or up to the adjacent improvements which are to remain, whichever occurs first. The bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture-conditioned to zero (0) to three (3) percent above optimum moisture content, and compacted to a minimum of 95 percent relative compaction. The resulting excavation should be backfilled to finished grades with engineered fill. The upper 8 inches of subgrade soils below new pavements should be compacted to a minimum of 95 percent relative compaction. It should be noted that without removing the existing undocumented fill, there is a potential for additional settlement of undocumented fill. However, the existing parking lot is currently supported by the undocumented fill subgrade and was not observed to contain any significant pavement distress associated with settlement. Consequently, at the owner's discretion, the existing undocumented fill can remain below pavements.

9.5.11 After site stripping, removal of root systems and removal of existing surface and subsurface improvements, the areas to receive exterior concrete slabs on grade outside the building pad preparation limits and pavements should be over-excavated to a minimum of 12 inches below preconstruction site grade, to a minimum of 12 inches below the proposed subgrade, to a minimum of 6 inches below improvements to be removed, and to the depth required to remove undocumented fills (if any), whichever is greater. The exposed bottom of the excavation should be scarified to a minimum of 12 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill to a stable condition. The zone of over-excavation, scarification and compaction (overbuild zone) should extend laterally a minimum of 3 feet outside the perimeters of pavements/slabs located outside the building pad preparation limits, or by a horizontal distance equal to the depth of fill, whichever is greater. Areas of instability should be stabilized by use of chemical soil stabilization or bridging materials such as noted in Section 9.1.4 of this report.

9.5.12 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

9.5.13 Any wells encountered or scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Moore Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters) should have their casings removed to a depth of at least 8 feet below preconstruction site grades or finished pad grades, whichever is deeper. In parking lot or landscape areas, the casings should be removed to a depth of at least 5 feet below site grades or finished grades. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill.

9.5.14 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

9.5.15 Final grading shall produce a subgrade ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad (before aggregate base is placed) shall not depress more than one-half (½) inch under the wheels of a fully loaded water truck, or equivalent loading. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the owner.

9.5.16 The Contractor should be responsible for the disposal of concrete, asphalt concrete, soil, spoils, etc. that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable for the intended use by the receiving party. The Contractor is responsible to perform the tests, assessments, etc. necessary to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

## 9.6 Engineered Fill

9.6.1 Based on the results of our investigation, the soils anticipated to be encountered during grading of the site are predominantly very low to low expansive sandy silt, sandy lean clays, clayey sands and silty sands. The onsite soils with an expansion index of less than 35 may be used as engineered fill below the recommended aggregate base provided they are aerated/moisture conditioned to meet the moisture content recommendations included in this report, are free of organics (less than 3 percent by weight), are processed (if necessary) to limit the maximum particle size to less than 6 inches in maximum diameter, and are free of debris. It should be noted that concrete and brick debris were encountered in some of the fill soils encountered at the site. In the event soils with an expansion index of greater than 35 are encountered, these soils should not be placed as fill within the upper 2 feet of the plan subgrade elevation below concrete slabs on grade.

The compactability of the native dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.

9.6.2 Import fill soil should be non-expansive and granular in nature with the following acceptance criteria recommended.

| | |
|---|---|
| Percent Passing 3-Inch Sieve | 100 |
| Percent Passing No. 4 Sieve | 75 - 100 |
| Percent Passing No. 200 Sieve | 15 - 50 |
| Expansion Index (ASTM D4829) | Less than 20 |
| Organics | Less than 3 percent by weight |
| Sulfates | < 0.05 percent by weight |
| Min. Resistivity | > 5,000 ohm-cm |

9.6.3 Prior to importing fill, the Contractor shall submit test data that demonstrates that the proposed import complies with the recommended criteria for both geotechnical and environmental compliance. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the City of Riverside) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, analytical data specific to the source of the import material in accordance with the Department of Toxic Substances Control, "Informational Advisory, Clean Imported Fill Material," dated October 2001. The list of constituents to be tested for the fill source shall be submitted to TotalEnergies for review and approval prior to the Contractor testing the fill. Contractors should provide a minimum of 7 working days to complete the geotechnical testing. As an alternative, imported fill may consist of aggregate base. Prior to importing the aggregate base, the contractor should provide a letter certifying that the material is non-contaminated.

9.6.4 Onsite soils used as engineered fill should be placed in loose lifts approximately 8 inches thick or less, moisture-conditioned to zero (0) to three (3) above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. The upper 8 inches of subgrade soils in pavement areas should be compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.6.5 **For fill soils placed deeper than 5 feet below finished grade and the upper 12 inches of the subgrade soils in pavement areas**, onsite and imported, non-expansive granular soils used as engineered fill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to three (3) percent above optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.6.6 **For fill soils placed within 5 feet from finished grade in non-pavement areas and between a depth of 12 inches and 5 feet below finished grade in pavement areas**, onsite and imported, non-expansive granular soils used as engineered fill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the onsite soils within 12 inches below the bottom of the aggregate base should be moisture-conditioned to within optimum to at least three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

9.6.7 Utility trench backfill should be placed in 8 inch lifts or less, moisture conditioned to slightly above optimum moisture content and compacted to a minimum of 92 percent relative compaction (ASTM D1557). Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved. The upper 12 inches of subgrade soils in pavement areas should be compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

9.6.8 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at a frequency of at least:

**Table No. 7**

| **Area** | **Minimum Test Frequency** |
|---|---|
| Building Pad | 1 test per 2,500 square feet per compacted lift, but not less than two tests per lift |
| Pavement Subgrade and Mass Grading Outside Building Pad | 1 test per 5,000 square feet per compacted lift |
| Walkways, Retaining Wall Backfill and Utility Lines | 1 test per 50 feet per lift |

9.6.9 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Crushed rock should be placed in thin (less than 8 inch) lifts and densified with a minimum of three (3) passes using a vibratory compactor.

9.6.10 Aggregate base below the interior building slab on grade shall be non-recycled and comply with Class 2 aggregate base (AB) per Caltrans Standard Specifications. Aggregate base used for pavement construction should comply with Class 2 aggregate base in accordance Caltrans Standard Specifications and may include recycled materials. Aggregate base shall be compacted to a minimum relative compaction of 95 percent in accordance with ASTM D1557 standards.

## 9.7 Shallow Spread Foundations

In order to mitigate the dry seismic settlement potential described in this report, this report provides alternative recommendations for supporting the foundations and floor slab loads on either: 1) shallow foundations such as conventional spread foundations or stiffened slabs including mat foundations combined with deep ground modification, or 2) shallow foundations such as conventional spread foundations or stiffened slabs including mat foundations combined with deep (7-feet below basement elevation) over-excavation. Recommendations for deep ground modification are presented in section 9.4 of this report. This section includes recommendations for shallow foundations.

9.7.1 This report recommends either deep (7-feet) over-excavation below basement elevation or the use of a specialty deep ground modification program, to address concerns with dry seismic settlement and to provide direct support of the foundations and floor slab loads. If deep ground modification is utilized, the foundation design should be coordinated with the design of the ground modification system.

9.7.2 Spread foundations for the proposed building supported on engineered fill soils prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 4,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. If mat foundations are proposed, the design engineer should provide a bearing pressure diagram to Moore Twining for review and for use in estimating static settlement.

9.7.3 If the deep ground modification option is used for support of foundations to reduce the depth of excavation to prepare subgrade soils for support of the foundations, after it has been determined that the installation of the deep ground modification has achieved the specified performance criteria and the subgrade has been prepared in accordance with the site preparation recommendations of this report, structural loads may be supported directly on the deep ground modification elements. On a preliminary basis, spread and continuous footings supported directly on the deep ground modification elements, may be designed for a maximum net allowable soil bearing pressure of 4,000 pounds per square foot for dead-plus-live loads and the foundation and floor slab elements should be designed with sufficient stiffness to transfer all loads to the ground modification elements. It is expected that higher allowable bearing pressures would be obtainable when the deep ground modification design is conducted, subject to determination of the ground modification design engineer. The details of the foundation design will need to be coordinated with the design of the deep ground modification. This value may be increased by one-third for short duration wind or seismic loads. Final determination of the allowable settlement and bearing capacity should be determined by the design build ground modification engineer.

9.7.4 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on 1) a total static settlement of 1 inch, 2) a differential static settlement of ½ inch in 30 linear feet, 3) a total seismic settlement of 1½ inch, and 4) a differential seismic settlement of ¾ inch in 30 lineal feet. If different settlement limits are required by the structural engineer, the tolerable limits for the structure/foundations should be specified by the structural engineer.

9.7.5 Perimeter foundations should have a minimum depth of 18 inches below the bottom of the slab or 18 inches below the lowest adjacent finished exterior ground surface, whichever is greater. Interior footings for the buildings should have a minimum depth of at least 18 inches below the bottom of the slab-on-grade. All footings should have a minimum width of 18 inches, regardless of load.

9.7.6 The foundations should be continuous around the perimeter of the proposed building to reduce moisture migration beneath the structures. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

9.7.7 This investigation also did not include conducting shear wave velocity measurements for determination of the Site Class, since this scope was not authorized. If the supplemental testing is conducted, the recommendations of this report will be updated after completion of the testing. The Site Classes were applied in accordance with Section 20.1 of ASCE 7-22 based on the Standard Penetration Test (SPT) N-values from the borings. The following seismic factors were summarized for site classes CD, D and DE using online data obtained from the Ground Motion Parameter Calculator provided by the Structural Engineers Association of California website (https://seismicmaps.org/) based upon a latitude of 33.978100 degrees and a longitude of -117.374999 degrees. The structural engineer should review the values and apply the most stringent parameters in design. If a ground motion hazard analysis is required for use in design, Moore Twining should be notified and requested to conduct additional field exploration and analysis

**Table No. 8**

| Seismic Factor | ASCE 7-22 Value | | |
|---|---|---|---|
| Site Class | CD | D | DE |
| Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects ($PGA_M$) | 0.66 | 0.62 | 0.54 |
| Spectral Response At Short Period (0.2 Second), $S_s$ | 1.64 | 1.64 | 1.64 |
| Spectral Response At 1-Second Period, $S_1$ | 0.61 | 0.61 | 0.61 |
| Maximum considered earthquake spectral response acceleration for short period, $S_{MS}$ | 1.89 | 1.82 | 1.6 |
| Maximum considered earthquake spectral response acceleration at 1 second, $S_{M1}$ | 1.21 | 1.62 | 2.26 |
| Five percent damped design spectral response accelerations for short period, $S_{DS}$ | 1.26* | 1.21* | 1.06* |
| Five percent damped design spectral response accelerations at 1-second period, $S_{D1}$ | 0.81 | 1.08 | 1.51 |

* - $S_{DS}$ value determined in accordance with ASCE 7-22, Section 11.4.4. This value is not applicable to the Simplified Lateral Force Analysis Procedure of ASCE 7-22, Section 12.14.8
The above data is subject to the disclaimers listed in the website https://seismicmaps.org/

9.7.8 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement. The moisture content of the foundation subgrade soils should be maintained by regular moisture conditioning.

9.7.9 The moisture conditions of the subgrade soils for the building pad should be maintained in accordance with the recommendations for engineered fill until placement of the aggregate base and vapor retarder.

9.7.10 Structural loads for lightly loaded (less than 1.5 kips per lineal foot) miscellaneous foundations (such as screen walls for trash enclosures, etc.) may be supported on engineered fill prepared in accordance with the recommendations included in the Site Preparation section of this report. The lightly loaded foundations should extend to a minimum depth of 18 inches below the lowest adjacent grade with a minimum width of 15 inches, regardless of load. Footings for miscellaneous, lightly loaded foundations may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

9.7.11 Sight lighting and pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 300 pounds per square foot may be used to resist axial loads, provided the upper 2 feet is ignored. Lateral load resistance may be estimated using the 2025 CBC non-constrained procedure (Section 1807.3.2.1). The allowable passive resistance of the native soils may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot to a maximum of 2,750 pounds per square foot. The passive pressure may be applied over twice the pier diameter. The upper 24 inches of subgrade soils in landscaped areas should be neglected in determining the total passive resistance.

## 9.8 Frictional Coefficient and Earth Pressures

9.8.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill prepared as recommended in this report can be used to resist lateral loads. An allowable coefficient of friction of 0.30 can be used for design. In areas where slabs are underlain by a synthetic moisture vapor membrane, an allowable coefficient of friction of 0.10 can be used for design.

9.8.2 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot. The upper 6 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

9.8.3 The layers of fine grained native clay soils are not considered suitable for use as backfill of retaining walls. Backfill of these features extending within a zone defined by a 1 Horizontal to 1 Vertical plane from the back of the new wall/subsurface structure foundations to the ground surface should consist of imported granular fill meeting the recommendations included in 9.6.2 of this report. This requirement should be depicted on the project plans. The active and at-rest pressures of the granular engineered fill may be assumed to be equal to the pressures developed by fluid with a density of 45 and 65 pounds per cubic foot, respectively. These pressures assume a level ground surface, drained conditions and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations, or hydrostatic water pressure. Where undrained/hydrostatic conditions are applied, the active and at-rest pressures of the in-place soils may be assumed to be equal to the pressures developed by a fluid with a density of 90 and 100 pounds per cubic foot, respectively. These pressures assume a level ground surface and do not include surcharge effects of construction equipment or adjacent foundations.

9.8.4 The wall designer should determine if seismic increments are required. The wall designer should determine if hydrostatic conditions could impact the walls. If seismic earth pressures are required, a seismic earth pressure increment of 20 pounds per cubic foot equivalent fluid pressure may be applied to the wall, with the resultant force applied at 2/3 the height of the wall, measured from the bottom of the foundations.

9.8.5 Retaining walls may be subject to lateral loading from pressures exerted from the soils, foundations, equipment loads and vehicular traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, equipment or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by retaining walls.

9.8.6 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

## 9.9 Retaining Walls and Site Walls

This section provides preliminary recommendations for design of site retaining walls. When grading and conceptual retaining wall plans have been developed, these plans should be provided to Moore Twining for review. Based on review of the plans, the following preliminary recommendations may need to be revised. It is preliminarily anticipated that walls retaining up to about 11 feet will be required to support the grade changes between the parking and alleyway at the project site.

9.9.1 Retaining wall plans, when available, should be reviewed by Moore Twining to evaluate the actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters.

9.9.2 As a minimum, structural loads for the taller retaining wall between the parking lot and the alley should be supported on engineered fill soils prepared in accordance with the recommendations in the "Site Preparation" section of this report. Spread and continuous footings for the taller retaining wall structure with a minimum depth of 18 inches below finished grade may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. It should be noted that retaining wall structures would be subject to the effects of dry seismic settlement. If it is desired to provide more resistance to impacts from dry seismic settlement for retaining wall foundations, the foundations may be supported on deep ground modification elements.

9.9.3 As a minimum, structural loads for miscellaneous lightly loaded (less than 1.5 kips per foot) foundations (such as short retaining walls, sound walls, screen walls, monument and pylon signs, etc.) should be supported on engineered fill soils prepared in accordance with the recommendations in the "Site Preparation" section of this report for miscellaneous lightly loaded foundations. Spread and continuous footings for lightly loaded structures with a minimum depth of 18 inches below finished grade may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. It should be noted that miscellaneous lightly loaded structures would be subject to the effects of dry seismic settlement. If it is desired to provide more resistance to impacts from dry seismic settlement for miscellaneous lightly loaded foundations, the foundations may be supported on deep ground modification elements.

9.9.4 Imported, granular, fill used for backfill of retaining walls from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface should meet the following requirements:

| | |
|---|---|
| Percent Passing 3-Inch Sieve | 100 |
| Percent Passing No. 4 Sieve | 75 - 100 |
| Percent Passing No. 200 Sieve | 0 - 20 |
| Plasticity Index | Less than 5 |

9.9.5 The import fill material should be tested for conformance with the recommended material characteristics.

9.9.6 Segmented wall design (if any) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for segmented wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. The wall designer should perform sufficient observations of the wall construction to certify that the wall was constructed in accordance with the design plans and specifications.

9.9.7 As a minimum, retaining walls should be constructed with a drainage system including a perforated pipe surrounded by at least 1 cubic foot of 3/4 inch crushed rock or Class 2 permeable material per lineal foot of drain pipe. In addition, retaining walls associated with the subsurface portion of the structure should include a full height geocomposite drainage board, or a minimum 12 inch wide section of drain rock encapsulated in a geotextile fabric. If open graded materials such as crushed rock are used as drain material surrounding drain pipes, these materials should be fully encased in filter fabric such as Mirafi 140 N and vibrated in place to a non-yielding condition under the observation of the geotechnical engineer. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric. Class 2 permeable material should be compacted to 95 percent relative compaction in accordance with ASTM D1557. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The drainage system should be designed by the wall designer and detailed on the plans. In landscape or uncovered areas, the top of the drainage section should be covered with a filter fabric and a minimum of 12 inches of onsite soils compacted as engineered fill.

9.9.8 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

9.9.9 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., or if effervescence is undesirable, waterproofing measures should be applied to walls. Waterproofing systems should be designed by a qualified professional.

9.9.10 Retaining walls may be subject to lateral loading from pressures exerted from the soils, groundwater, foundations, and vehicular traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

## 9.10 Basement Slabs-on-Grade

9.10.1 The basement slab on grade is recommended to be supported over 6 inches of non-recycled aggregate base non-expansive engineered fill extending to the depth recommended below foundations in the Site Preparation section of this report.

9.10.2 In the event a mat foundation is used for design, a static modulus of subgrade reaction of 50 psi/inch may be used for preliminary design of mat foundations. If deep ground modification is performed, final determination of the slab support parameters, such as stiffness, should be determined by the design build ground modification engineer.

9.10.3 The interior floor slab should be reinforced for the anticipated temperature and shrinkage stresses, settlement and swell. A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on the estimated settlements. The following should be anticipated for design: 1) a total settlement and heave of 1 inch; 2) a differential static settlement of ½ inch in 30 lineal feet, 3) a total seismic settlement of 1½ inch; and 4) a differential seismic settlement of ¾ inch in 30 lineal feet. Greater vertical movement could result if positive surface drainage is not established and maintained as recommended in this report, or if the subsurface soils are subjected to elevated moisture conditions from sources such as poor drainage, inadequate maintenance, over-irrigation, utility leaks, etc.

9.10.4 The recommendations provided herein are intended only for the design of interior concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, cement mixers, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

9.10.5 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

9.10.6 A vapor retarder should be placed below interior building slabs where moisture could permeate into the interior and create problems. Refer to the American Concrete Institute's Guide to Concrete Floor and Slab Construction (ACI 302.1R) for selection and installation of moisture vapor retarders. It is recommended that a Stegowrap 15 vapor retarder be used where moisture could permeate into the interior and create problems, such as where flooring or floor slab applications will contain moisture sensitive materials (or other slab applications or uses). The vapor retarder should overlay the compacted 6 inch layer of aggregate base. It should be noted that placing the PCC slab directly on the vapor retarder may increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab unless a watertight roofing system is in place prior to slab construction to reduce the amount vapor emission through the slab-on-grade. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking.

The underslab membrane should have a high puncture resistance (minimum of approximately 2,200 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform

to the current ASTM E 1745 Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs and ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of floor covering, floor covering adhesive or other slab material applications be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

9.10.7 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

9.10.8 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.

9.10.9 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structures, 4) providing adequate drainage away from the structures, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structures.

9.10.10 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity, etc., at a frequency and method as specified by the flooring manufacturer or as required by the plans and specifications, whichever is most stringent. The results of vapor transmission tests, pH tests, internal relative humidity tests, ambient building conditions, etc. should be within floor manufacturer's and adhesive manufacturer's specifications at the time the floor is placed. It is recommended that the floor manufacturer and subcontractor review and approve the test data prior to floor covering installation.

9.10.11 To reduce the potential for damaging slabs during construction, the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to interior columns; and 2) the construction equipment which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

9.10.12 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill

as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

## 9.11 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc.

9.11.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

9.11.2 Exterior concrete slabs-on-grade (such as sidewalks, etc.) located directly adjacent to the building should be supported on a minimum of 6 inches of aggregate base over engineered fill prepared as recommended in the Site Preparation section of this report. Exterior concrete slabs-on-grade located outside the building pad preparation limits (not directly attached to the buildings) should be supported on a minimum of 6 inches of aggregate base over engineered fill prepared as recommended in the Site Preparation section of this report.

9.11.3 The moisture content of the native subgrade soils should be verified to be at least 0 to 3 percent above optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

9.11.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the non-expansive fill. This should reduce the potential for infiltration of water (and associated expansive soil movement) into the aggregate base below exterior slabs.

9.11.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

## 9.12 Asphalt Concrete (AC) Pavements

9.12.1 The subgrade soils for asphalt concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report. As part of the final preparation, the upper 12 inches of the subgrade soils should be moisture conditioned and compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D 1557.

9.12.2 The following pavement sections are based on an R-value of 45, traffic index values ranging from 5.0 to 9.0 and a minimum of 3 inches of asphalt concrete is recommended. It should be noted that if pavements are constructed prior to construction of the buildings, the traffic index value should account for construction traffic. The actual traffic index values applicable to the site should be determined by the project civil engineer.

**Table No. 9**
**Two-Layer Asphalt Concrete Pavements**

| Traffic Index | AC thickness, inches | AB thickness, inches | Compacted Subgrade, inches |
|---|---|---|---|
| 5.0 | 2.5 | 4.0 | 12 |
| 5.5 | 3.0 | 4.0 | 12 |
| 6.0 | 3.5 | 4.0 | 12 |
| 6.5 | 3.5 | 5.5 | 12 |
| 7.0 | 4.0 | 5.5 | 12 |
| 7.5 | 4.5 | 6.0 | 12 |
| 8.0 | 4.5 | 7.5 | 12 |
| 8.5 | 5.0 | 7.5 | 12 |
| 9.0 | 5.5 | 8.0 | 12 |

AC - Asphalt Concrete compacted as recommended in this report
AB - Class II Aggregate Base with minimum R-value of 78 and compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

9.12.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should extend at least to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

9.12.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

9.12.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

9.12.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

9.12.7 Pavement materials and construction method should conform to the State of California Standard Specifications.

9.12.8 It is recommended that the base 2 inch thick course of asphalt concrete consist of a ¾ inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation.

9.12.9 The asphalt concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

9.12.10 The asphalt concrete should comply with the requirements for a Type A asphalt concrete in accordance with the current State of California Department of Transportation (Caltrans) Standard Specification, or the requirements of the governing agency, whichever is more stringent.

## 9.13 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 500 psi. The design professional should specify where Portland cement concrete pavements are used based on the anticipated type and frequency of traffic.

9.13.1 The subgrade soils for Portland cement concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report. As part of the final preparation, the upper 12 inches of the subgrade soils should be moisture conditioned and compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D 1557.

9.13.2 The following preliminary Portland cement concrete pavement sections have been prepared for average daily truck traffic ranging from 1 to 60 trucks per day. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic. The design thicknesses were estimated using Streetpave at www.pavementdesigner.org based on the following parameters: 1) a minimum design compressive strength of 3,500 psi, 2) a modulus of elasticity of 3,372,000 psi, 3) a design life of 20 years, 4) truck loading consisting of 1 single axle load of 16 kips and two 32 kip tandem axles, 5) 90 percent reliability and 5 percent cracking, and 6) a modulus of subgrade reaction of 180 psi/in based on the design R-value of 5 and considering 4 inches of aggregate base underlying the pavement. A traffic index in the lower range (5.5 or 6.0) could be applied to pavements which only support automobile traffic such as parking stalls, or the below-grade parking slab proposed for the subject building.

**Table No. 10**
**Portland Cement Concrete Pavements**

| Traffic Index | ADTT | PCC Thickness (inches) | Aggregate Base (inches) | Min. Compacted Subgrade (inches) |
|---|---|---|---|---|
| 5.5 | 1 | 5.5 | 6.0 | 12 |
| 6.0 | 2 | 6.0 | 6.0 | 12 |
| 7.0 | 7 | 6.0 | 6.0 | 12 |
| 8.0 | 22 | 6.5 | 6.0 | 12 |
| 9.0 | 60 | 6.5 | 6.0 | 12 |

ADTT - Average Daily Truck Traffic based on a loaded garbage/dumpster truck
PCC - Portland Cement Concrete (minimum Modulus of Rupture=500 psi)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

9.13.3 The PCC pavement should be constructed in accordance with the American Concrete Institute requirements, the requirements of the project plans and specifications, whichever is the most stringent. The pavement design engineer should include appropriate construction details and specifications for construction joints, contraction joints, joint filler, concrete specifications, curing methods, etc.

9.13.4 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC pavement.

9.13.5 Jointing is one of the most critical aspects of the PCC pavement design and construction. Joint spacing, joint type and load transfer devices have significant impacts on the pavement design and performance. Thus, the detailing of joints needs to be considered carefully and applied with clear details on the project plans by the pavement designer/detailer. Positive load transfer devices such as dowels are commonly used at contraction joints whenever the designer cannot be assured aggregate interlock will be maintained.

9.13.6 Specifications for the concrete mixtures used in the PCC pavement to reduce the effects of excessive shrinkage (such as curling and excessive shrinkage at joints), including maximum water requirements for the concrete mix, allowable shrinkage limits, curing methods, etc. should be provided by the designer/detailer of the PCC slabs. In addition, as noted in Section 9.13.5, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement to maintain stability. The minimum PCC thickness noted in this report assumes aggregate interlock occurs at contraction joints. However, curling and excessive shrinkage can disengage aggregate interlock and allow greater pavement deflection at free edges.

9.13.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or load transfer devices are recommended for construction joints to transfer loads. The joint details should be detailed by the pavement design engineer and provided on the plans.

9.13.8 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

9.13.9 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

9.13.10 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

9.13.11 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

9.13.12 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

9.13.13 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

9.13.14 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

## 9.14 Slopes, Shoring and Temporary Excavations

9.14.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades, classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

9.14.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 1.5:1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be shored.

9.14.3 In no case should excavations extend below a 2H to 1V zone below existing utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

9.14.4 All soils disturbed as part of the shoring removal shall be over-excavated and compacted as engineered fill. In addition, all cavities and void space resulting from the shoring removal activity shall be backfilled with engineered fill or a controlled density fill material to backfill the voids created by removal of the shoring. All voids resulting from removal of shoring shall be backfilled and all soils disturbed from the shoring removal shall be over-excavated and compacted as engineered fill.

9.14.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owner should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

## 9.15 Utility Trenches

9.15.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable, the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 92 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying Moore Twining when these conditions occur and arrange for Moore Twining to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of bedding, pipe and backfill of the trench.

9.15.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation

requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned and compacted per sections 9.6 of this report. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

9.15.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in table below (minimum manufacturer requirements), or as necessary to provide sufficient space to achieve the required compaction, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

**Table No. 11**
**Minimum Trench Widths for HDPE Pipe with**
**Sand Bedding Initial Backfill**

| Inside Diameter of HDPE Pipe (inches) | Outside Diameter of HDPE Pipe (inches) | Minimum Trench Width (inches) per ASTM D2321 |
|---|---|---|
| 12 | 14.2 | 30 |
| 18 | 21.5 | 39 |
| 24 | 28.4 | 48 |
| 36 | 41.4 | 64 |
| 48 | 55 | 80 |

9.15.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Crushed rock should be placed in thin (less than 8 inch) lifts and densified with a minimum of three (3) passes using a vibratory compactor.

9.15.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned and compacted per the recommendations of section 9.6 of this report. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

9.15.6 On-site soils and approved imported engineered fill may be used as final backfill (12 inches above the pipe to the ground surface) in trenches below the required imported non-expansive fill.

9.15.7 Jetting of trench backfill is not allowed to compact the backfill soils.

9.15.8 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

9.15.9 Storm drains and/or utility lines should be designed to be "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil movement causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. The Contractor is required to video inspect or pressure test the wet utilities prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are "watertight." The Contractor shall provide the Owner a copy of the results of the testing. The Contractor is required to repair all noted deficiencies at no cost to the owner.

9.15.10 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

## 9.16 Corrosion Protection

9.16.1 Based on the corrosion severity ratings in Section 7.8 of this report, the analytical results of sample analyses indicate the samples had a resistivity value of 9,300 ohms-centimeter. Based on the resistivity values, the soils exhibit an "moderately corrosive"corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "moderately corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

9.16.2 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318 the sulfate concentration from chemical testing of soil samples falls in the S0 classification (less than 0.10 percent by weight) for concrete. Therefore, there are no restrictions required regarding the type, water-to-cement ratio, and strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio is recommended for slabs on grade as recommended in the "Basement Slabs on Grade" section of this report.

9.16.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

## 10.0 DESIGN CONSULTATION

10.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork operations, foundations and paving prior to finalization to determine whether they are consistent with our recommendations.

10.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

10.3 If Moore Twining is not retained for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

## 11.0 CONSTRUCTION MONITORING

11.1 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs, pavements, etc. be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

## 12.0 NOTIFICATION AND LIMITATIONS

12.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions.

12.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

12.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

12.4 Changed site conditions, or relocation of proposed structure(s), may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

12.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structure on this site not discussed herein is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

12.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

12.7 This report presents the results of a geotechnical engineering investigation and geologic/seismic hazards investigation only and should not be construed as an environmental audit or study.

12.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

12.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

## 13.0 CLOSING

We appreciate the opportunity to be of service to the City of Riverside. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

**MOORE TWINING ASSOCIATES, INC.**
Geotechnical Engineering Division

DRAFT

Shaun Reich, EIT
Project Engineer

DRAFT

Allan Harker, CEG
Certified Engineering Geologist

DRAFT

Read L. Andersen, RGE
Geotechnical Division Manager

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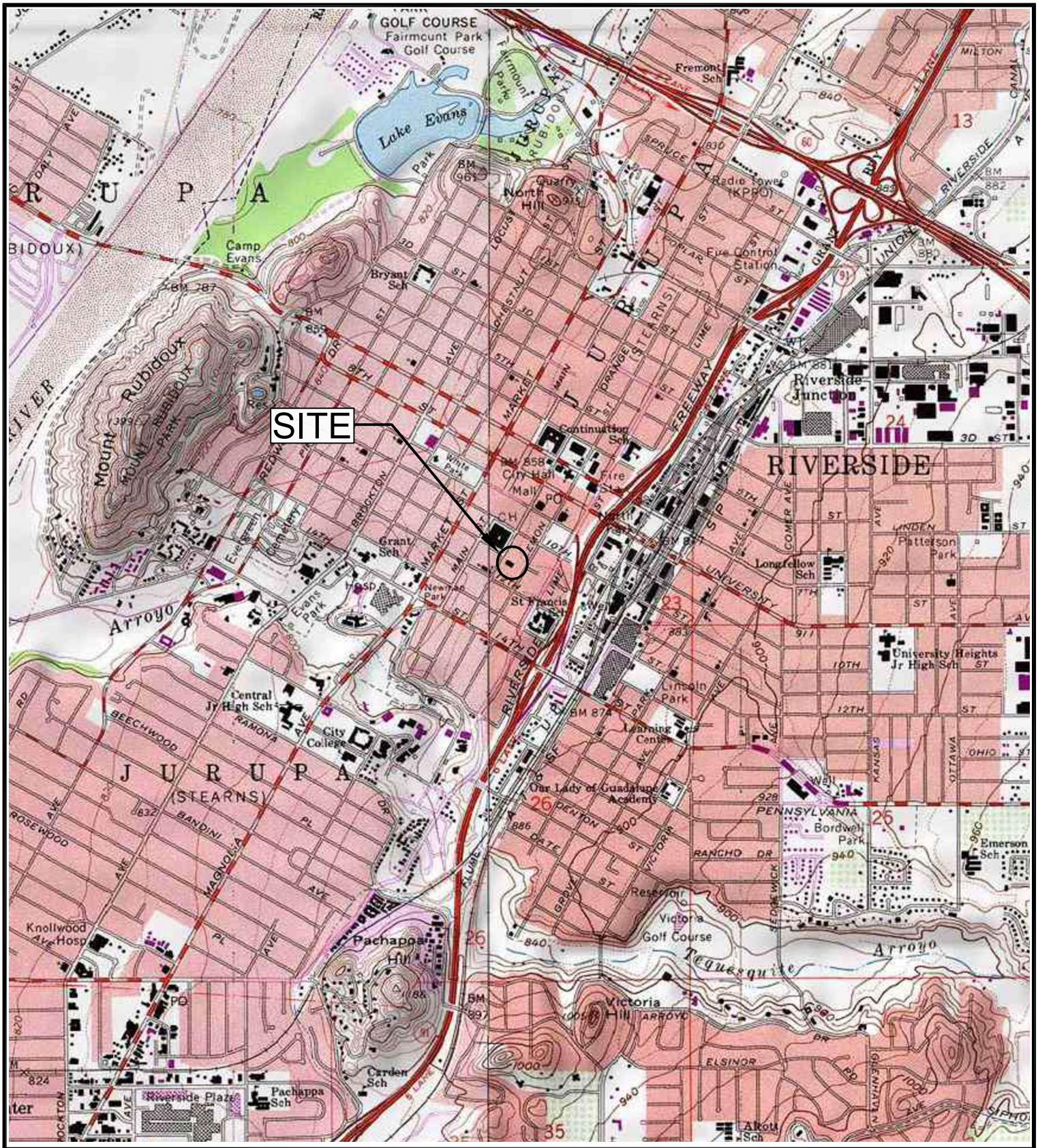
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# APPENDIX A

## DRAWINGS

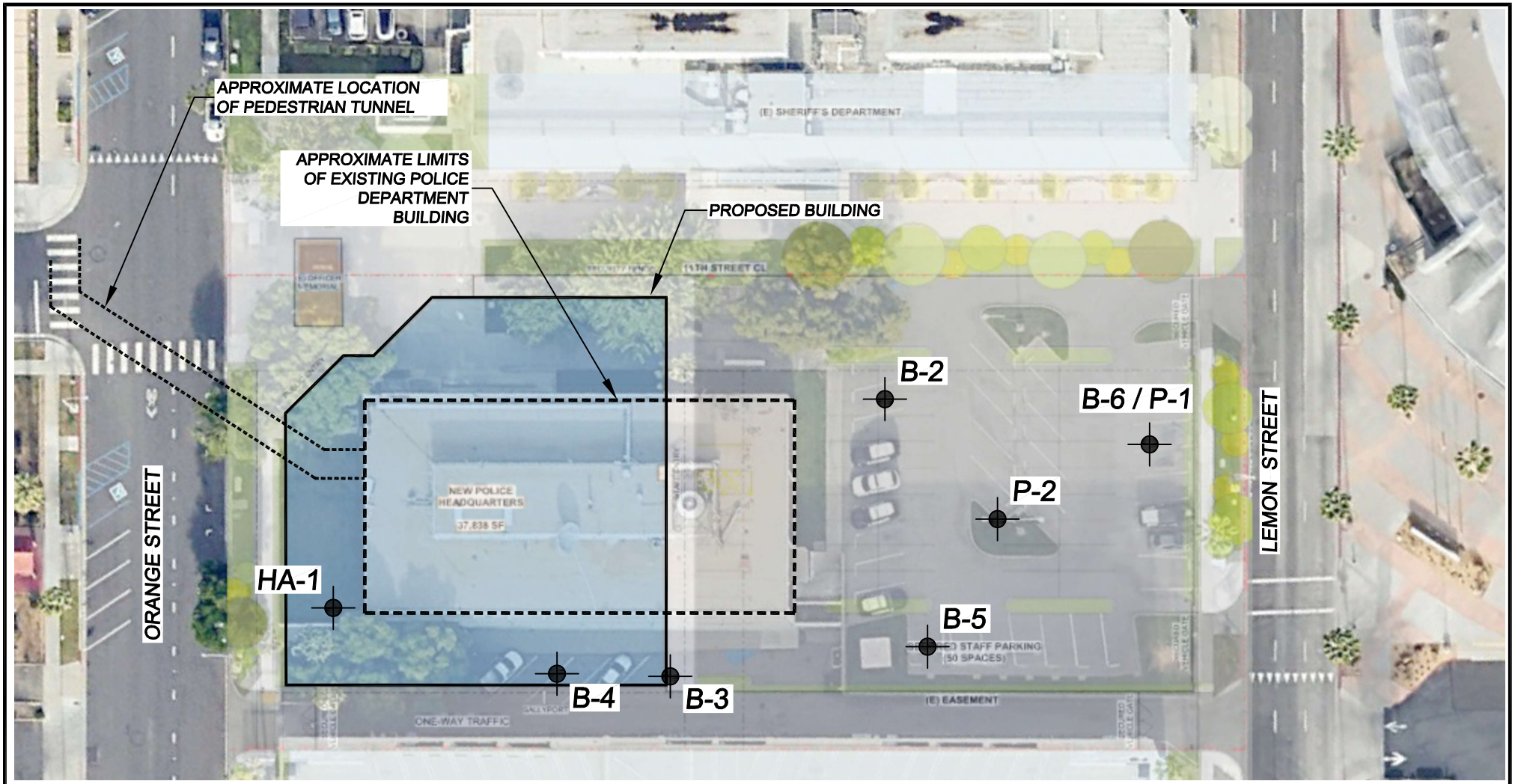
Drawing No. 1 - Site Location Map
Drawing No. 2 - Test Boring Location Map
Drawing No. 3 - Regional Geologic Map
Drawing No. 4 - Map of Major Active and Potentially Active Faults Relative to Site
Drawing No. 5 - Historical Earthquake Epicenter Map
Drawing No. 6 - Site Geologic Map
Drawing No. 7 - Geologic Soil Profile Cross Section A-A'
Drawing No. 8 - Geologic Soil Profile Cross Section B-B'




SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
RIVERSIDE EAST, CALIFORNIA QUADRANGLE 1978

0 2000
APPROXIMATE SCALE
IN FEET

| | | | |
|---|---|---|---|
| SITE LOCATION MAP<br>PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING<br>4102 ORANGE STREET<br>RIVERSIDE, CALIFORNIA | FILE NO.: 01602-01-01 | DATE: 10/08/2025 | MOORE TWINING ASSOCIATES, INC. |
| | DRAWN BY: RM | APPROVED BY: | |
| | PROJECT NO. H01602.01 | DRAWING NO. 1 | |




B-1 — APPROXIMATE TEST BORING LOCATION

HA-1 — APPROXIMATE HAND AUGER BORING LOCATION

P-1 — APPROXIMATE PERCOLATION TEST LOCATION

N

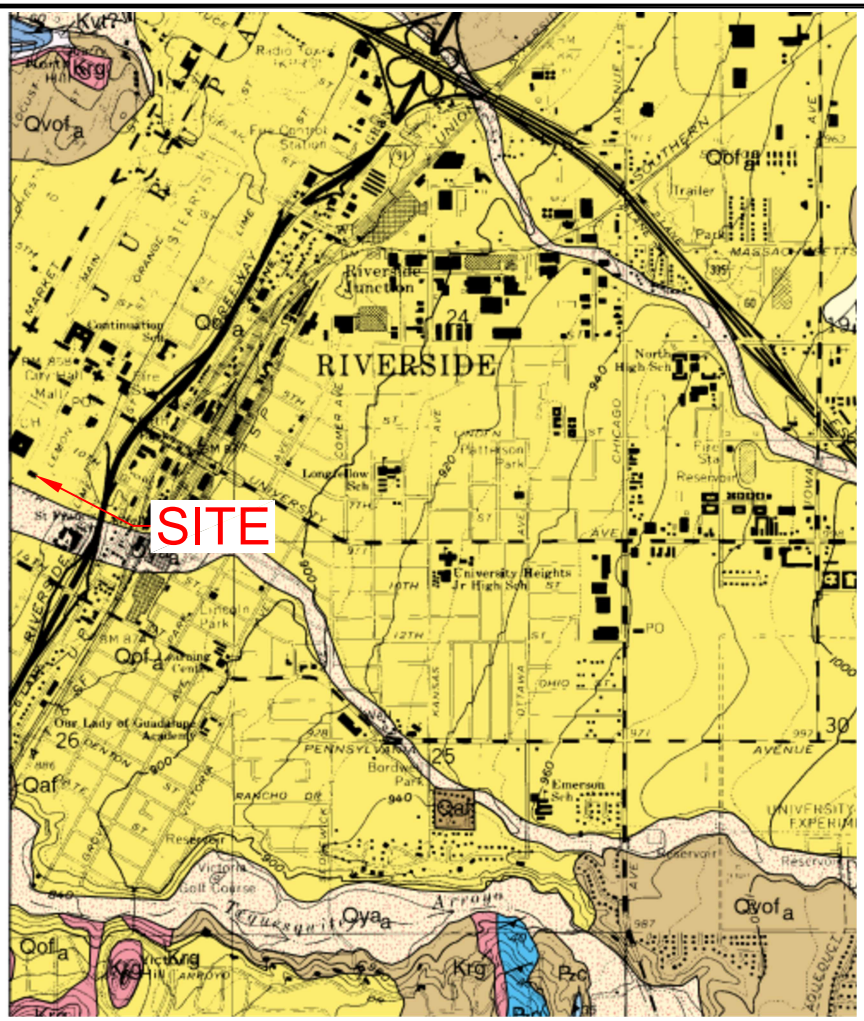
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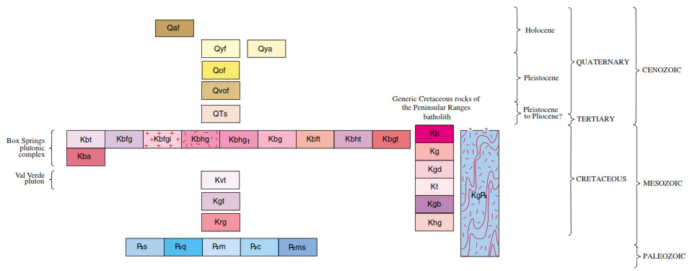
APPROXIMATE SCALE IN FEET

TEST BORING LOCATION MAP
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 2 |

MOORE TWINING ASSOCIATES, INC.
Since 1898




Qyf — **Young alluvial fan deposits (Holocene and late Pleistocene)**—Gray-hued sand and cobble- and gravel-sand deposits derived chiefly from rocks of Peninsular Ranges batholith. Found in restricted drainages along west edge of quadrangle, but contiguous with much more extensively developed deposits west of quadrangle

Qya — **Young axial channel deposits (Holocene and late Pleistocene)**—Gray, unconsolidated alluvium consisting of medium- to fine-grained sand and lesser silt flooring several low relief valleys and their tributaries in northwestern and northeastern part of quadrangle. Includes sediments in Tequesquite Arroyo and Pigeon Pass Valley

**OLD SURFICIAL DEPOSITS**—Sedimentary units that are moderately consolidated and slightly to moderately dissected. Older surficial deposits have upper surfaces that are capped by moderately to well-developed pedogenic soils (A/AB/B/$C_{ox}$ profiles and Bt horizons as much as 1 to 2 m thick and maximum hues in the range of 10YR 5/4 and 6/4 through 7.5YR 6/4 to 4/4 and mature Bt horizons reaching 5YR 5/6). Includes:

Qof — **Old alluvial fan deposits (late to middle Pleistocene)**—Indurated, sandy alluvial fan deposits developed extensively in western part of quadrangle. Most of unit is slightly to moderately dissected and reddish-brown. Some Qof includes thin, discontinuous surface layer of Holocene alluvial fan material

**VERY OLD SURFICIAL DEPOSITS**—Sediments that are slightly to well consolidated to indurated, and moderately to well dissected. Upper surfaces are capped by moderate to well developed pedogenic soils (A/AB/B/$C_{ox}$ profiles having Bt horizons as much as 2 to 3 m thick and maximum hues in the range 7.5YR 6/4 and 4/4 to 2.5YR 5/6)

Qvof — **Very old alluvial fan deposits (early Pleistocene)**—Mostly well-dissected, well-indurated, reddish-brown sand deposits. Commonly contains duripans and locally silcretes. Forms large area in southeastern part of quadrangle in area of March Air Force Base, and numerous smaller areas in northern part of quadrangle. Derived chiefly from rocks of southern California batholith

Pzm — **Marble (Paleozoic?)**—Marble; white to light-gray, locally bluish-gray and blue, coarse to extremely coarse grained

Pzc — **Calc-silicate rocks (Paleozoic?)**—Heterogeneous, massive to well-layered calc-silicate rocks

Pzms — **Marble and schist, undifferentiated (Paleozoic?)**—Intermixed marble, calc-silicate rock, and biotite schist. Mapped on North Hill in northwestern part of quadrangle

shaped, meso-to melanocratic inclusions are common.

Krg — **Granite of the Riverside area (Cretaceous)**—Medium- to coarse-grained, massive- to faintly-foliated, leucocratic biotite granite. Contains about 1 to 3 percent biotite. Inclusions are sparse or absent except locally in western part of body, west of quadrangle, where rock contains 2 to 8 percent biotite and sparse to abundant inclusions of quartz diorite, granodiorite, and fine-grained mafic rock. At Mount Rubidoux, west of quadrangle, rocks contain sparse hypersthene and fayalitic olivine and moderately abundant equant inclusions of dark-gray fine-grained rock

*SOURCE:* "Geologic Map of the Riverside East 7.5' Quadrangle, Riverside County, California," prepared by Douglas M. Morton and Brett Cox and the U.S. Department of the Interior, U.S. Geological Survey, from Open-File Report 01-452, dated 2001

N

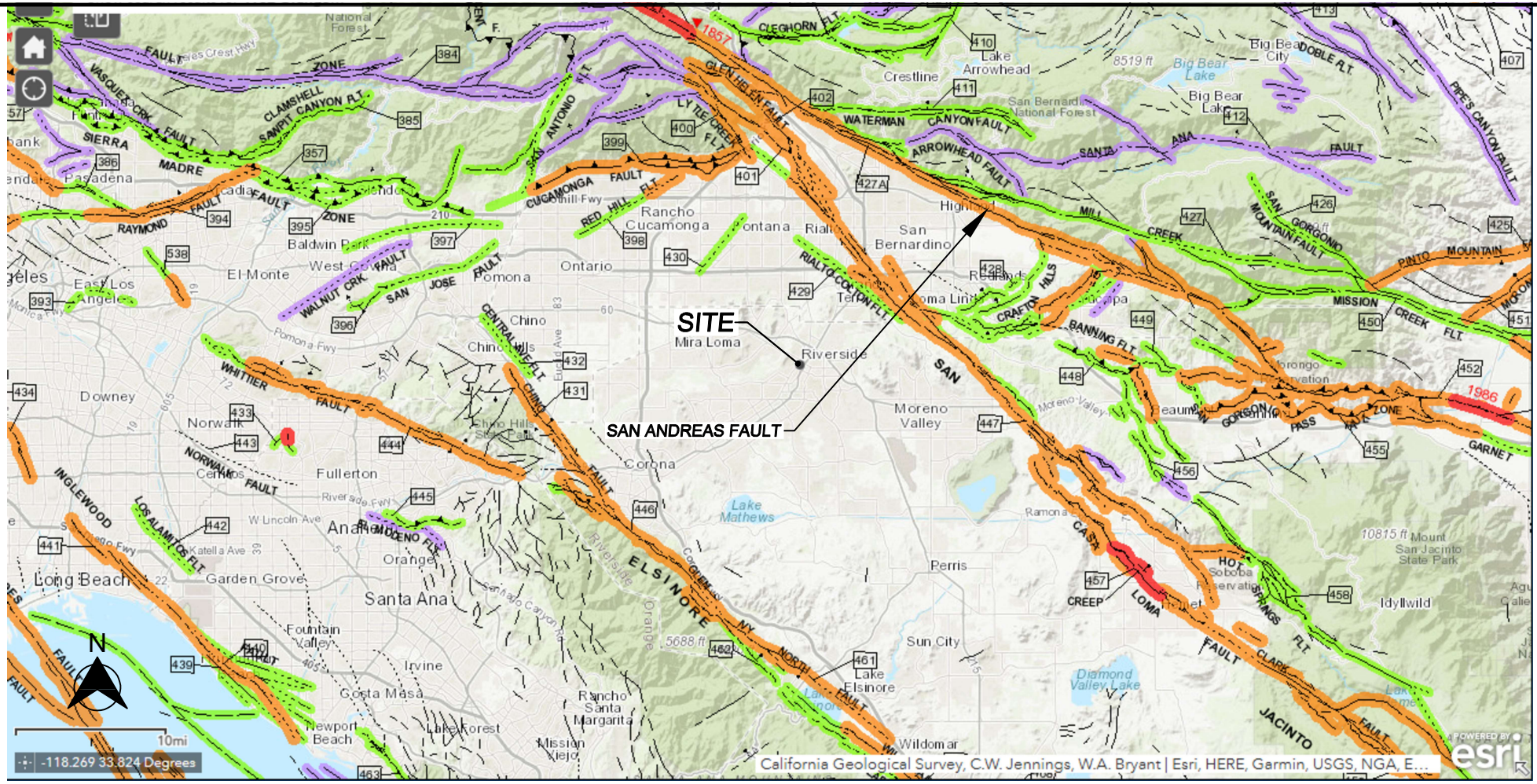
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APPROXIMATE SCALE IN MILES

REGIONAL GEOLOGIC MAP
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

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|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 3 |

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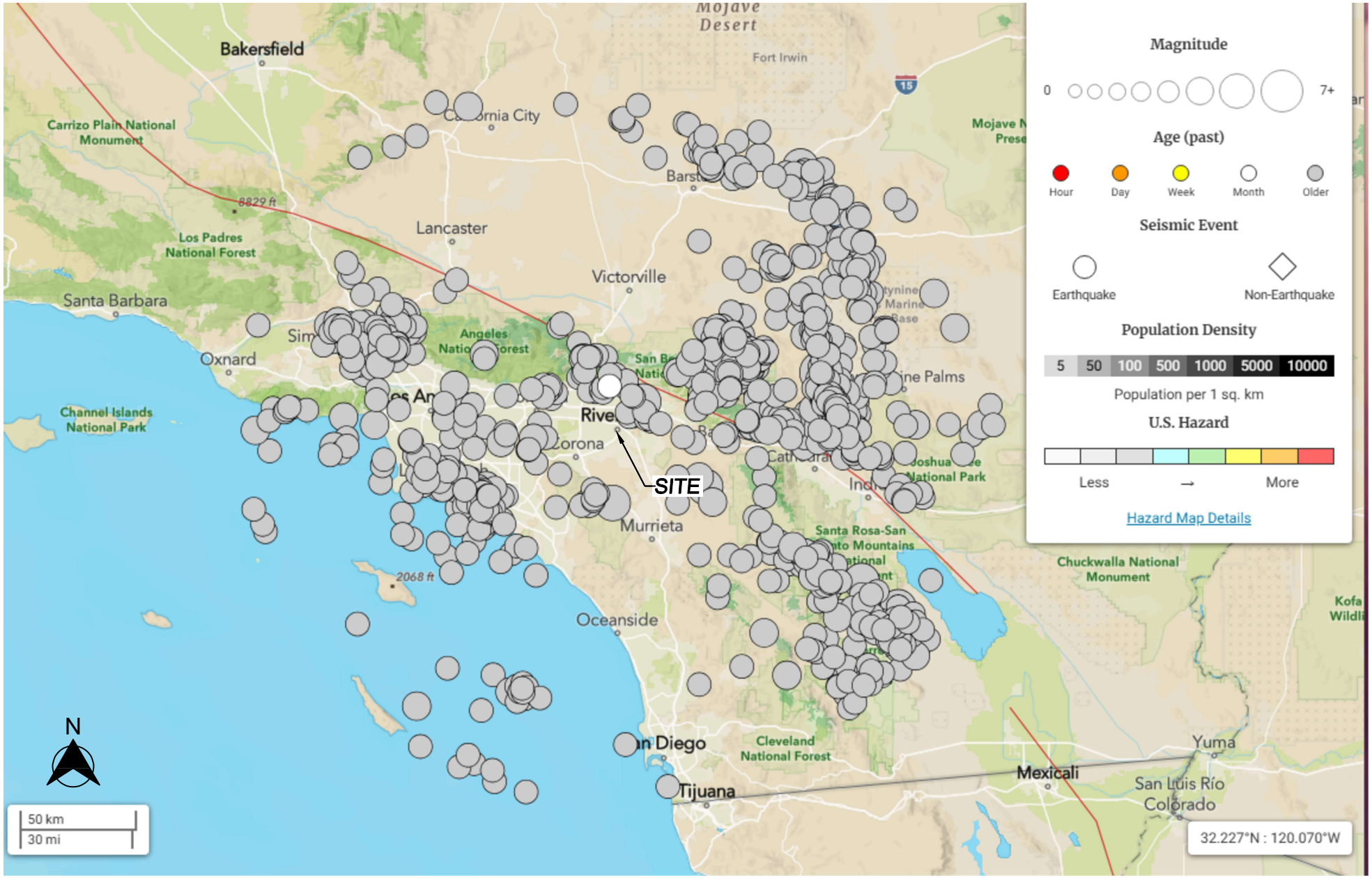

## LEGEND

- *Fault along which historic (last 200 years) displacement has occurred*
- *Holocene fault displacement (during past 11,700 years) without historic record*
- *Late Quaternary fault displacement (during past 700,000 years)*
- *Quaternary fault (age undifferentiated)*

MAP OF ACTIVE AND POTENTIALLY ACTIVE FAULTS RELATIVE TO SITE
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 4 |

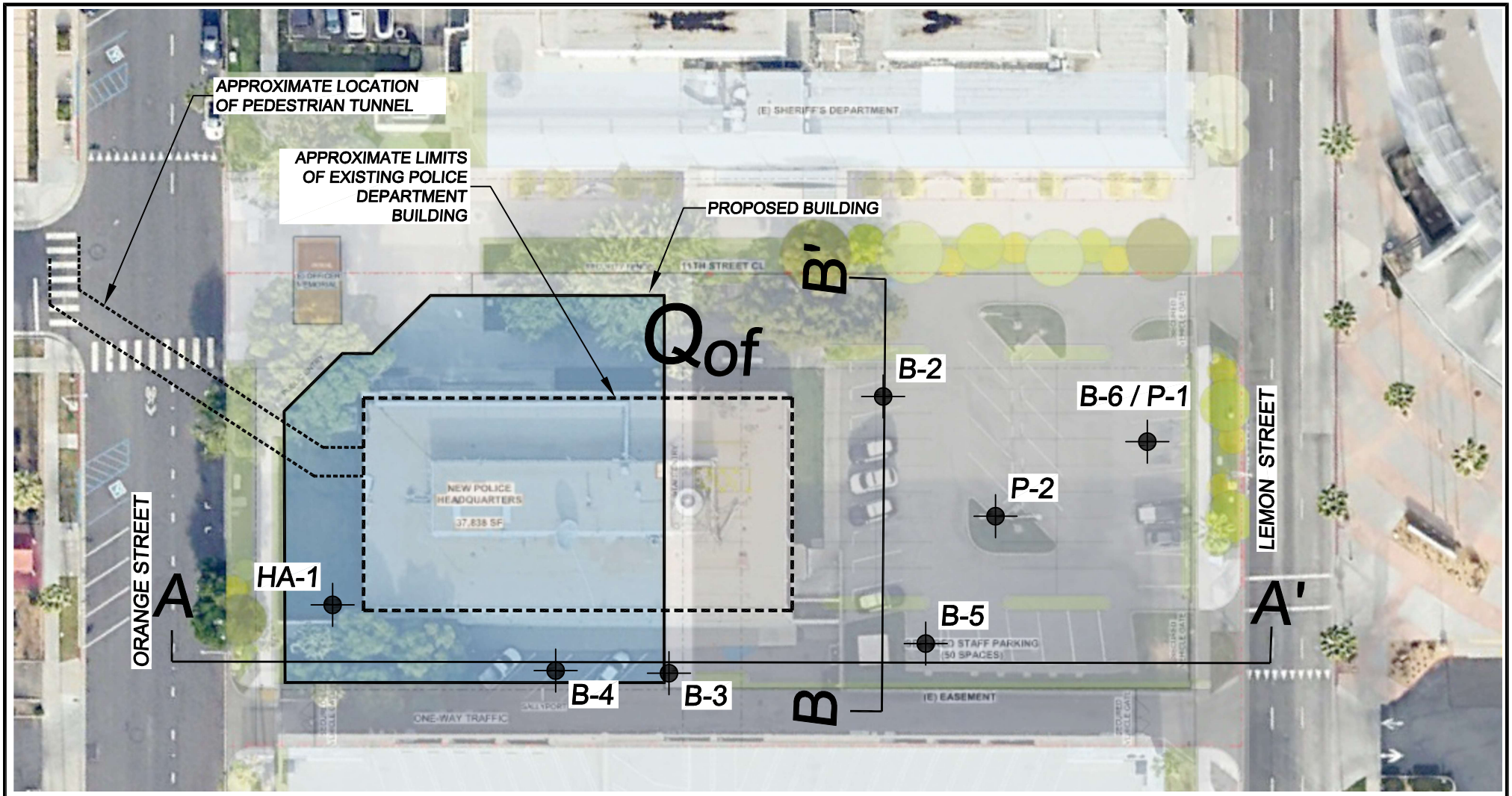
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HISTORICAL EARTHQUAKE EPICENTER MAP
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 5 |

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A — A' CROSS SECTION

Qof - OLD ALLUVIAL FAN DEPOSITS

B-1 — APPROXIMATE TEST BORING LOCATION

HA-1 — APPROXIMATE HAND AUGER BORING LOCATION

P-1 — APPROXIMATE PERCOLATION TEST LOCATION

N

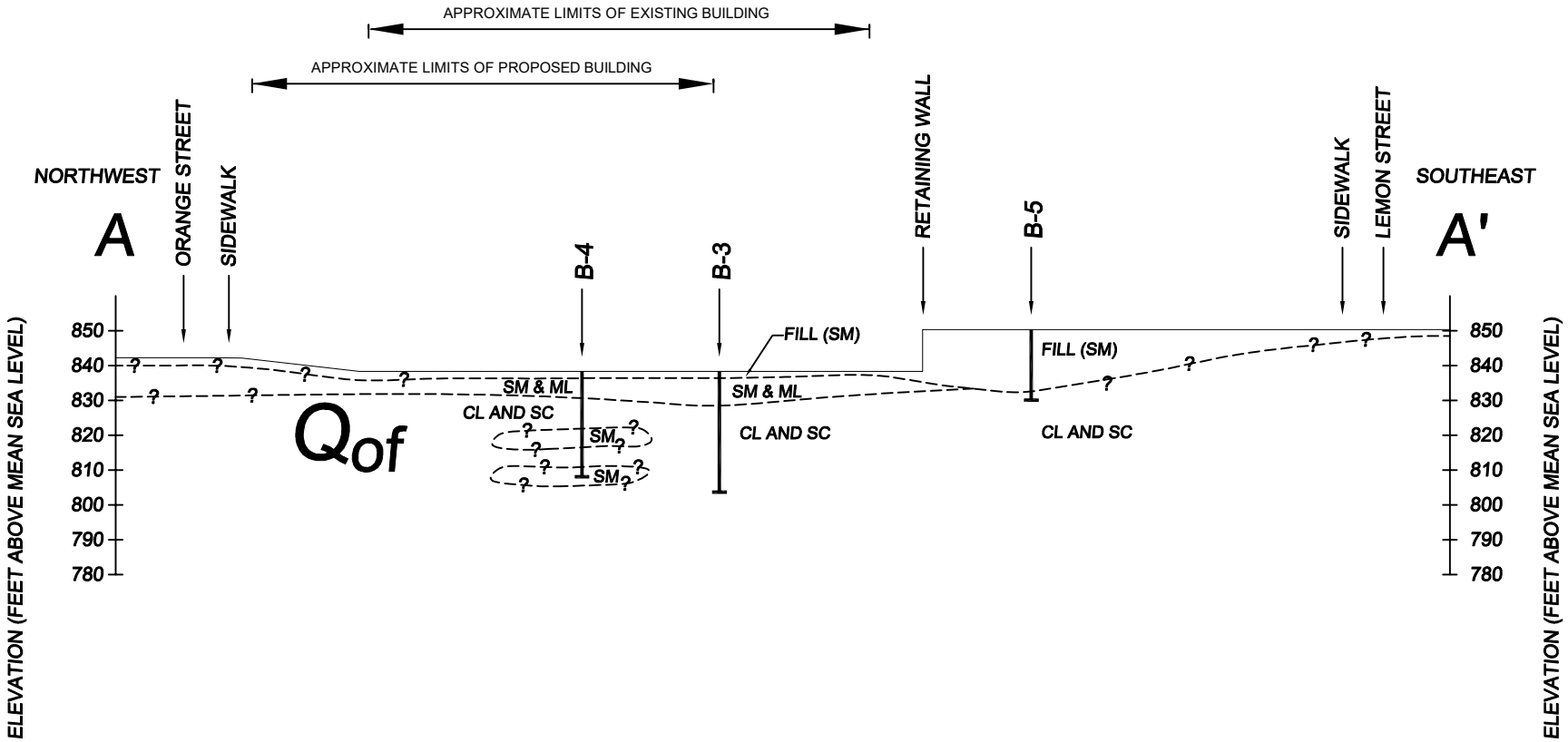
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APPROXIMATE SCALE IN FEET

SITE GEOLOGIC MAP
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 6 |

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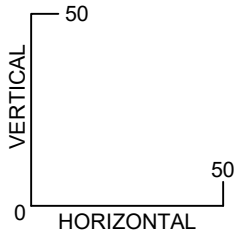



**GEOLOGIC UNITS**

$Q_{of}$: OLD ALLUVIAL FAN DEPOSITS

**SOIL CLASSIFICATION**

SM: SILTY SAND
SC: CLAYEY SAND
CL: SANDY LEAN CLAY
ML: SANDY SILT

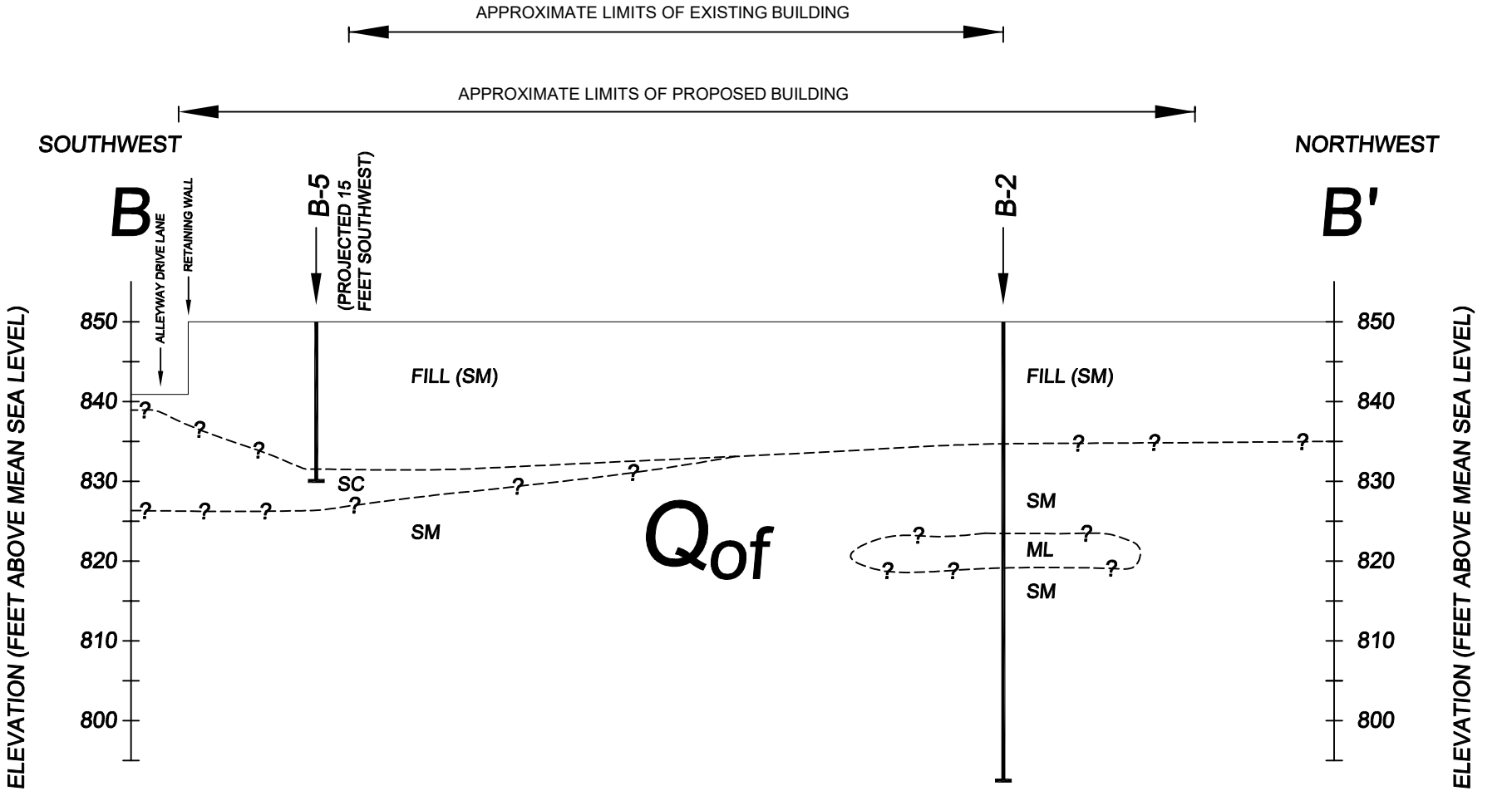



APPROXIMATE SCALE
IN FEET

GEOLOGIC SOIL PROFILE CROSS SECTION A-A'
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 7 |

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GEOLOGIC SOIL PROFILE CROSS SECTION B-B'
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 10/09/2025 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 8 |

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# APPENDIX B

## LOG OF TEST BORINGS

This appendix contains the final boring logs. The logs represents our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at the test boring locations. Also, the passage of time may result in changes in the soil conditions.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

MOORE TWINING ASSOCIATES, INC.

# Test Boring: HA-1

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** N/A

**Auger Type:** Hand Auger with 5-inch Diameter Bucket

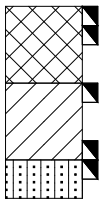
**Hammer Type:** N/A

**Logged By:** SR

**Date:** 8/21/2025

**Elevation:** 849

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 0 |  | FILL | 3 Inches of grass and rootlets over FILL - SILTY SAND; very moist, fine to medium grained, light brown | | | 16.3 |
| | | CL | Wet, dark brown, trace clay | From 2-2.5' PI = 13 LL = 21 -200 = 57.1% Sand = 42.4% +4 = 0.5% | | 15.7 |
| 845 5 | | SM | NATIVE-SANDY LEAN CLAY; moist, low plasticity, dark brown | | | 12.6 |
| | | | Decrease in moisture content, light brown | | | 10.0 |
| 840 10 | | | SILTY SAND; moist, fine to medium grained, trace coarse sand, brown | | | |
| | | | Bottom of boring HA-1 at 5 feet BSG. | | | |
| 835 15 | | | | | | |
| 830 20 | | | | | | |
| 825 25 | | | | | | |

**Notes:** *Grass landscaping adjacent to Orange Street.*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-2

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

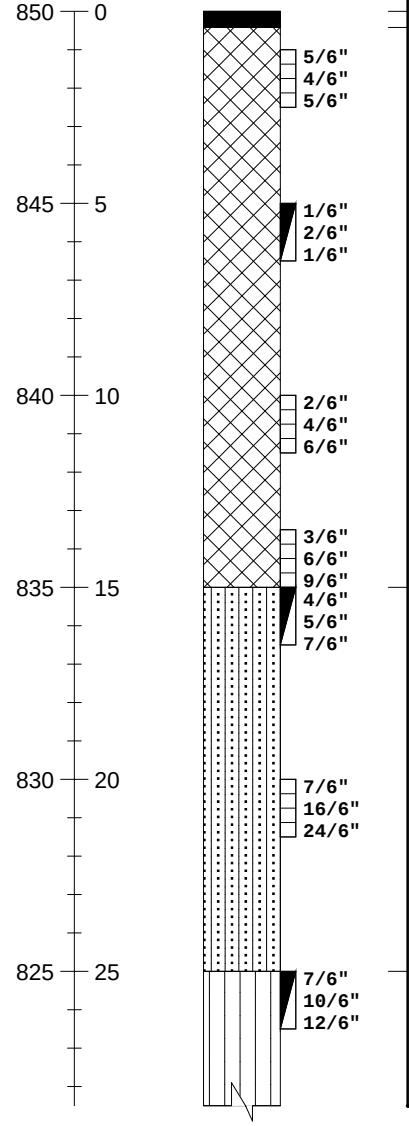
**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/2025

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E




| ELEVATION/ DEPTH (feet) | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|
| | AC | Asphalt Concrete Thickness = 5 inches | | | |
| | FILL | FILL - SANDY SILT; medium stiff, damp, non-plastic, brown, trace fine gravel | DD = 105.2 pcf<br>-200 = 64.0%<br>Sand = 33.1%<br>+4 = 2.9% | 9 | 6.7 |
| | | Soft | -200 = 59.5% | 3 | |
| | | Medium stiff, with plastic debris | DD = 101.9 pcf<br>-200 = 59.4% | 10 | 5.5 |
| | | Stiff, with organics (rootlets), trace fine gravel | From 13.5-15'<br>DD = 117.2 pcf<br>-200 = 63.1%<br>Sand = 36.3%<br>+4 = 0.6% | 15 | 6.7 |
| | SM | NATIVE - SILTY SAND; medium dense, moist, medium-to-coarse grained, trace fine grained sand and fine gravel, brown | From 15-16.5'<br>-200 = 20.4% | 12 | 3.3 |
| | | With interbedded lenses of clay up to 3 inches in thickness | DD = 122.5 pcf<br>ø = 36°<br>c = 560 psf<br><br>Increased drill resistance | 40 | 6.8 |
| | ML | SANDY SILT; very stiff, moist, non-plastic, brown | -200 = 52.3%<br>Sand = 47.6%<br>+4 = 0.1% | 22 | 10.5 |

**Notes:** *Asphalt pavement at surface*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-2

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

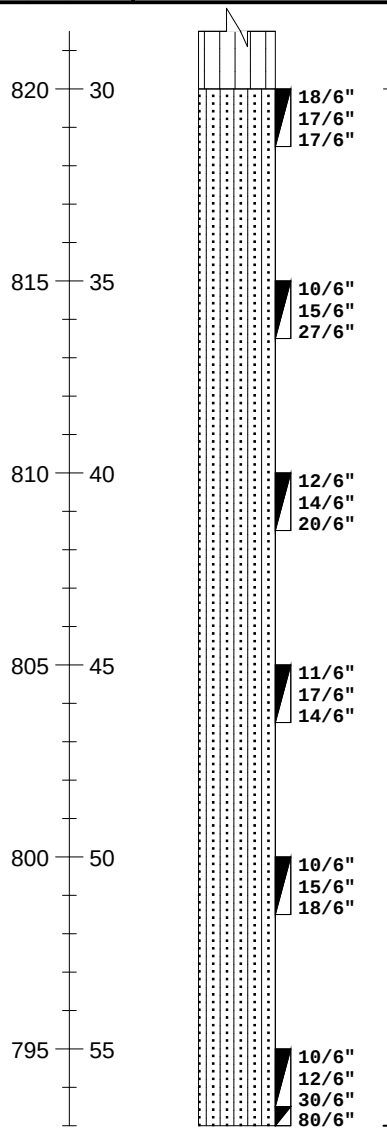
**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/2025

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E




| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| | | SM | SILTY SAND; dense, fine to coarse grained, brown, with interbedded lenses of clay up to 2 inches thickness | | 34 | 10.3 |
| | | | Without clay lenses | | 42 | |
| | | | | Increased drill resistance | 34 | |
| | | | Fine with medium grained sand | Increased drill resistance | 31 | |
| | | | | | 33 | |
| | | | | | 42 | |
| | | | | | >50 | |

**Notes:** *Asphalt pavement at surface*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-2

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/2025

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 790 — 60 | | | Very dense, moderately cemented | | | |
| 785 — 65 | | | Bottom of boring B-2 at 57 feet BSG. | | | |
| 780 — 70 | | | | | | |
| 775 — 75 | | | | | | |
| 770 — 80 | | | | | | |
| 765 — 85 | | | | | | |

**Notes:** *Asphalt pavement at surface*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-3

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

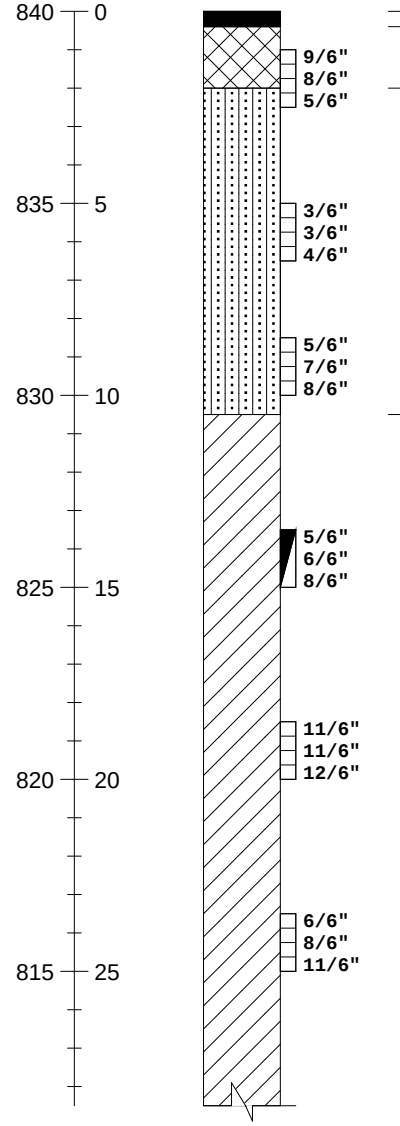
**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/21/2025

**Elevation:** 840

**Depth to Groundwater First Encountered During Drilling:** N/E




| USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|
| AC | Asphalt Concrete Thickness = 4.5 inches | | | |
| FILL | | From 1-2.5': DD = 110.6 pcf PI = 13 LL = 21 -200 = 41.1% Sand = 50.1% +4 = 8.8% | 13 | 4.7 |
| SM | FILL - CLAYEY SAND; loose, moist, fine-to-medium grained, brown, with calcification, with a little fine gravel | | | |
| | At 2 feet - NATIVE - SILTY SAND; loose, moist, fine-to-medium grained, brown | | 7 | 5.3 |
| | Fine and coarse grained | From 1-5': EI = 0 SR = 9,300 ohm-cm pH = 8.1 Cl = ND SS = 0.0011% | 15 | 6.1 |
| CL | SANDY LEAN CLAY; damp, low plasticity, brown | From 5-6.5': DD = 108.0 pcf | | |
| | Stiff, trace fine gravel | From 8.5-10': DD = 108.4 pcf From 13.5-15' PI = 10 LL = 23 -200 = 59.9% Sand = 39.6% +4 = 0.5% | 14 | 12.3 |
| | With 4-inch thick of interbedded lenses of fine and coarse-grained silty sand | DD = 114.2 pcf | 23 | 11.9 |
| | Medium plasticity, brown, trace fine gravel | -200 = 53.5% Sand = 46.3% +4 = 0.2% | 14 | 10.8 |

**Notes:** *Asphalt pavement at surface. Overlay separated by aluminum foil(?)*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-3

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/21/2025

**Elevation:** 840

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 810 — 30 | 6/6" 8/6" 11/6" | | | Increased drill resistance | 19 | |
| 805 — 35 | 12/6" 12/6" 23/6" | SC | CLAYEY SAND; dense, moist, fine to coarse grained, reddish-brown, with calcification | | 35 | |
| | | | Bottom of Boring B-3 at 35 feet BSG. | | | |
| 800 — 40 | | | | | | |
| 795 — 45 | | | | | | |
| 790 — 50 | | | | | | |
| 785 — 55 | | | | | | |

**Notes:** *Asphalt pavement at surface. Overlay separated by aluminum foil(?)*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-4

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

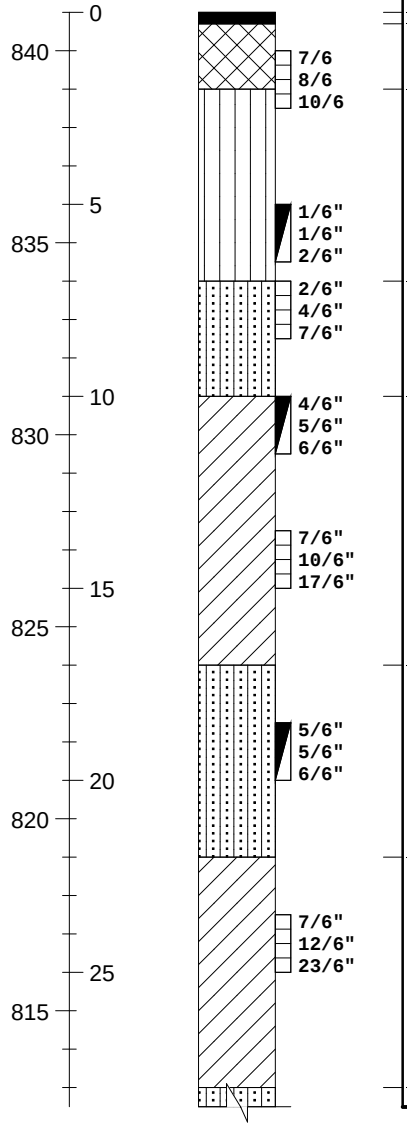
**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/21/2025

**Elevation:** 841

**Depth to Groundwater First Encountered During Drilling:** N/E




| ELEVATION/ DEPTH (feet) | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|
| | AC | Asphalt Concrete Thickness = 4 inches | | | |
| | FILL | | DD = 109.2 pcf | 18 | 5.4 |
| | ML | FILL - SILTY SAND; medium dense, damp, fine-to-medium grained, with fine gravel, brown | | | |
| | | At 2 feet - NATIVE - SANDY SILT; damp, low plasticity, brown Soft, trace fine gravel | -200 = 58.2% Sand = 39.0% +4 = 2.8% | 3 | 5.4 |
| | SM | SILTY SAND; loose, moist, fine-to-medium grained, trace coarse sand, brown | From 7-8.5': DD = 115.7 pcf PI = NP LL = NV ø = 40° c = 130psf | 11 | 6.0 |
| | CL | SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown | | 11 | 10.6 |
| | | Very stiff, increase in moisture content, low-to-medium plasticity | DD = 106.9 pcf | 27 | 17.3 |
| | SM | SILTY SAND; medium dense, moist, fine to medium grained, brown | | 11 | 12.8 |
| | CL | SANDY LEAN CLAY; very stiff, moist, low-to-medium plasticty, brown | DD = 110.6 pcf | 35 | 15.2 |

**Notes:** *Asphalt pavement at surface.*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-4

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/21/2025

**Elevation:** 841

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 30 / 810 | 11/6" 18/6" 20/6" | SM | SILTY SAND; dense, moist, fine-to-medium grained, brown | | 38 | |
| | | | Bottom of boring B-4 at 30 feet BSG. | | | |
| 35 / 805 | | | | | | |
| 40 / 800 | | | | | | |
| 45 / 795 | | | | | | |
| 50 / 790 | | | | | | |
| 55 / 785 | | | | | | |

**Notes:** *Asphalt pavement at surface.*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-5

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 6-5/8" Diameter Hollow Stem

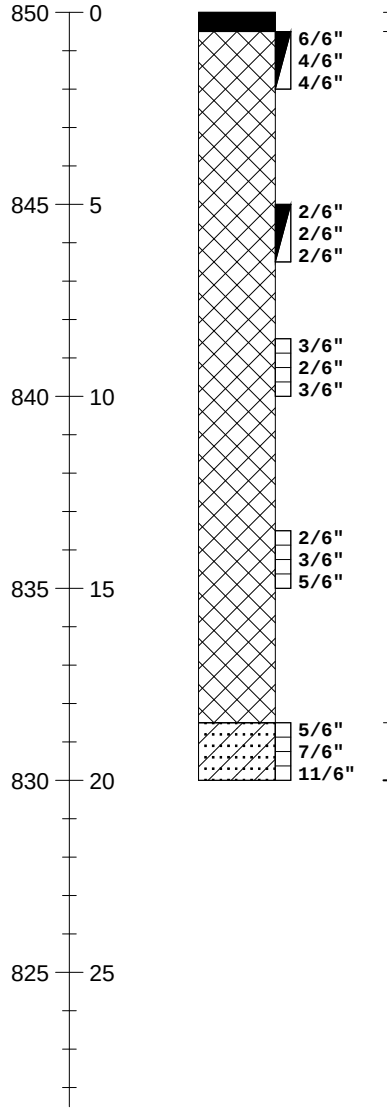
**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/25

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E




| ELEVATION/ DEPTH (feet) | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|
| | AC | Asphalt Concrete Thickness = 5 inches | | 8 | 9.4 |
| | FILL | FILL - SILTY SAND; loose, moist, fine to medium grained, brown, with asphalt debris | | | |
| | | Brown to dark brown, no debris, trace fine gravel | -200 = 43.3%<br>Sand = 55.5%<br>+4 = 1.2% | 4 | 10.5 |
| | | | DD = 105.2 pcf | 5 | 9.4 |
| | | | DD = 101.8 pcf | 8 | 9.1 |
| | SC | NATIVE - CLAYEY SAND; medium dense, moist, fine-to-medium grained, dark brown | DD = 116.2 pcf<br>PI = 10<br>LL = 21<br>-200 = 45.3%<br>Sand = 53.9%<br>+4 = 0.8% | 18 | 10.5 |
| | | Bottom of boring B-5 at 20 feet BSG. | | | |

**Notes:** *Asphalt pavement at surface.*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: B-6/P-1

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 8" Diameter Hollow Stem

**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/2025

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 850 — 0 | 7/6" 5/6" 6/6" | AC FILL | Asphalt Concrete Thickness = 6 inches (petromat observed at 2 inches below paved surface) | No recovery, asphalt clogged shoe of sampler | 11 | |
| | | | FILL - SILTY SAND; loose, moist, with fine-to-medium grained, brown | From 0.5-5': R-value = 45 | | |
| | 4/6" 5/6" 5/6" | | | DD = 100.2 pcf | 10 | 7.4 |
| 845 — 5 | | SM | NATIVE - SILTY SAND; loose, moist, with fine to medium grained sand, brown | | | |
| | 2/6" 2/6" 3/6" | ML | SANDY SILT; medium stiff moist, non-plastic, brown | -200 = 56.3% | 5 | |
| 840 — 10 | | | Bottom of boring B-6 at 10 feet BSG. | | | |
| 835 — 15 | | | | | | |
| 830 — 20 | | | | | | |
| 825 — 25 | | | | | | |

**Notes:** *Boring converted to percolation test hole P-1 after reaching target depth of 10 feet BSG.*

**Figure Number**

MOORE TWINING ASSOCIATES, INC.

# Test Boring: P-2

**Project:** Riverside Police Department Replacement

**Project Number:** H01602.01

**Drilled By:** JS

**Drill Type:** CME 75

**Auger Type:** 8" Diameter Hollow Stem

**Hammer Type:** 140 pound Auto Trip

**Logged By:** SR

**Date:** 8/20/2025

**Elevation:** 850

**Depth to Groundwater First Encountered During Drilling:** N/E

| ELEVATION/ DEPTH (feet) | SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA | USCS | Soil Description | Remarks | N-Values blows/ft. | Moisture Content % |
|---|---|---|---|---|---|---|
| 850 — 0 | | AC | Asphalt Concrete Thickness = 6 inches | | | |
| | | FILL | FILL - SILTY SAND; fine-to-medium grained, brown, with asphalt debris | | | |
| 845 — 5 | | | | | | |
| | 2/6" 2/6" 3/6" | | Loose, moist, fine with medium grained, brown | -200 = 44.9% | 5 | |
| 840 — 10 | | | Bottom of boring P-2 at 10 feet BSG. | | | |
| 835 — 15 | | | | | | |
| 830 — 20 | | | | | | |
| 825 — 25 | | | | | | |

**Notes:**

**Figure Number**

# KEY TO SYMBOLS

| Symbol | Description |
|---|---|
| **Strata symbols** | |
|  | Fill |
|  | Lean Clay |
|  | Silty Sand |
|  | Asphalt concrete |
|  | Silt |

| Symbol | Description |
|---|---|
|  | Clayey Sand |
| **Misc. Symbols** | |
|  | Drill rejection |
|  | Boring continues |
| **Soil Samplers** | |
|  | Bulk/Grab sample |

**Notes:**

1. Exploratory borings were drilled on August 20 & 21, 2025 using a CME 75 drill rig equipped with 6-5/8" outside diameter hollow stem augers and a 4-1/2" outside-diameter hand auger.

2. Groundwater was not encountered in any of the borings.

3. Boring locations were measured or paced from existing features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an SPT equivalent N-value.

6. Results of tests conducted on samples recovered are reported on the logs.

| | |
|---|---|
| DD = Natural dry density (pcf) | LL = Liquid Limit (%) |
| +4 = Percent retained on the No. 4 sieve(%) | PI = Plasticity Index (%) |
| -200 = Percent passing the No. 200 sieve (%) | EI = Expansion Index |
| Sand = Percent passing the No. 4 sieve and retained on No. 200 sieve (%) | Gravel = Percent passing 3-inch & retained on No. 4 sieves(%) |
| SR = Soil resistivity (ohms-cm) | pH = Soil pH |
| SS = Soluble sulfates (%) | Cl = Soluble chlorides (%) |
| ø = Internal Angle of Friction (degrees) | c = Cohesion (psf) |
| pcf = Pounds per cubic foot | psf = Pounds per square foot |
| O.D. = Outside diameter | AMSL = Above mean sea level |
| N/A = Not applicable | N/E = Not encountered |
| NP = Non-plastic | NV = Non-viscous |
| BSG = Below Site Grade Surface | |
| ND = Not Detected (Reporting Limits below 0.0006%) | |

# KEY TO SYMBOLS

**Symbol Description**

**Soil Samplers**



**California Modified split barrel ring sampler**



**Standard penetration test**

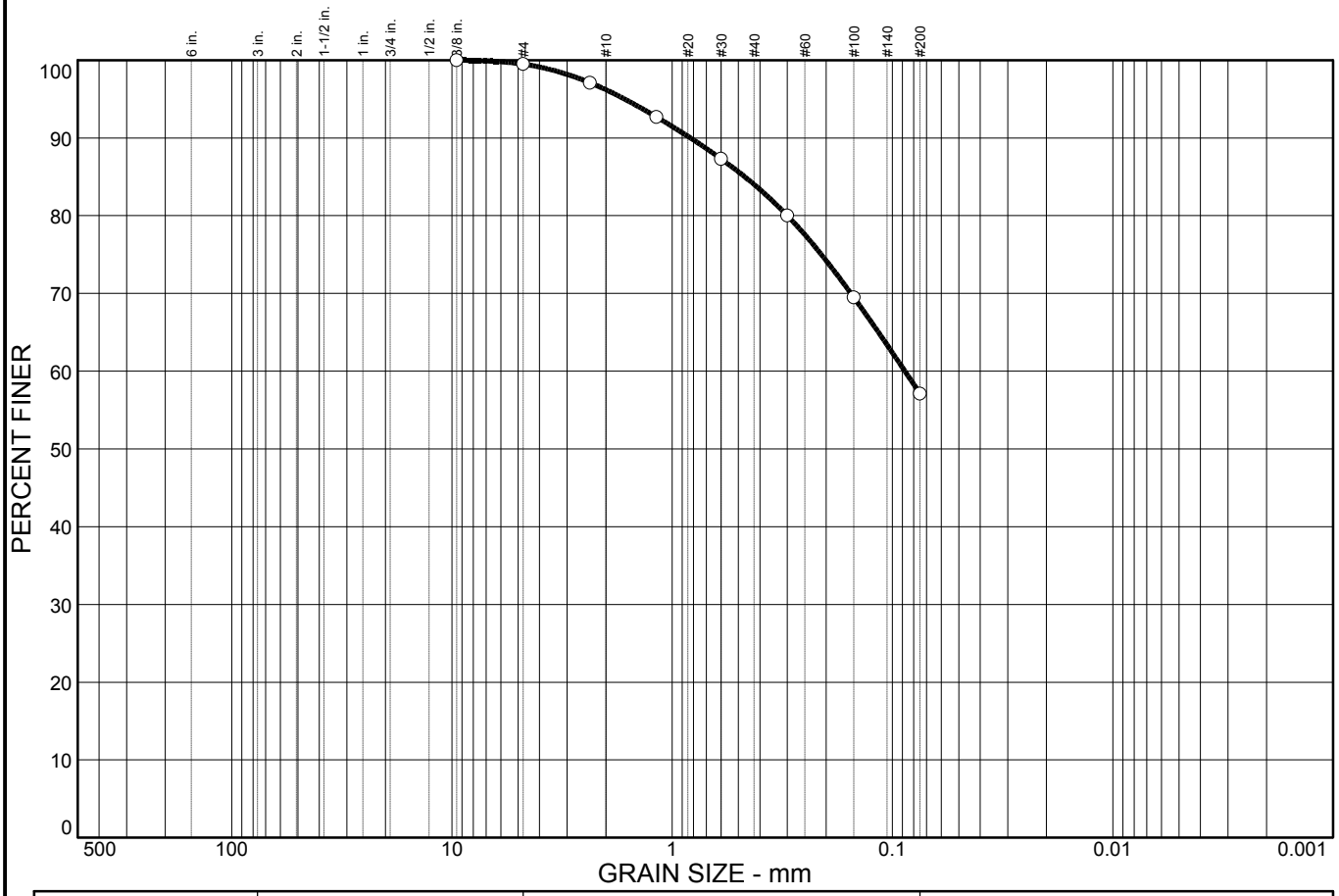
# APPENDIX C

## RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

| These Included: | To Determine: |
|---|---|
| Moisture Content (ASTM D2216) | Moisture contents representative of field conditions at the time the sample was taken. |
| Dry Density (ASTM D2937) | Dry unit weight of sample representative of in-situ or in-place undisturbed condition. |
| Grain-Size Distribution (ASTM D422) | Size and distribution of soil particles, i.e., sand, gravel and fines (silt and clay). |
| Atterberg Limits (ASTM D4318) | Determines the moisture contents where the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state (plastic limit) |
| Expansion Index (ASTM D4829) | Swell potential of soil with increases in moisture content. |
| Consolidation (ASTM 2435) | The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior. |
| Direct Shear (ASTM D3080) | Soil shearing strength under varying loads and/or moisture conditions. |
| Moisture-Density Relationship (ASTM D1557) | The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort. |
| R-Value (ASTM 2844) | The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load. |
| Sulfate Content (Cal Test 417) | Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type. |
| Chloride Content (Cal Test 422) | Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel. |
| Resistivity (ASTM G187) | The potential of the soil to corrode metal. |
| pH (CalTest 643) | The acidity or alkalinity of subgrade material. |

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.5 | 3.3 | 12.2 | 26.9 | 57.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.5 | | |
| #8 | 97.1 | | |
| #16 | 92.7 | | |
| #30 | 87.3 | | |
| #50 | 80.0 | | |
| #100 | 69.5 | | |
| #200 | 57.1 | | |

* (no specification provided)

**Material Description**

Sandy lean clay

**Atterberg Limits**

PL= 8 LL= 21 PI= 13

**Coefficients**

$D_{85}$= 0.469 $D_{60}$= 0.0879 $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= CL AASHTO=

**Remarks**

**Sample No.:** HA-1 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 25-30"

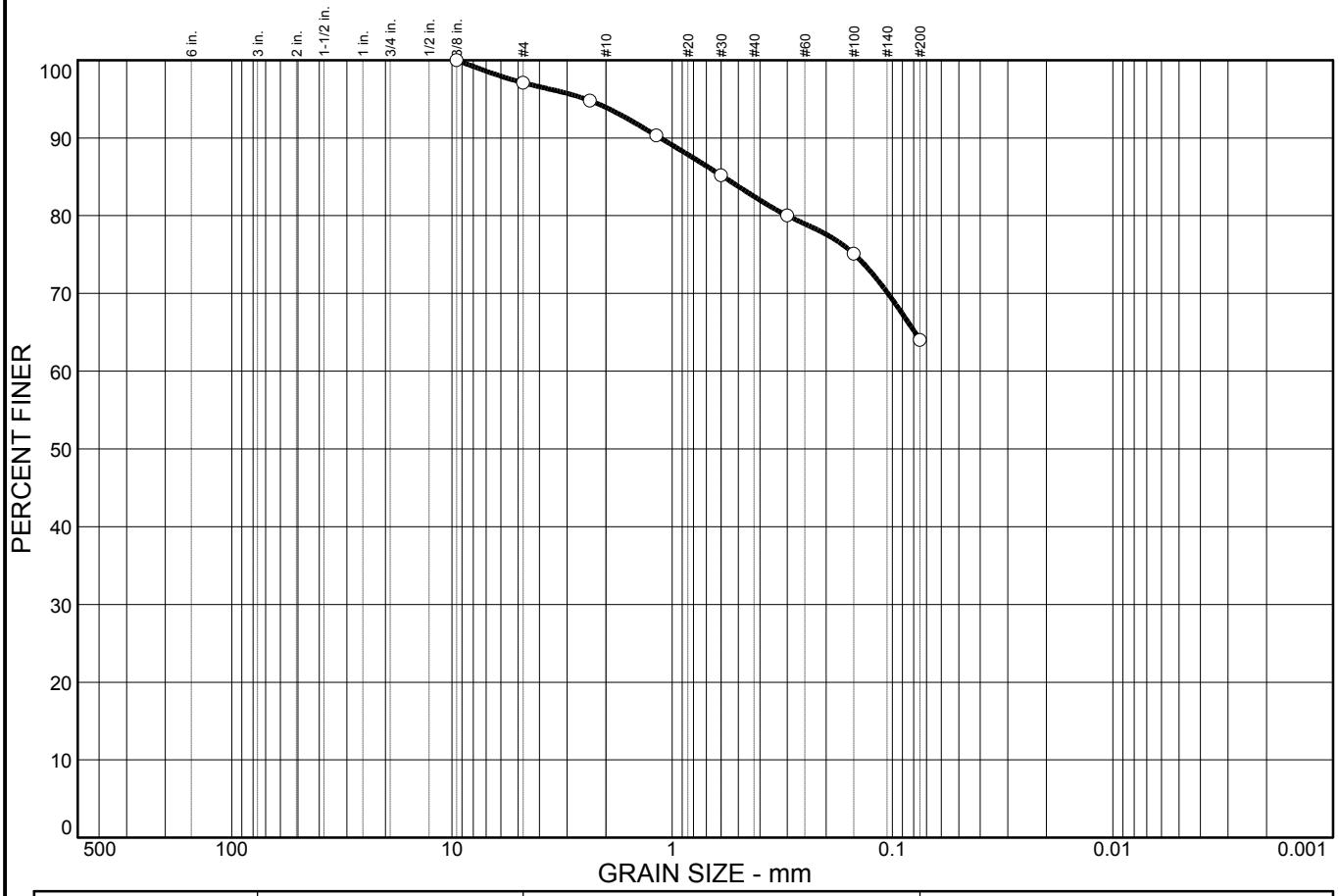
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 2.9 | 3.2 | 11.5 | 18.4 | 64.0 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 97.1 | | |
| #8 | 94.8 | | |
| #16 | 90.3 | | |
| #30 | 85.2 | | |
| #50 | 80.0 | | |
| #100 | 75.1 | | |
| #200 | 64.0 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 0.585 $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 1-2.5'

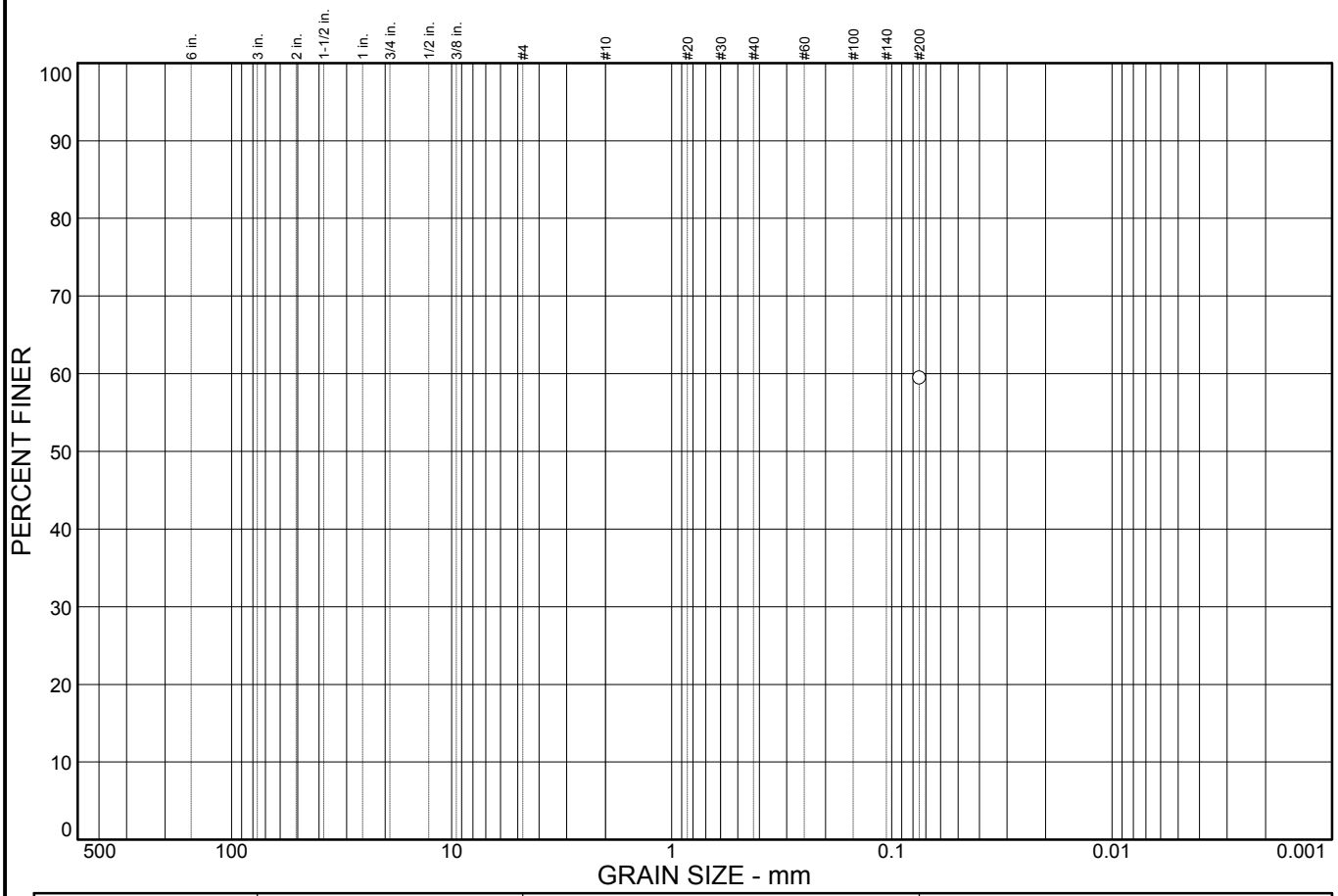
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| | | | | | | 59.5 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 59.5 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 5-6.5'

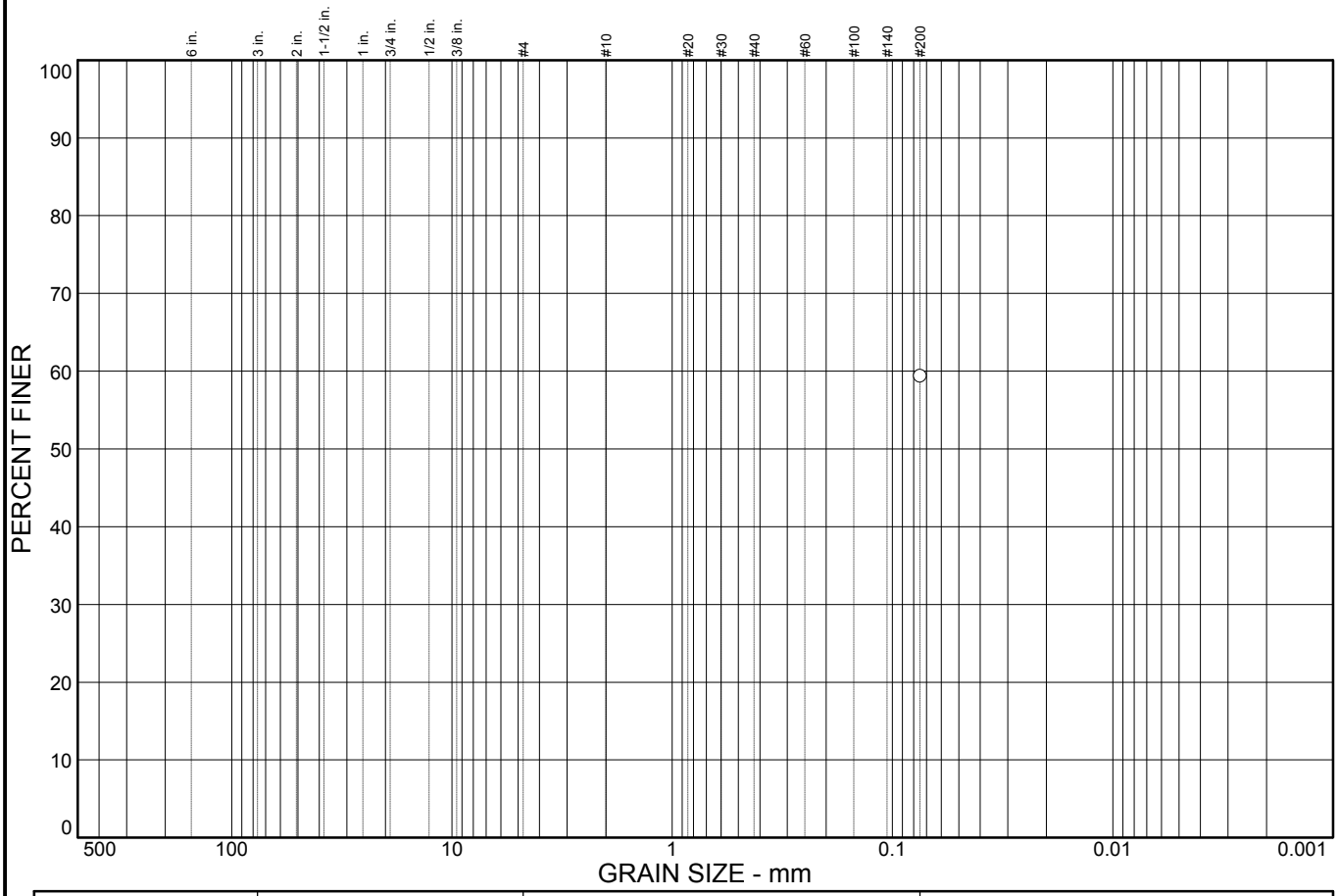
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| | | | | | | 59.4 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 59.4 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 10-11.5'

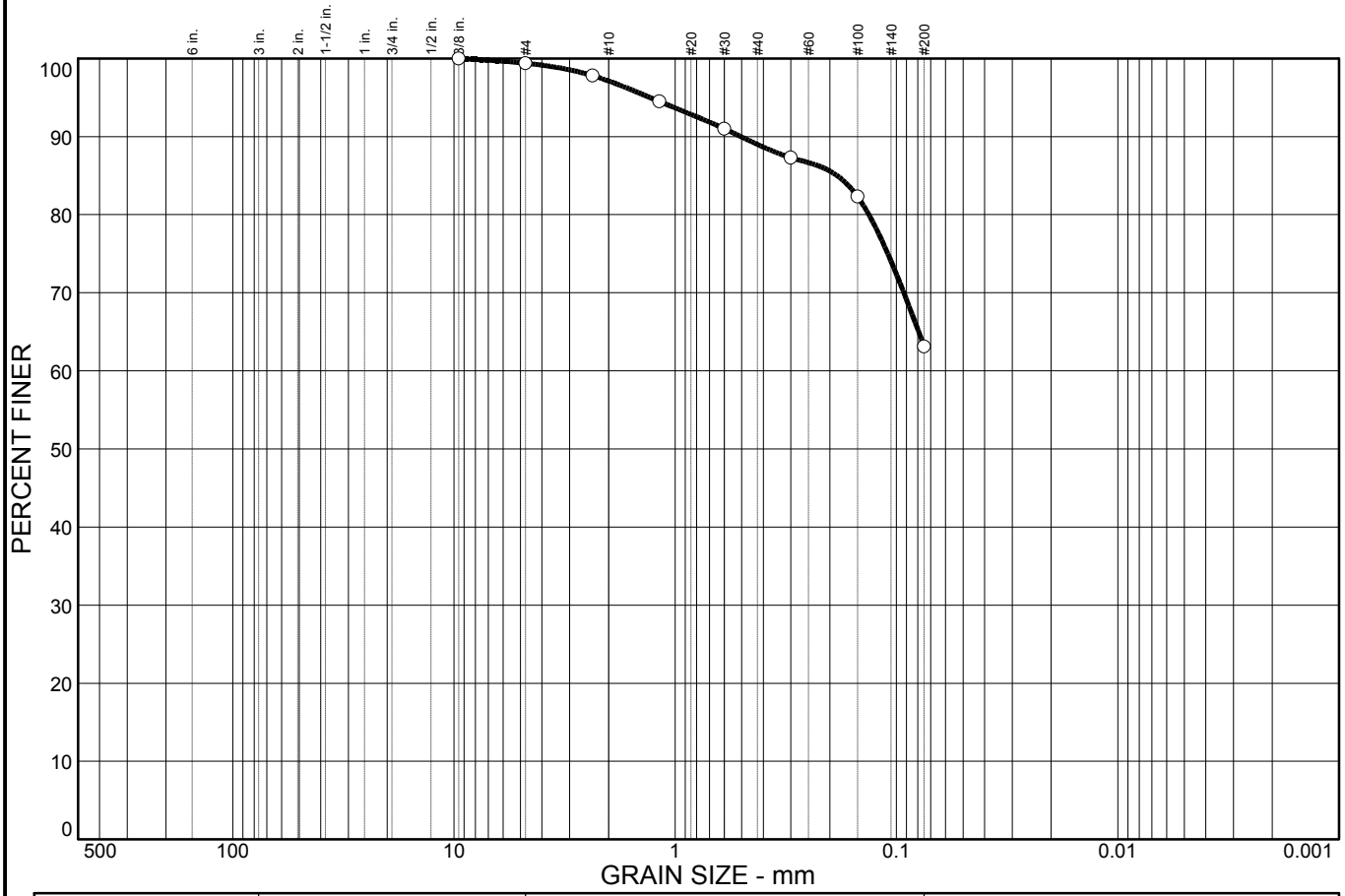
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.6 | 2.3 | 8.1 | 25.9 | 63.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.4 | | |
| #8 | 97.8 | | |
| #16 | 94.5 | | |
| #30 | 91.0 | | |
| #50 | 87.3 | | |
| #100 | 82.3 | | |
| #200 | 63.1 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 0.186 $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 13.5-15'

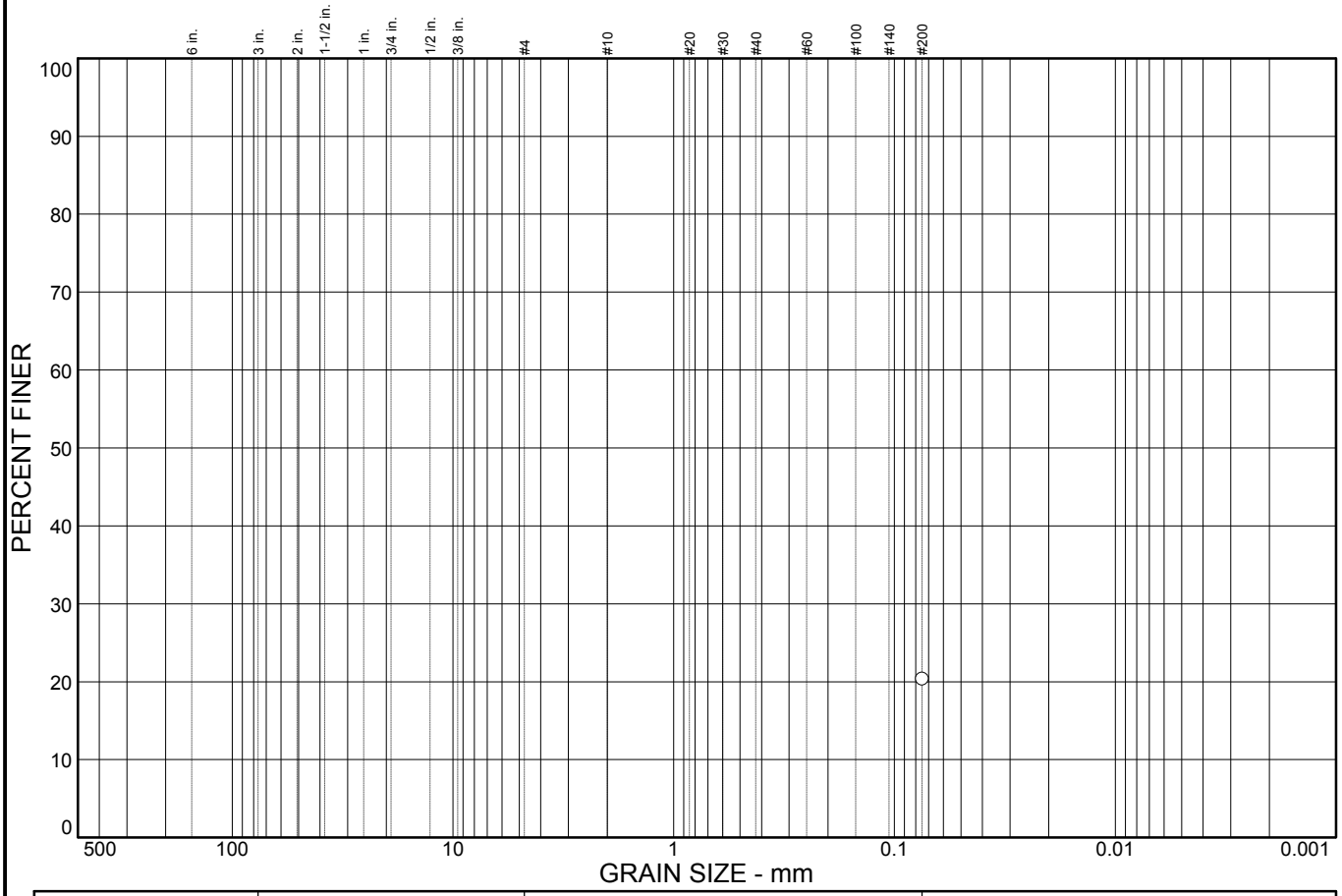
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| | | | | | | 20.4 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 20.4 | | |

* (no specification provided)

**Material Description**

Silty sand

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 15-16.5'

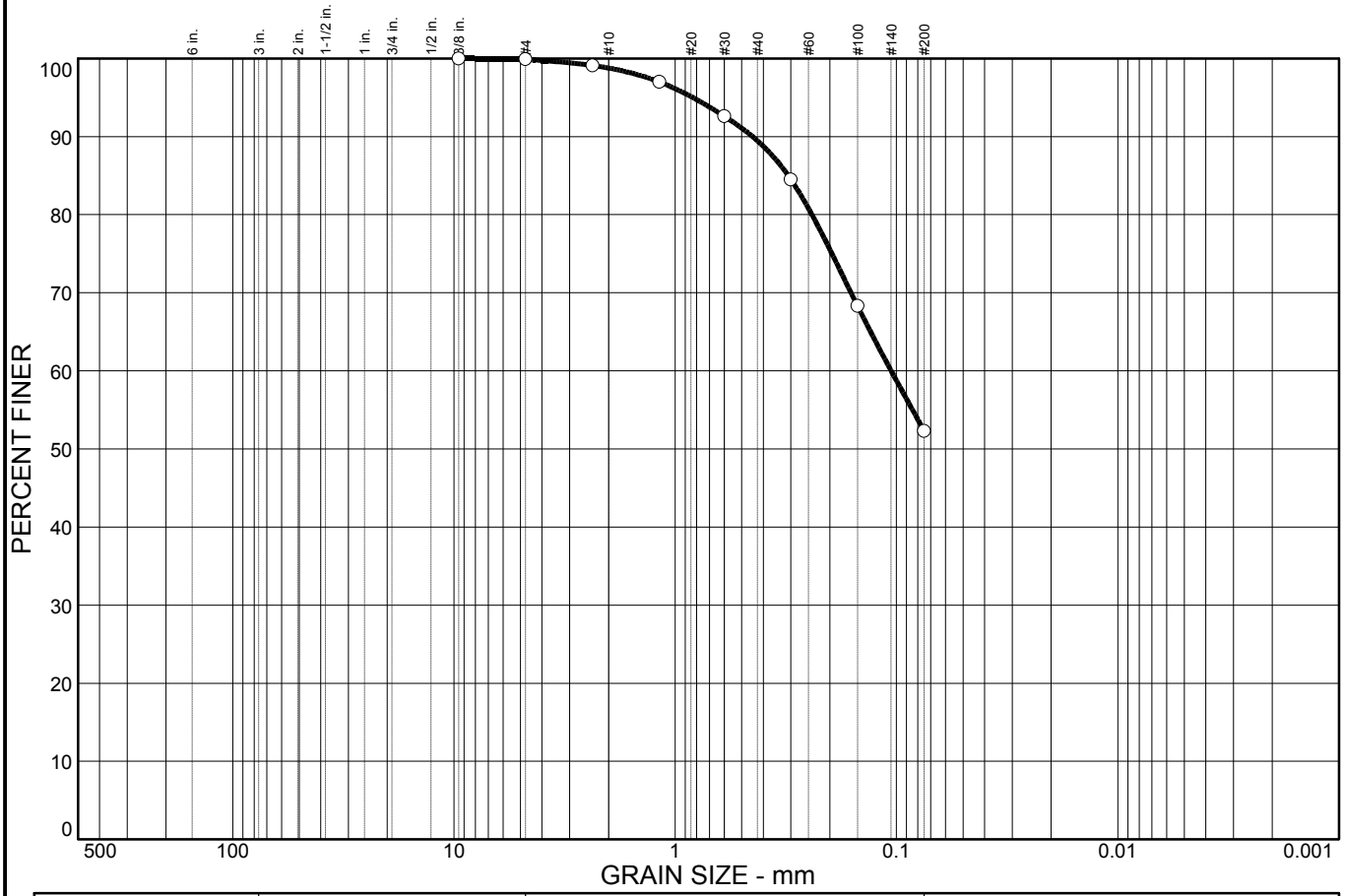
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.1 | 1.1 | 9.3 | 37.2 | 52.3 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.9 | | |
| #8 | 99.1 | | |
| #16 | 97.0 | | |
| #30 | 92.6 | | |
| #50 | 84.5 | | |
| #100 | 68.3 | | |
| #200 | 52.3 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 0.309 $D_{60}$= 0.106 $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 25-26.5'

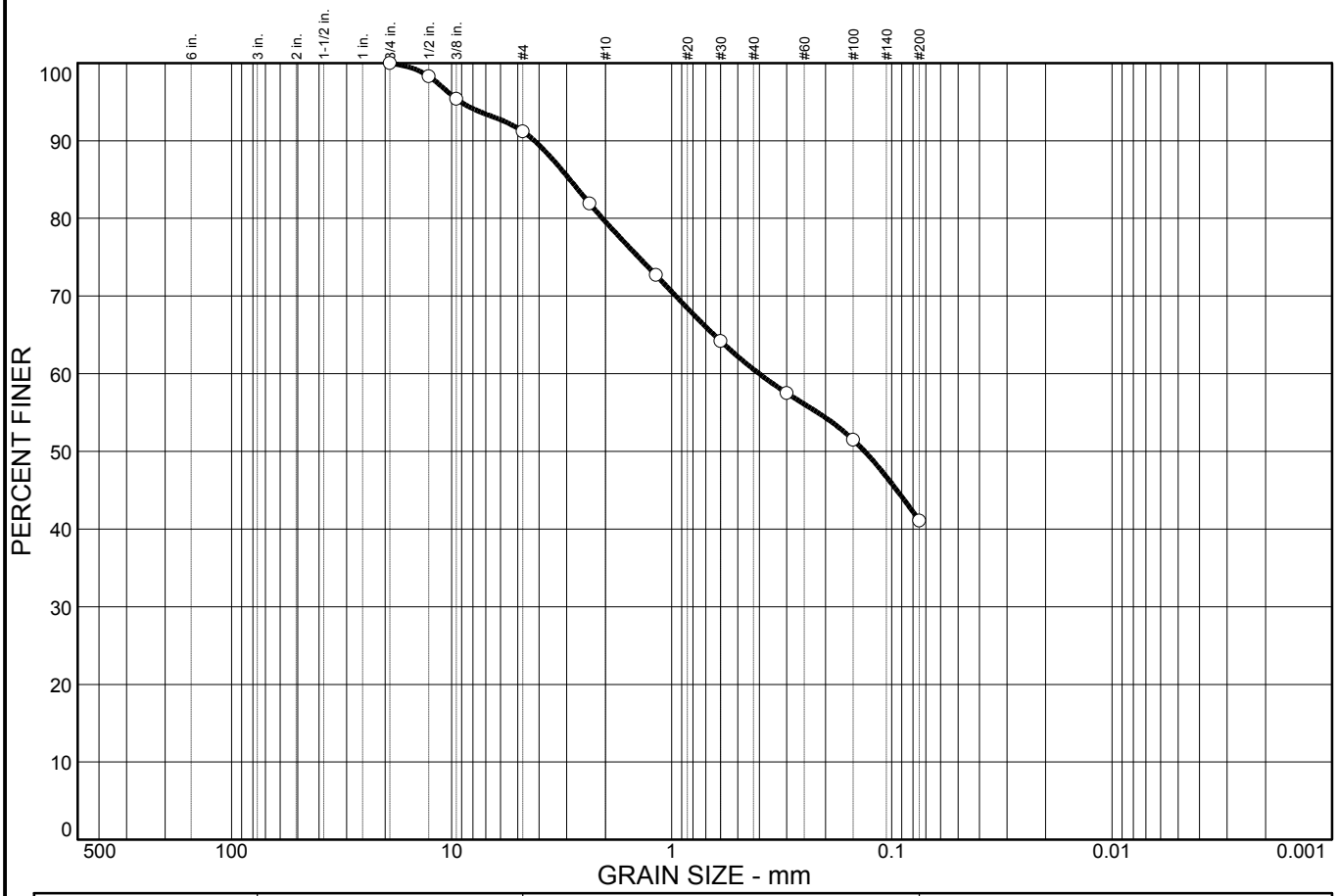
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 8.8 | 11.6 | 19.0 | 19.5 | 41.1 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/4 in. | 100.0 | | |
| 1/2 in. | 98.3 | | |
| 3/8 in. | 95.4 | | |
| #4 | 91.2 | | |
| #8 | 81.9 | | |
| #16 | 72.7 | | |
| #30 | 64.2 | | |
| #50 | 57.5 | | |
| #100 | 51.5 | | |
| #200 | 41.1 | | |

* (no specification provided)

**Material Description**

Clayey sand

**Atterberg Limits**

PL= 8 LL= 21 PI= 13

**Coefficients**

$D_{85}$= 2.91 $D_{60}$= 0.400 $D_{50}$= 0.133
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SC AASHTO=

**Remarks**

**Sample No.:** B-3 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 1-2.5'

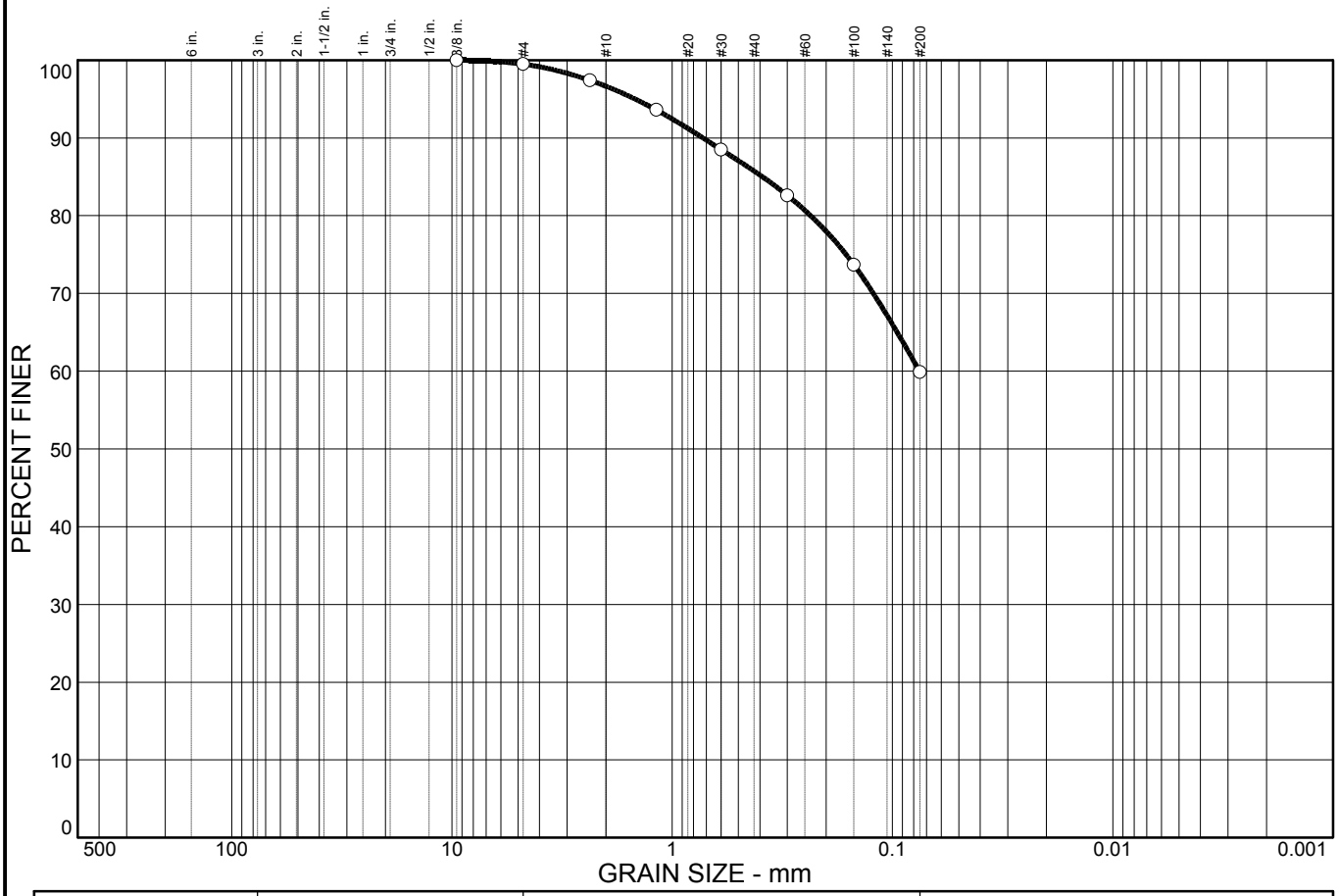
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.5 | 2.9 | 10.9 | 25.8 | 59.9 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.5 | | |
| #8 | 97.4 | | |
| #16 | 93.6 | | |
| #30 | 88.5 | | |
| #50 | 82.6 | | |
| #100 | 73.7 | | |
| #200 | 59.9 | | |

* (no specification provided)

**Material Description**

Sandy lean clay

**Atterberg Limits**

PL= 13 LL= 23 PI= 10

**Coefficients**

$D_{85}$= 0.390 $D_{60}$= 0.0753 $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= CL AASHTO=

**Remarks**

**Sample No.:** B-3 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 13.5-15'

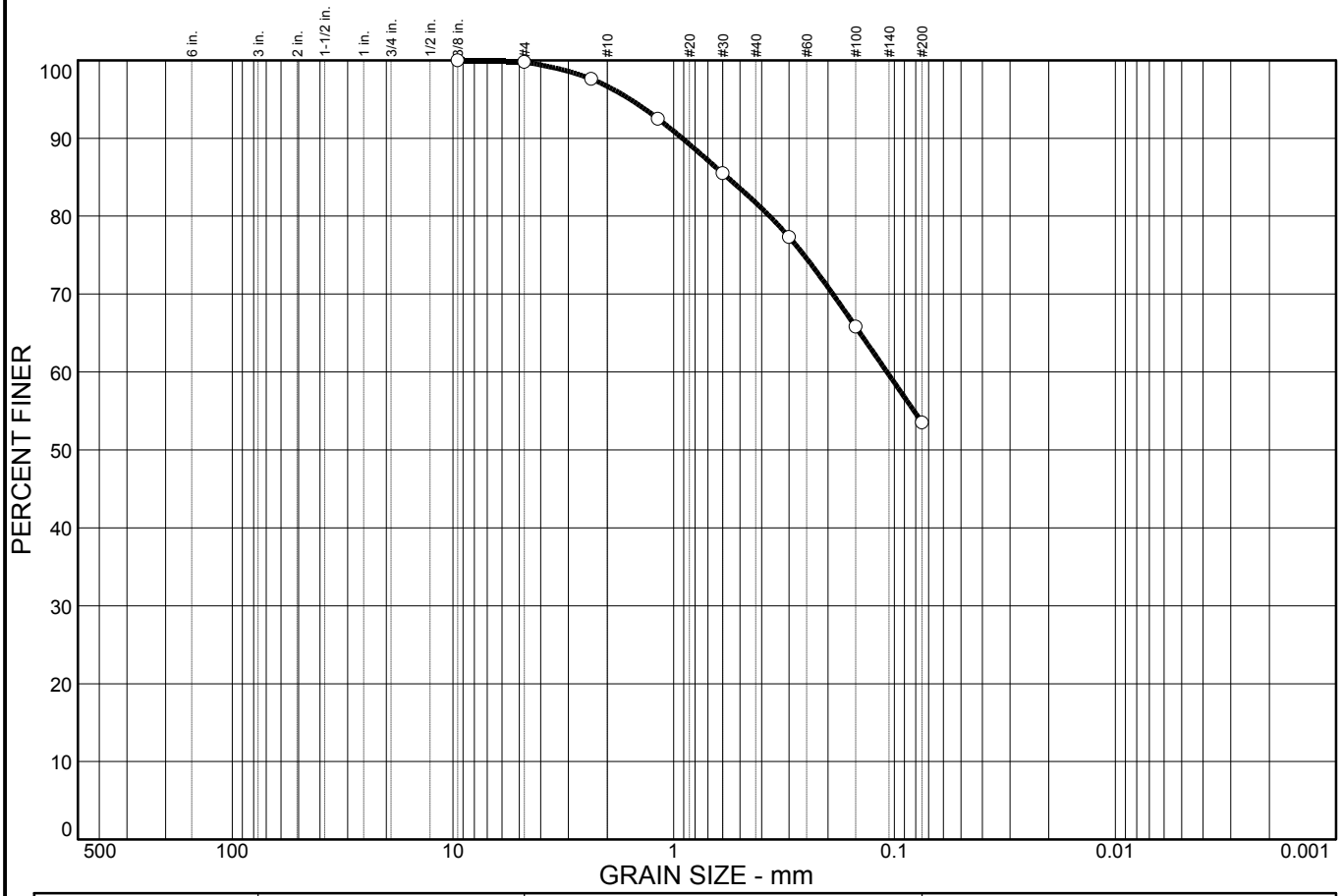
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.2 | 3.1 | 15.0 | 28.2 | 53.5 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.8 | | |
| #8 | 97.6 | | |
| #16 | 92.5 | | |
| #30 | 85.5 | | |
| #50 | 77.3 | | |
| #100 | 65.8 | | |
| #200 | 53.5 | | |

* (no specification provided)

**Material Description**

Sandy lean clay

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 0.573 $D_{60}$= 0.108 $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= CL AASHTO=

**Remarks**

**Sample No.:** B-3 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 23.5-25'

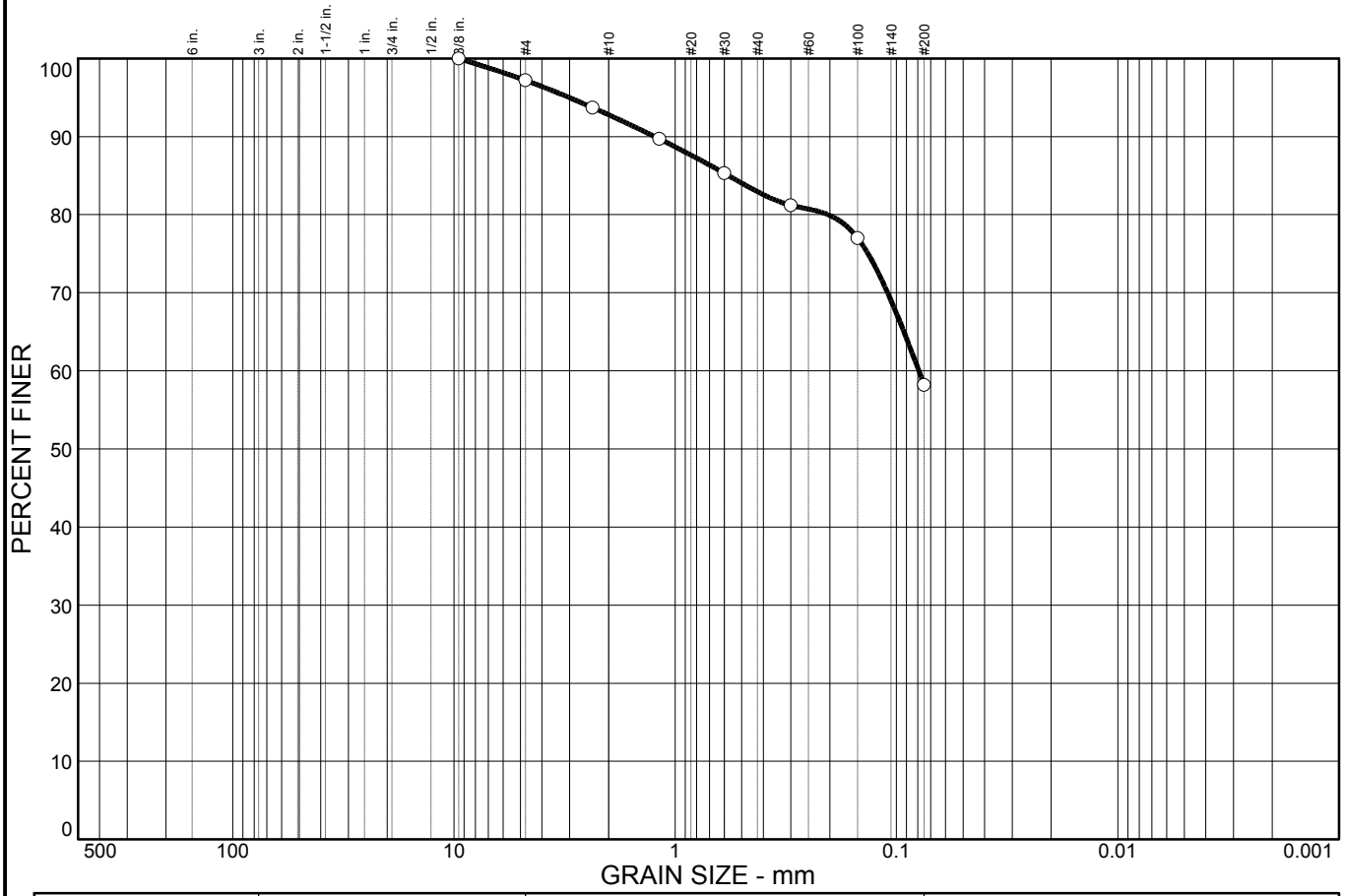
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 2.8 | 4.4 | 9.8 | 24.8 | 58.2 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 97.2 | | |
| #8 | 93.7 | | |
| #16 | 89.7 | | |
| #30 | 85.3 | | |
| #50 | 81.2 | | |
| #100 | 77.0 | | |
| #200 | 58.2 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 0.575 $D_{60}$= 0.0792 $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-4 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 5-6.5'

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 1.2 | 7.0 | 21.0 | 27.5 | 43.3 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 98.8 | | |
| #8 | 93.7 | | |
| #16 | 85.0 | | |
| #30 | 75.8 | | |
| #50 | 65.4 | | |
| #100 | 53.4 | | |
| #200 | 43.3 | | |

* (no specification provided)

**Material Description**

Silty sand

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 1.18 $D_{60}$= 0.220 $D_{50}$= 0.121
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

**Sample No.:** B-5 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 5-6.5'

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**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| 0.0 | 0.0 | 0.8 | 7.1 | 25.0 | 21.8 | 45.3 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| 3/8 in. | 100.0 | | |
| #4 | 99.2 | | |
| #8 | 94.1 | | |
| #16 | 84.1 | | |
| #30 | 72.5 | | |
| #50 | 62.4 | | |
| #100 | 54.4 | | |
| #200 | 45.3 | | |

* (no specification provided)

**Material Description**

Clayey sand

**Atterberg Limits**

PL= 11 LL= 21 PI= 10

**Coefficients**

$D_{85}$= 1.24 $D_{60}$= 0.245 $D_{50}$= 0.106
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SC AASHTO=

**Remarks**

**Sample No.:** B-5 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 18.5-20'

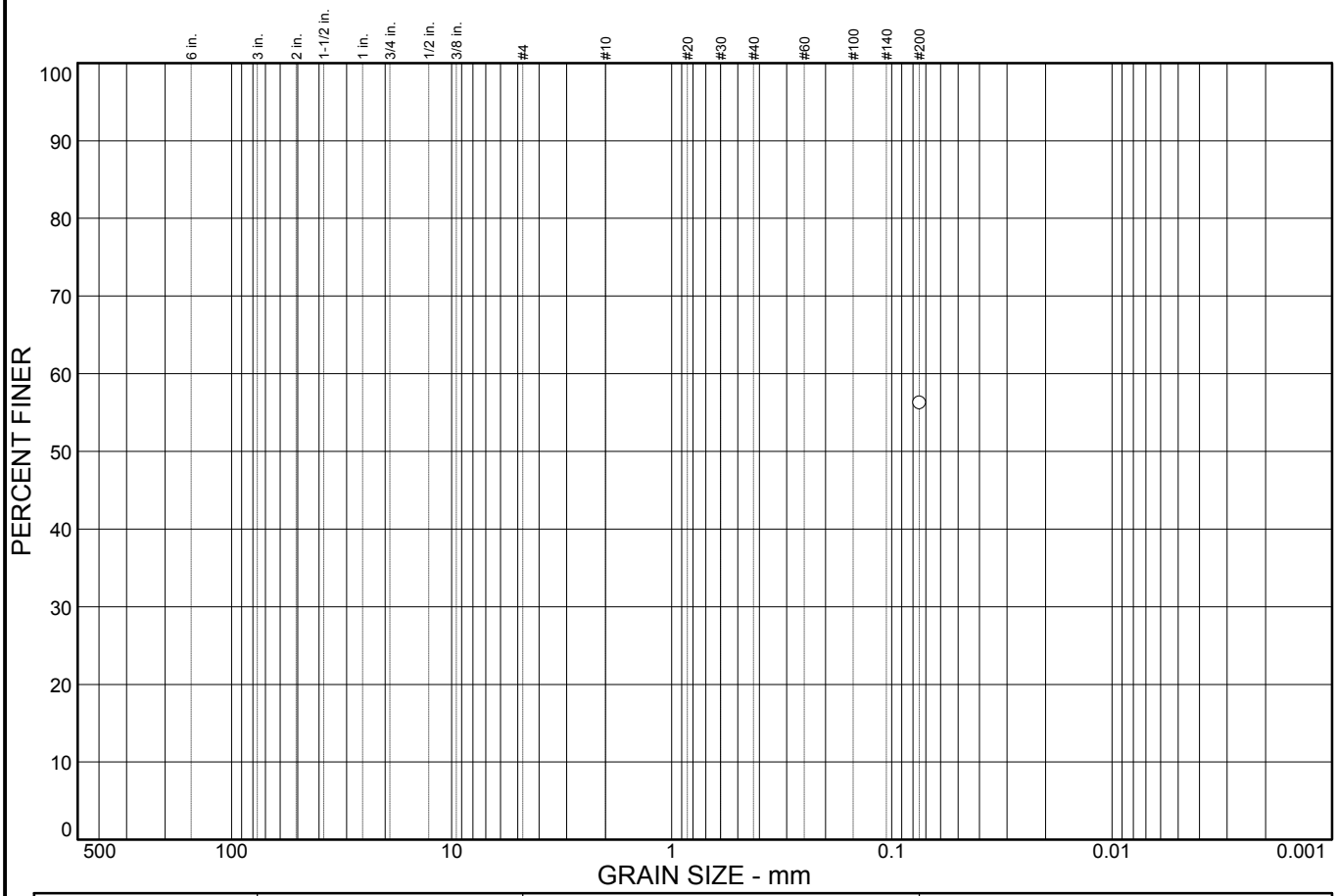
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| | | | | | | 56.3 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 56.3 | | |

* (no specification provided)

**Material Description**

Sandy silt

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= ML AASHTO=

**Remarks**

**Sample No.:** B-6 / P-1 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 8.5-10'

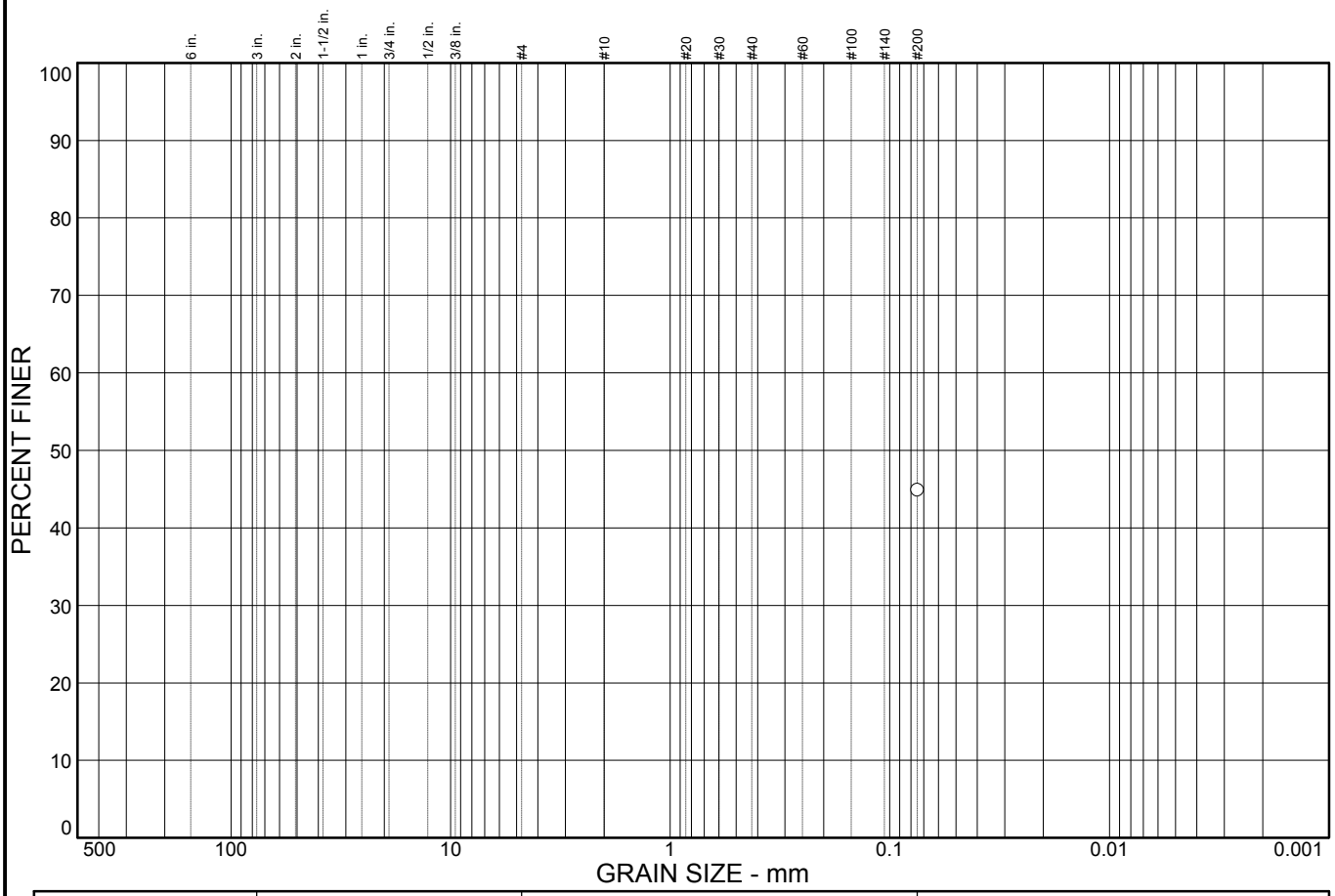
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# Particle Size Distribution Report




| % COBBLES | % GRAVEL | | % SAND | | | % FINES | |
|---|---|---|---|---|---|---|---|
| | CRS. | FINE | CRS. | MEDIUM | FINE | SILT | CLAY |
| | | | | | | 44.9 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| #200 | 44.9 | | |

* (no specification provided)

**Material Description**

Silty sand

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= $D_{60}$= $D_{50}$=
$D_{30}$= $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

**Sample No.:** P-2 **Source of Sample:** **Date:** 8/20/25
**Location:** **Elev./Depth:** 8.5-10'

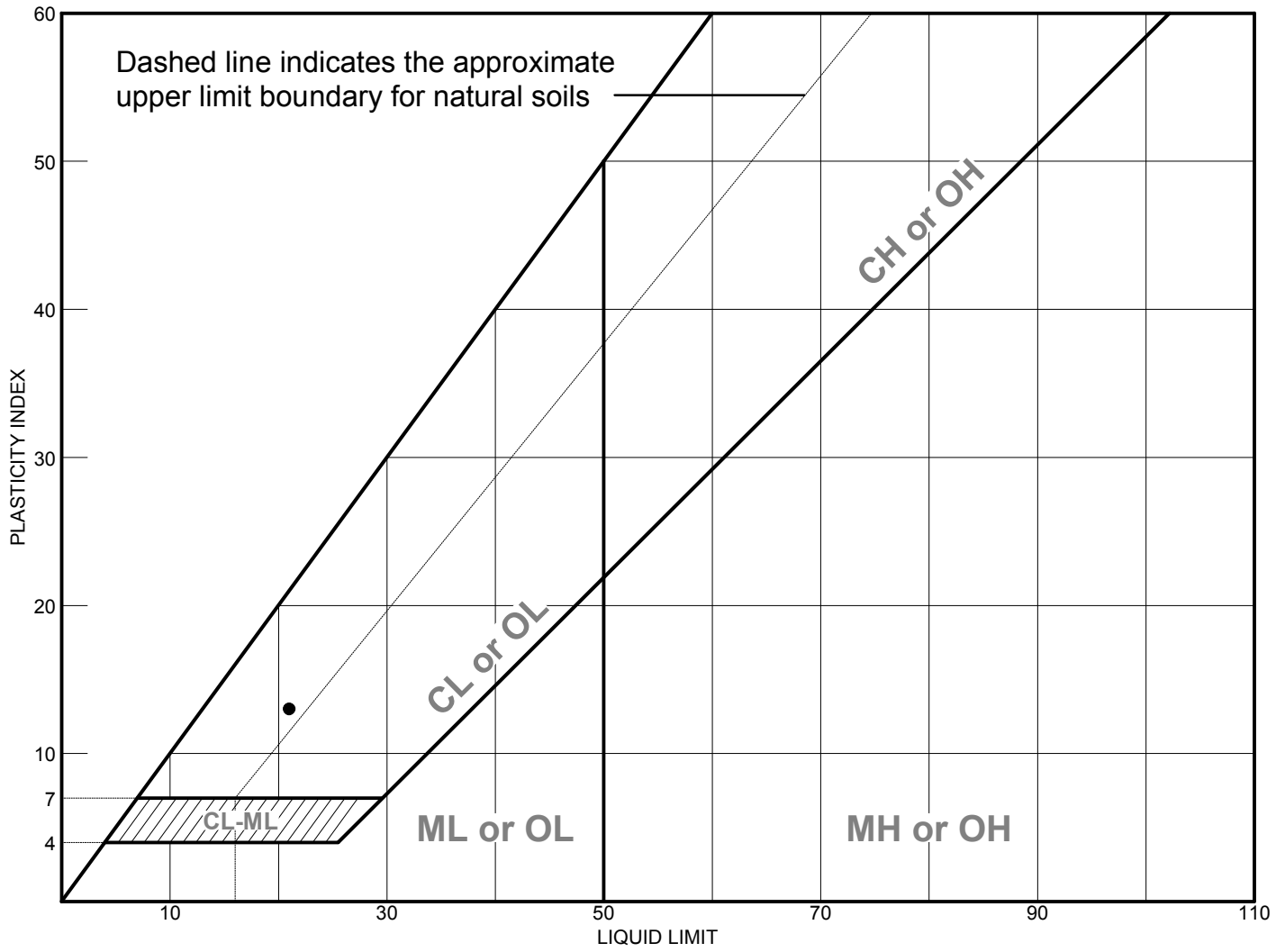
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

**Project No:** H01602.01 **Figure**

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Sandy lean clay | 21 | 8 | 13 | 84.0 | 57.1 | CL |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

● **Source:** **Sample No.:** HA-1 **Elev./Depth:** 25-30"

**Remarks:**

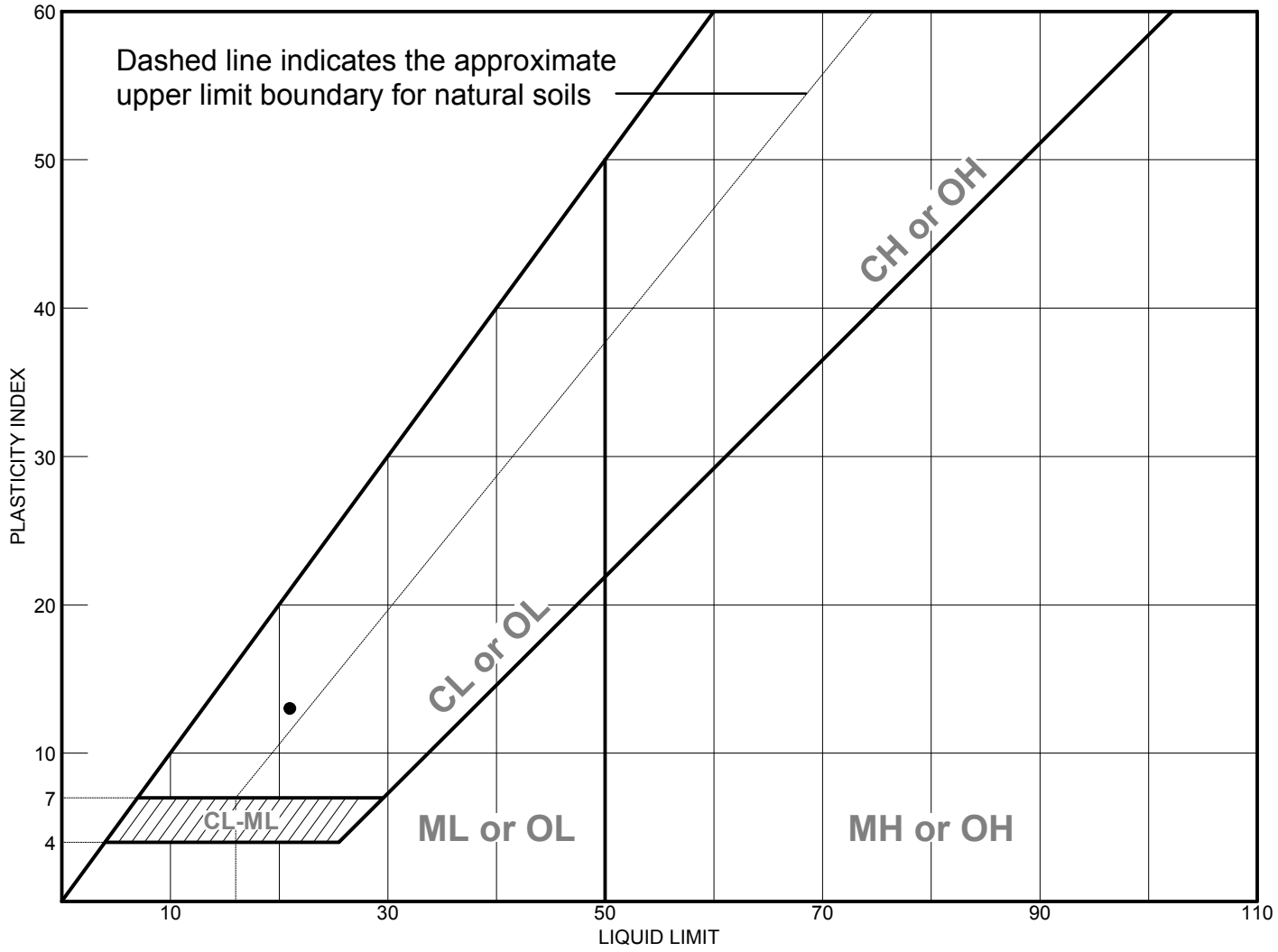
●

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**Fresno, CA**

**Figure**

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Clayey sand | 21 | 8 | 13 | 60.6 | 41.1 | SC |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

● **Source:** **Sample No.:** B-3 **Elev./Depth:** 1-2.5'

**Remarks:**

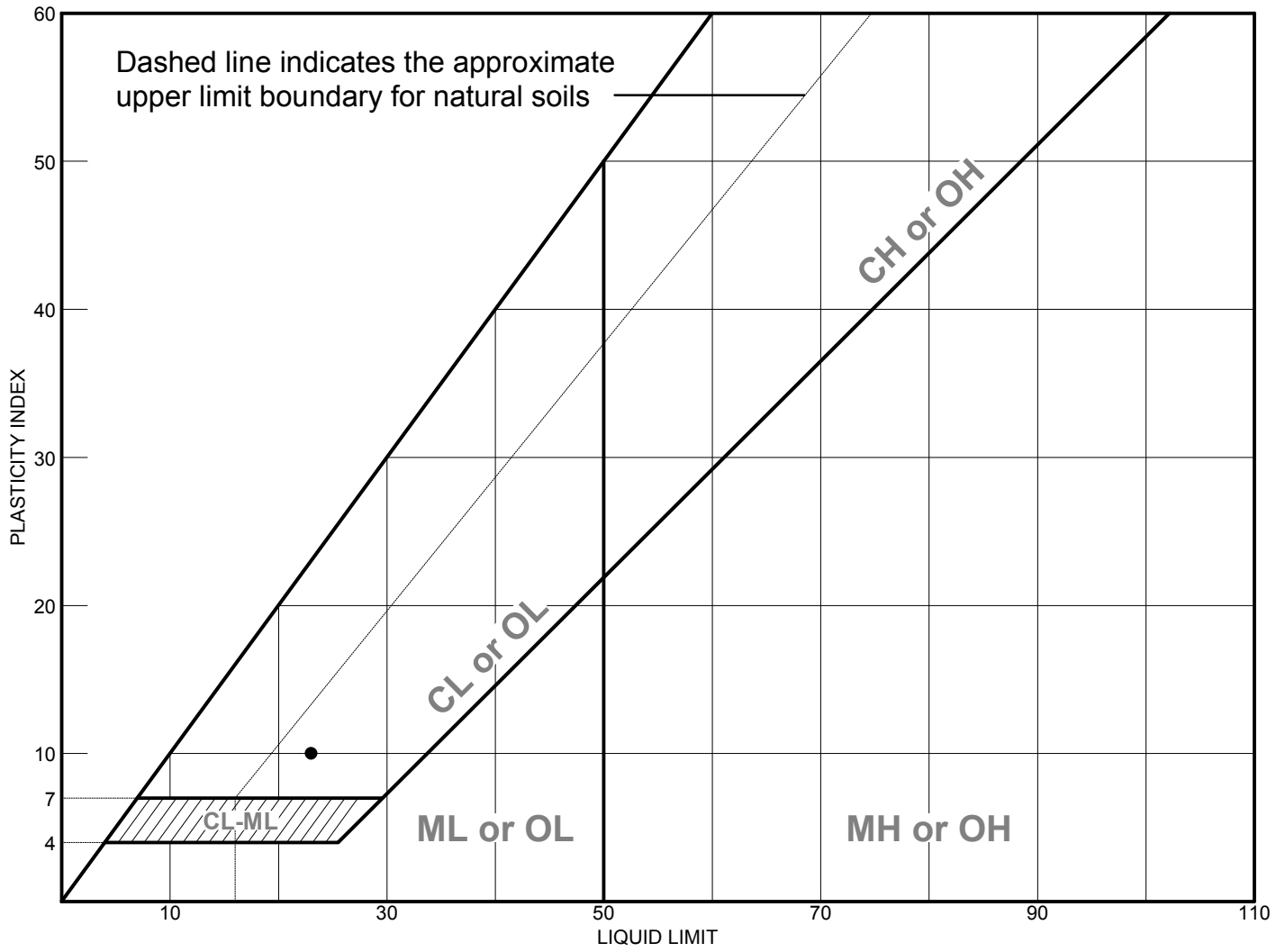
●

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Sandy lean clay | 23 | 13 | 10 | 85.7 | | CL |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

● **Source:** **Sample No.:** B-3 **Elev./Depth:** 13.5-15'

**Remarks:**

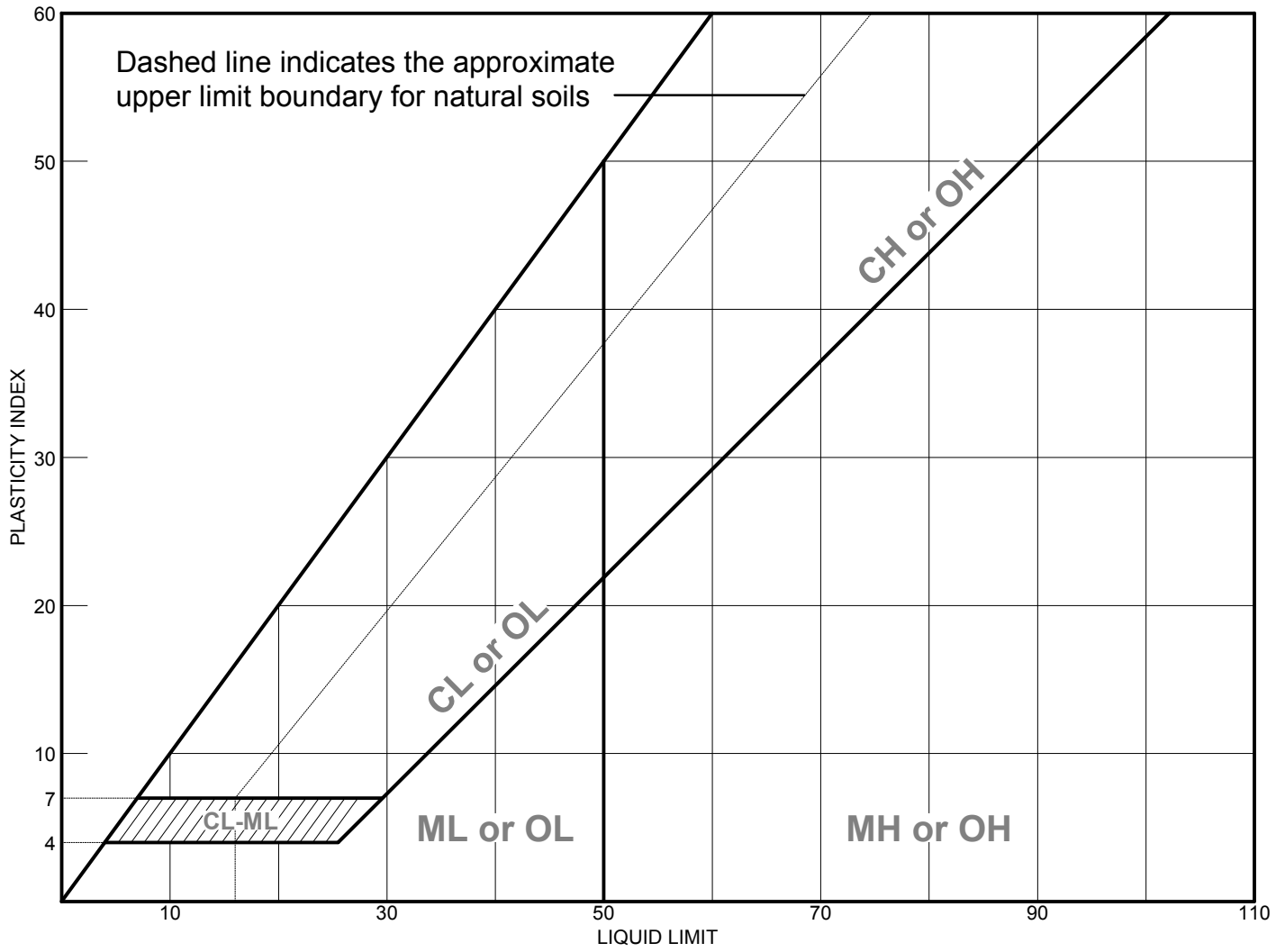
●

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Silty sand | NV | NP | NP | | | SM |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 7-8.5'

**Remarks:**

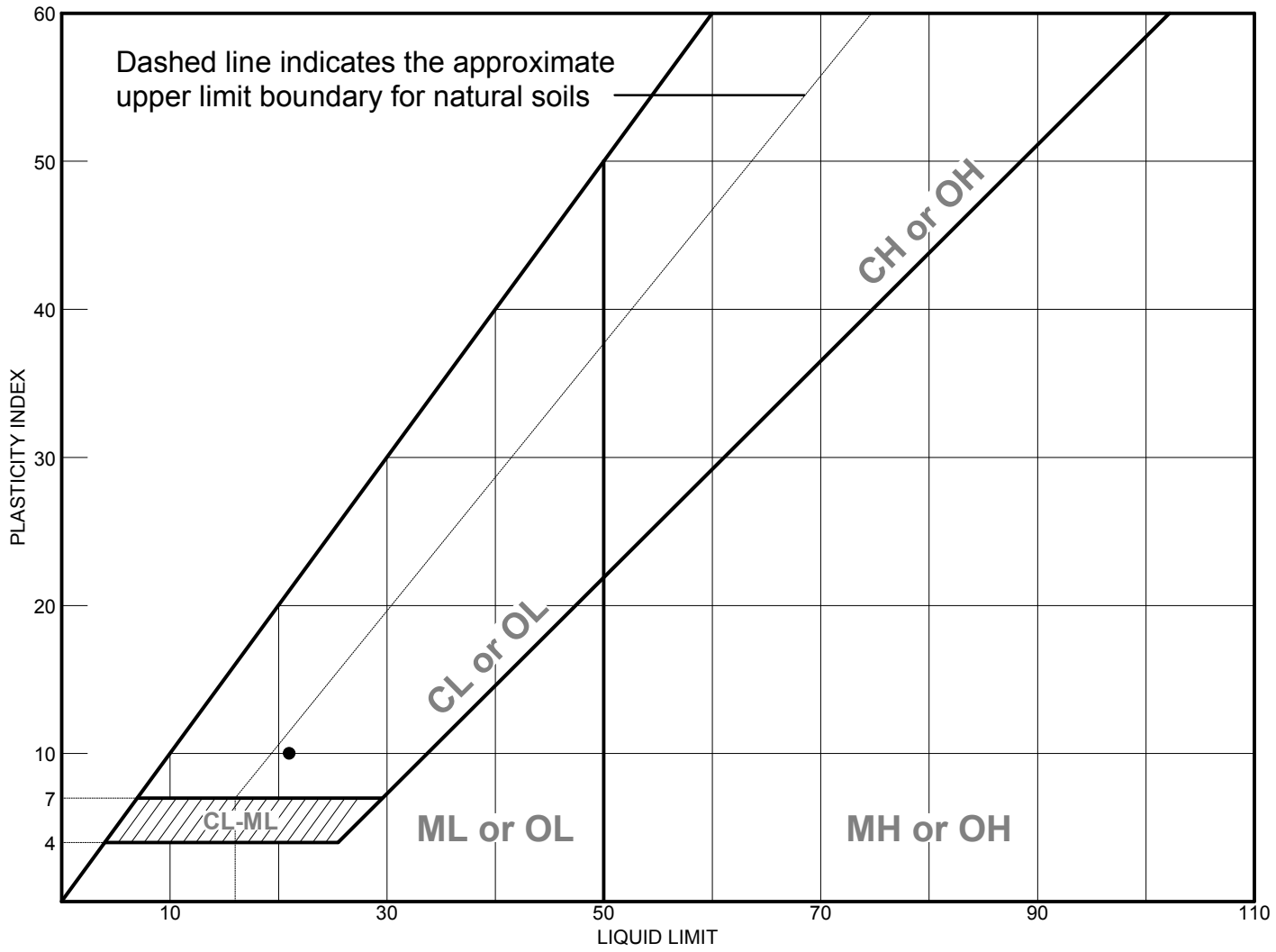
●

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Clayey sand | 21 | 11 | 10 | 67.1 | 45.3 | SC |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

● **Source:** **Sample No.:** B-5 **Elev./Depth:** 18.5-20'

**Remarks:**

●

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

MOORE TWINING ASSOCIATES, INC.

## EXPANSION INDEX TEST, ASTM D4829

| | | | |
|---|---|---|---|
| MTA PROJECT NAME: | Riverside PD Replacement HQ | REPORT DATE: | 9/25/2025 |
| | | TEST DATE: | 9/3/2025 |
| MTA PROJECT NO.: | H01602.01 | | |
| SAMPLE I.D.: | B-3 @ 1-5' | | |
| SAMPLED BY: | SR | | |
| SAMPLE DATE: | 8/20/2025 | TESTED BY: | CJ |
| MATERIALS DESCRIPTION: | Clayey sand/Silty sand | | |
| % PASSING # 4 SIEVE | 100 | | |

| Initial Moisture Determination: | | Final Moisture Determination: | |
|---|---|---|---|
| Pan + Wet Soil Wt., gm | 250.0 | Wet Soil Wt., lbs | 0.9744 |
| Pan + Dry Soil Wt., gm | 232.4 | Dry Soil Wt., lbs | 0.8718 |
| Pan Wt., gm | 0.0 | | |
| Initial % Moisture Content | 7.6 | Final % Moisture Content | 11.8 |

| **Initial Expansion Data:** | | **Final Expansion Data:** | |
|---|---|---|---|
| Ring + Sample Wt., lbs | 0.9378 | Ring + Sample Wt., lbs | 0.9744 |
| Ring Wt., lbs | 0.0000 | Ring Wt., lbs | 0.0000 |
| Remolded Wt., lbs | 0.9378 | Remolded Wt., lbs | 0.9744 |
| Remolded Wet Density, pcf | 129.0 | Remolded Wet Density, pcf | 134.6 |
| Remolded Dry Density, pcf | 119.9 | Remolded Dry Density, pcf | 120.4 |

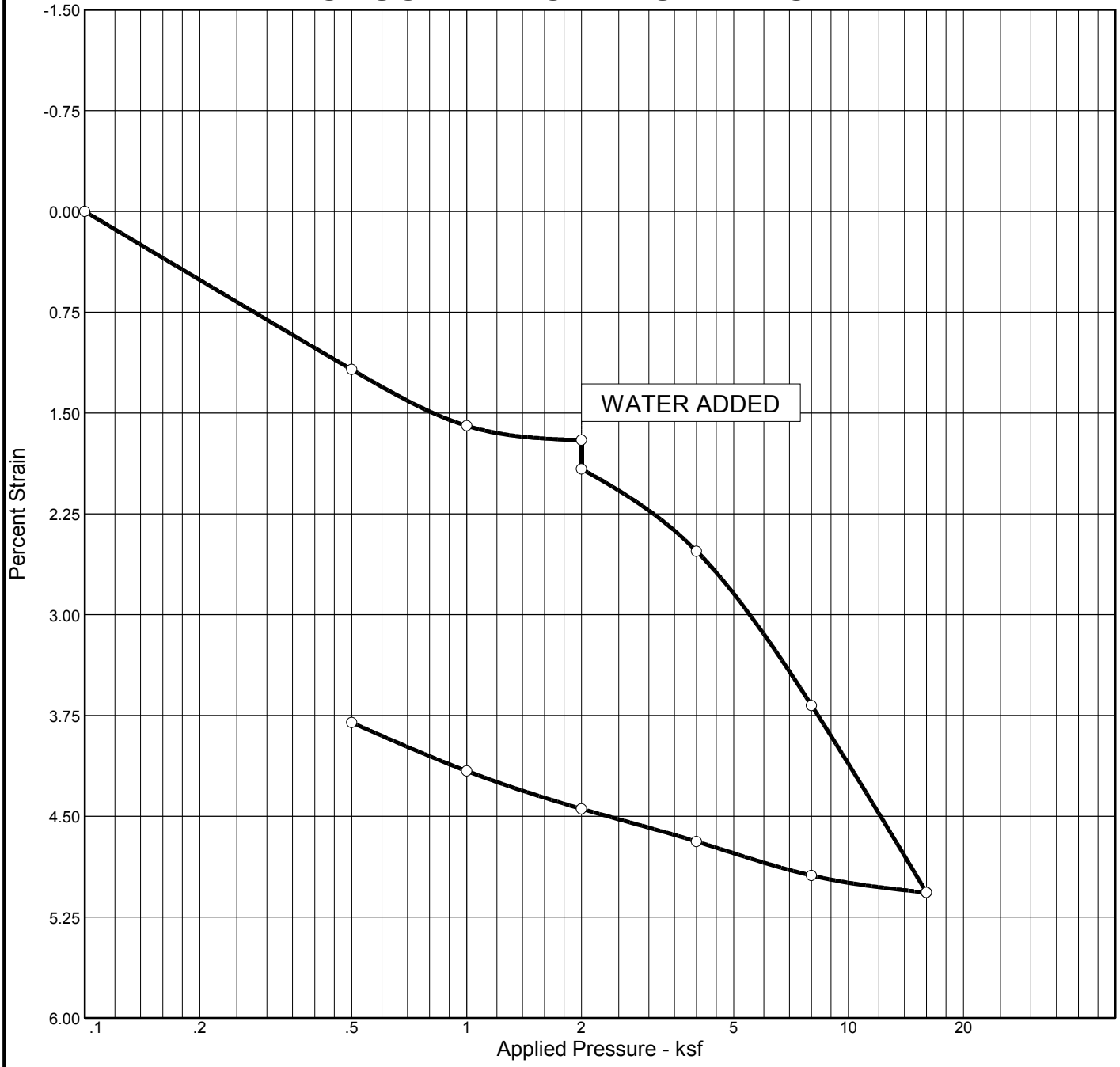
| Expansion Data: | | Initial Volume | Final Volume |
|---|---|---|---|
| | | 0.00727222 | 0.00724 |
| Initial Gage Reading, in: | 0.0408 | | |
| Final Gage Reading, in: | 0.0364 | | |
| Expansion, in: | -0.0044 | | |
| **Expansion Index** | 0 | **Comments:** | **Very Low Expansion Potential** |

### Classification of Expansive Soils. (Table No.1 From ASTM D4829)

| Expansion Index | Potential Expansion |
|---|---|
| 0-20 | Very Low |
| 21-50 | Low |
| 51-90 | Medium |
| 91-130 | High |
| >130 | Very High |


PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Clpse. % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 63.0 % | 9.4 % | 118.5 | | | 2.65 | | 4.84 | 0.06 | 0.01 | | 0.2 | 0.396 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Silty sand | SM | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

**Source:** **Sample No.:** B-2 **Elev./Depth:** 20-21.5'

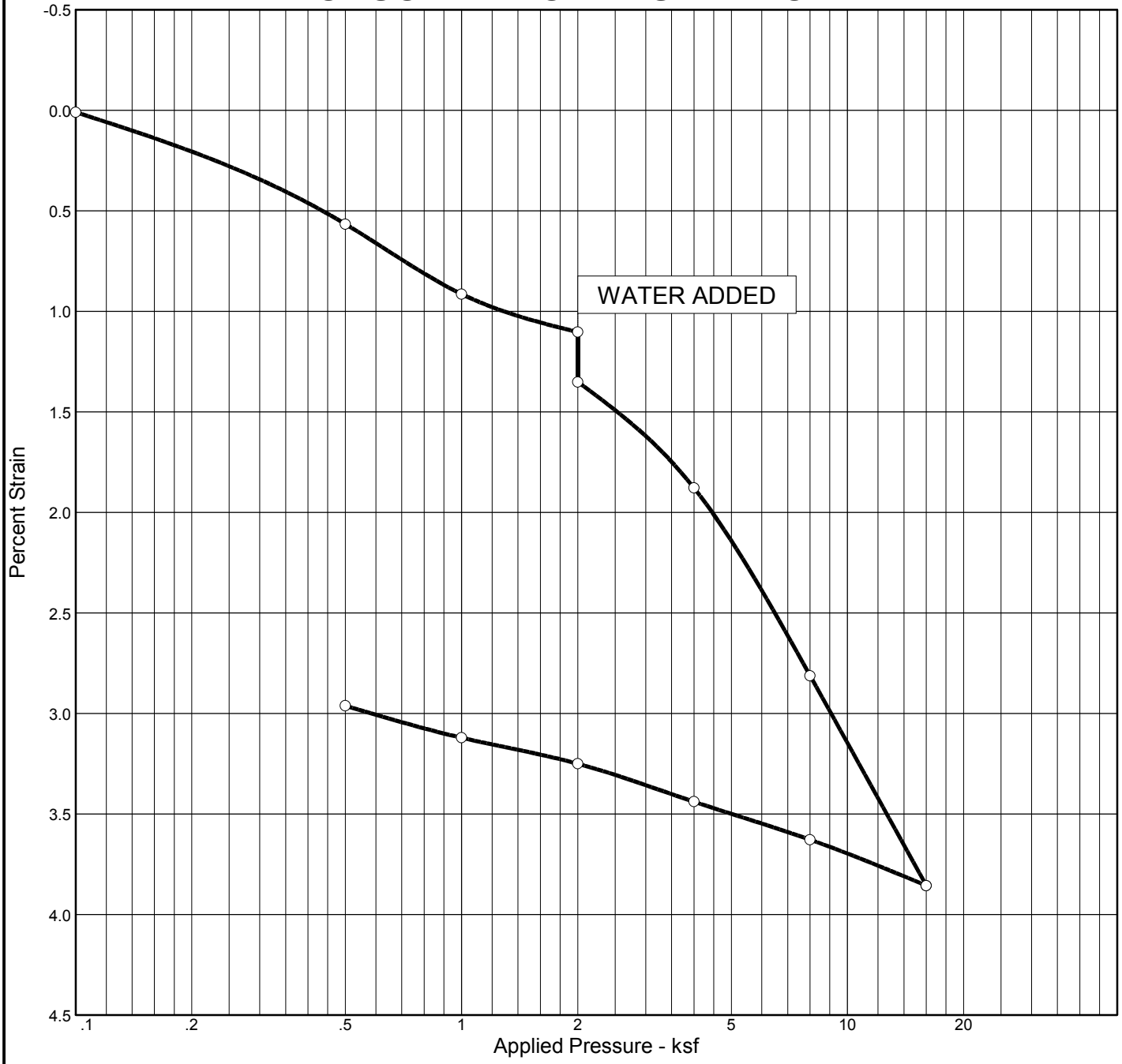
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Remarks:**

**Figure**

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Clpse. % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 44.7 % | 7.7 % | 113.5 | 21 | 13 | 2.65 | | 4.48 | 0.05 | 0.01 | | 0.3 | 0.458 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Clayey sand | SC | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

**Source:** **Sample No.:** B-3 **Elev./Depth:** 1-2.5'

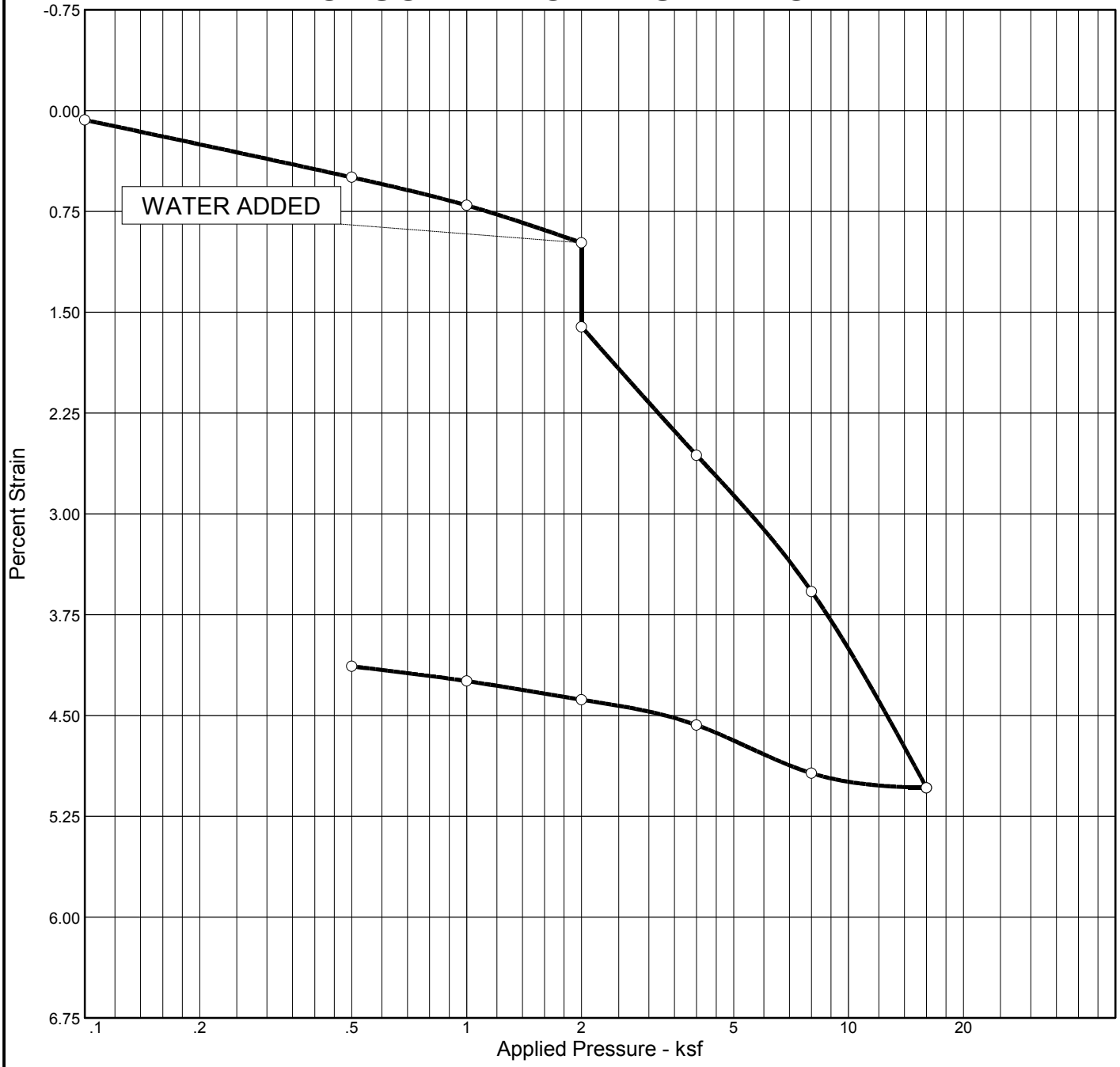
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Remarks:**

**Figure**

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Clpse. % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 26.9 % | 4.4 % | 115.6 | | | 2.65 | | 8.37 | 0.07 | 0.01 | | 0.6 | 0.431 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Silty sand | SM | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

**Source:** **Sample No.:** B-3 **Elev./Depth:** 8.5-10'

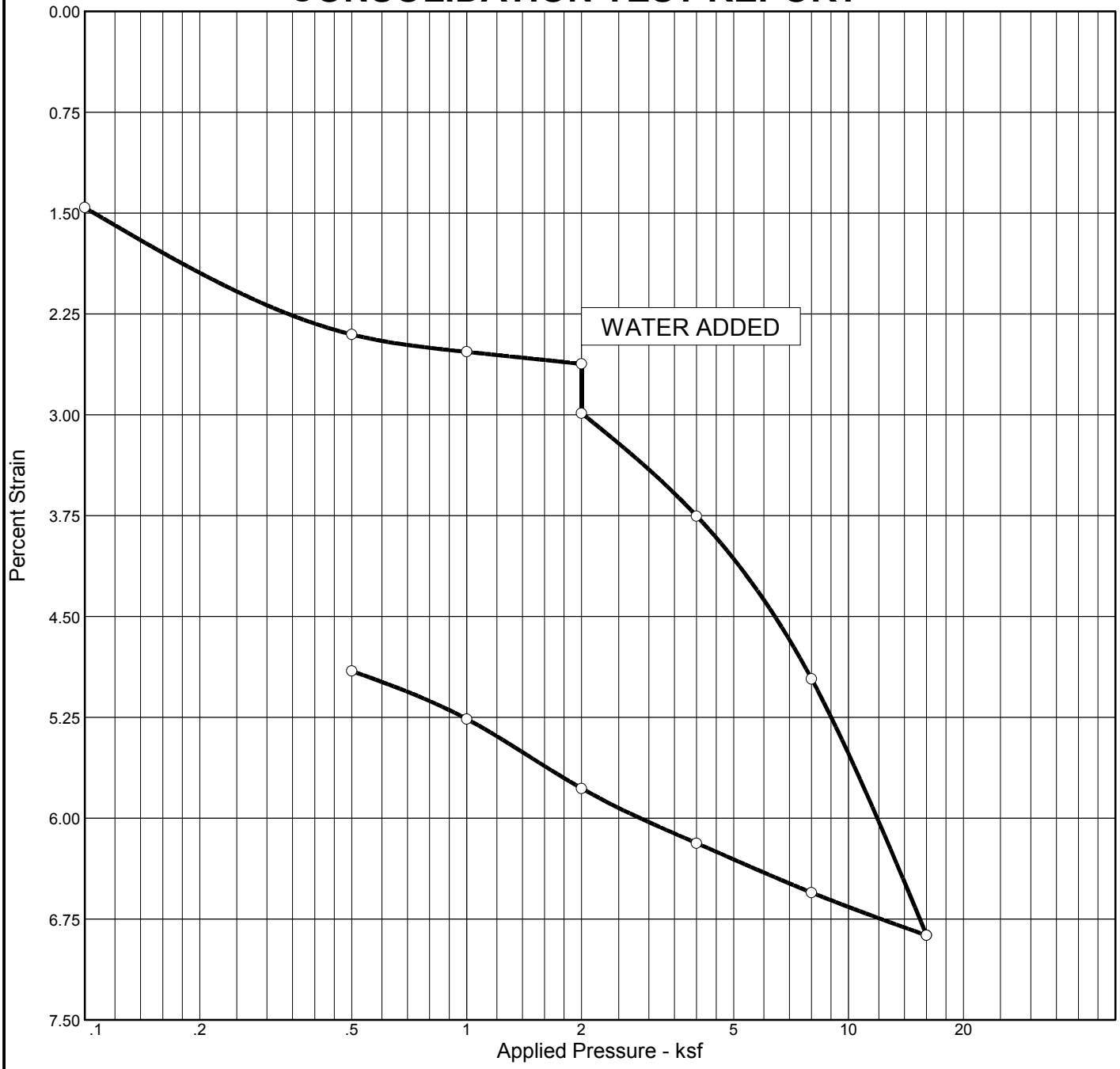
**Remarks:**

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Clpse. % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 94.9 % | 17.6 % | 110.8 | | | 2.65 | | 7.77 | 0.10 | 0.02 | | 0.4 | 0.493 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Sandy lean clay | CL | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

**Source:** **Sample No.:** B-3 **Elev./Depth:** 18.5-20'

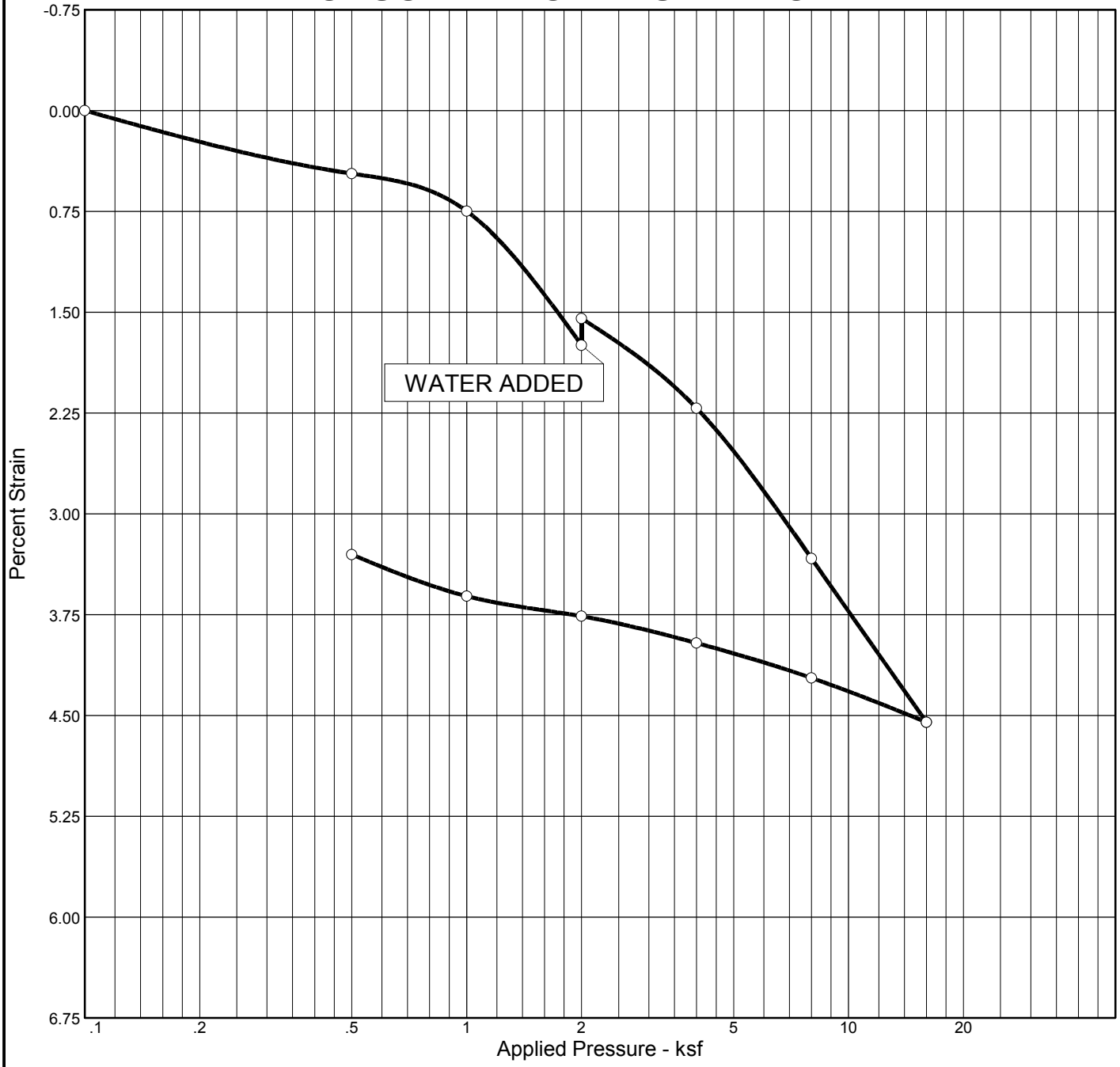
**Remarks:**

**Moore Twining Associates, Inc.**

**Fresno, CA**

**Figure**

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Swell % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 52.0 % | 9.9 % | 110.1 | | | 2.65 | | 2.27 | 0.06 | 0.01 | 2.55 | 0.2 | 0.503 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Sandy silt | ML | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

**Source:** **Sample No.:** B-4 **Elev./Depth:** 1-2.5'

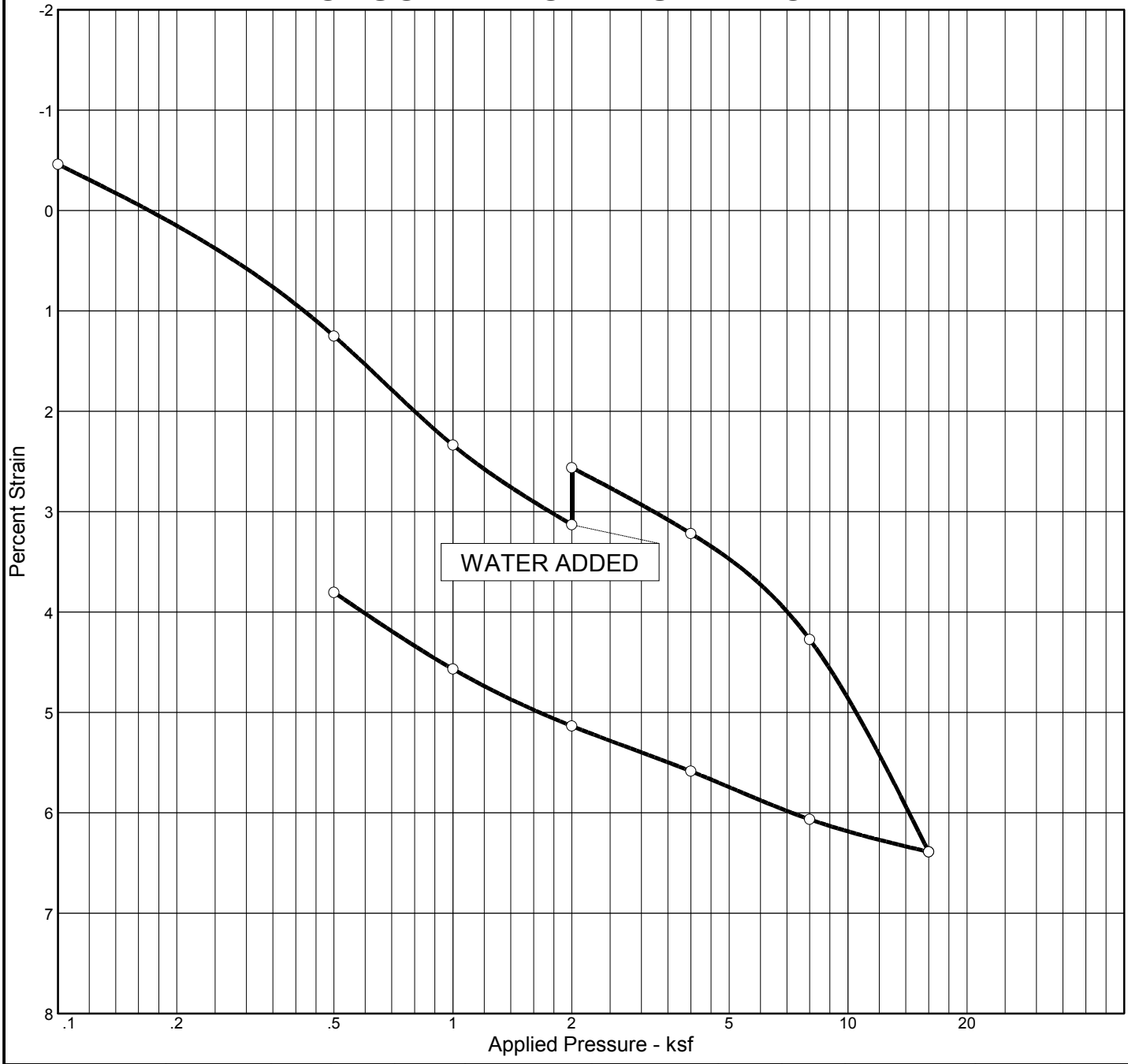
**Moore Twining Associates, Inc.**

**Fresno, CA**

**Remarks:**

**Figure**

# CONSOLIDATION TEST REPORT




| Natural | | Dry Dens. (pcf) | LL | PI | Sp. Gr. | Overburden (ksf) | $P_c$ (ksf) | $C_c$ | $C_s$ | Swell Press. (ksf) | Swell % | $e_o$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sat. | Moist. | | | | | | | | | | | |
| 98.0 % | 17.4 % | 112.5 | 21 | 10 | 2.65 | | 8.02 | 0.11 | 0.03 | 3.67 | 0.5 | 0.471 |

| MATERIAL DESCRIPTION | USCS | AASHTO |
|---|---|---|
| Clayey sand | SC | |

**Project No.** H01602.01 **Client:** City of Riverside

**Project:** Riverside PD Replacement HQ

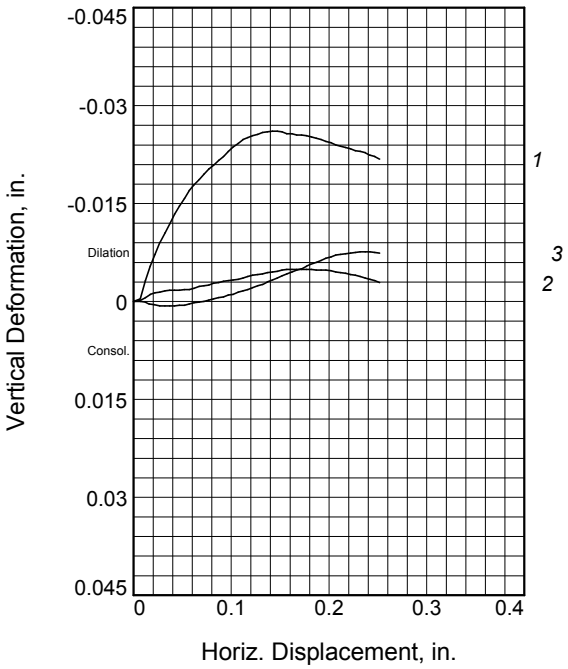
**Source:** **Sample No.:** B-5 **Elev./Depth:** 18.5-20'

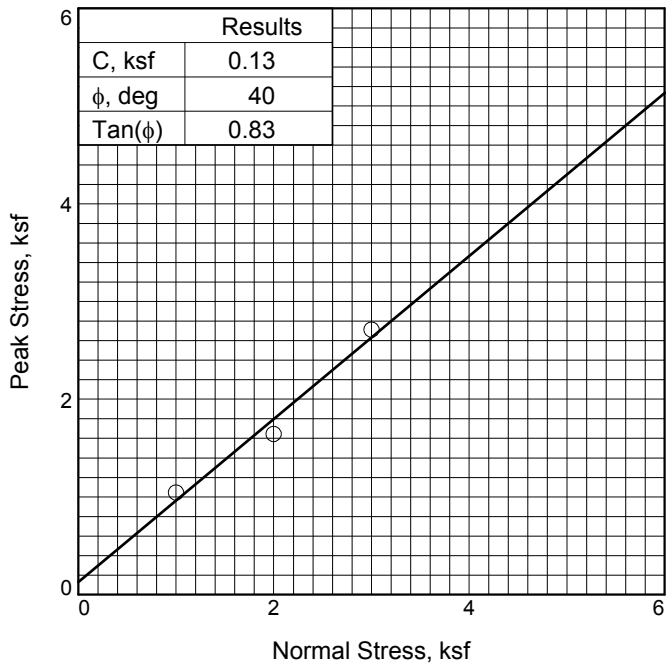
**Moore Twining Associates, Inc.**

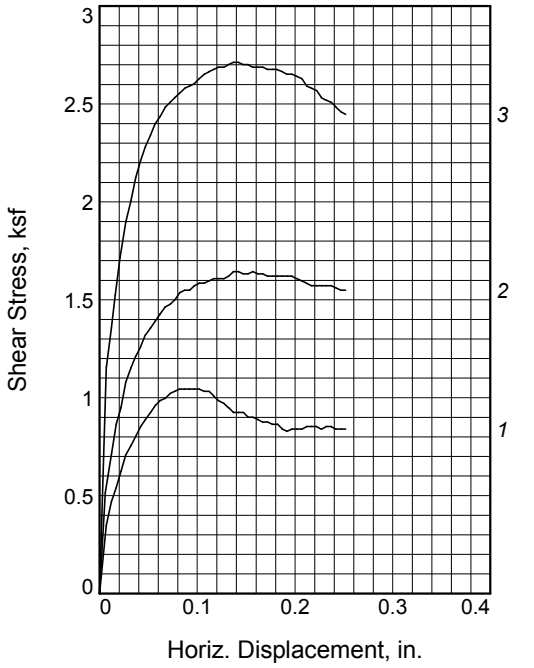
**Fresno, CA**

**Remarks:**

**Figure**




| Sample No. | | 1 | 2 | 3 |
|---|---|---|---|---|
| Initial | Water Content, % | 6.9 | 7.6 | 8.3 |
| | Dry Density, pcf | 120.7 | 119.6 | 125.1 |
| | Saturation, % | 49.2 | 52.8 | 68.2 |
| | Void Ratio | 0.3711 | 0.3835 | 0.3228 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 1.00 | 1.00 | 1.00 |
| At Test | Water Content, % | 12.9 | 13.0 | 10.8 |
| | Dry Density, pcf | 122.8 | 122.1 | 127.4 |
| | Saturation, % | 98.9 | 96.9 | 95.5 |
| | Void Ratio | 0.3470 | 0.3554 | 0.2986 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 0.98 | 0.98 | 0.98 |
| Normal Stress, ksf | | 1.00 | 2.00 | 3.00 |
| Peak Stress, ksf | | 1.04 | 1.64 | 2.71 |
| Displacement, in. | | 0.08 | 0.14 | 0.14 |
| Ultimate Stress, ksf | | | | |
| Displacement, in. | | | | |
| Strain at peak, % | | 3.4 | 5.7 | 5.7 |

**Sample Type:**
**Description:** Silty sand

**LL=** NV **PI=** NP
**Specific Gravity=** 2.65
**Remarks:**

**Figure** _____

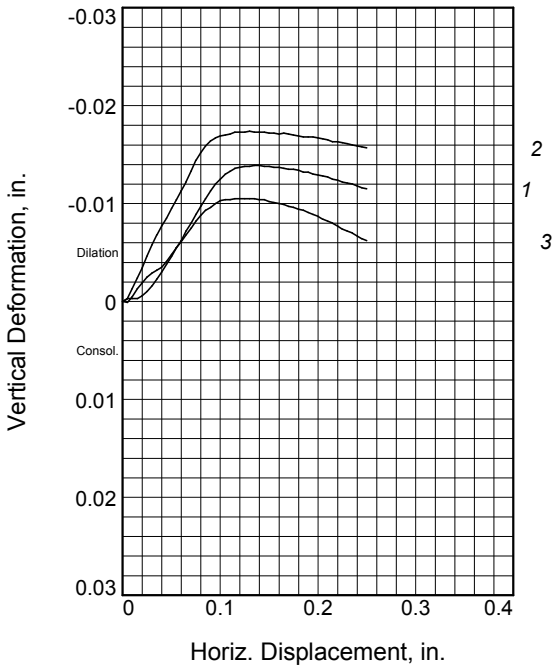
**Client:** City of Riverside

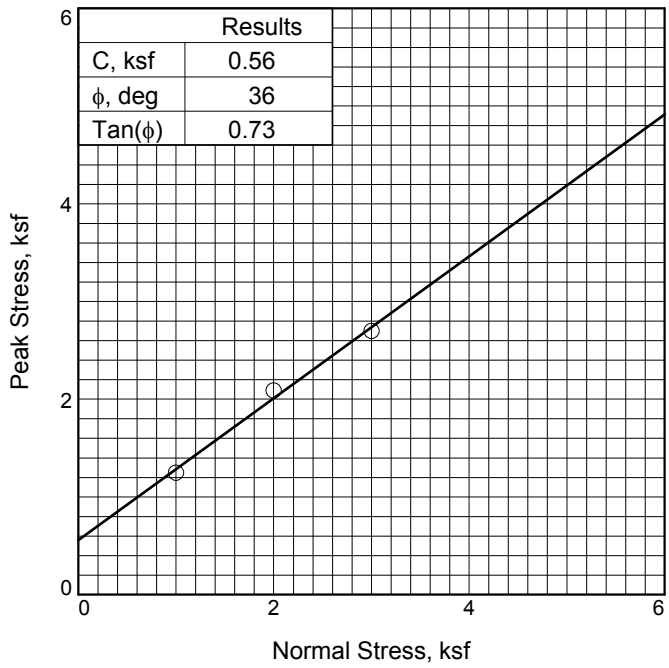
**Project:** Riverside PD Replacement HQ

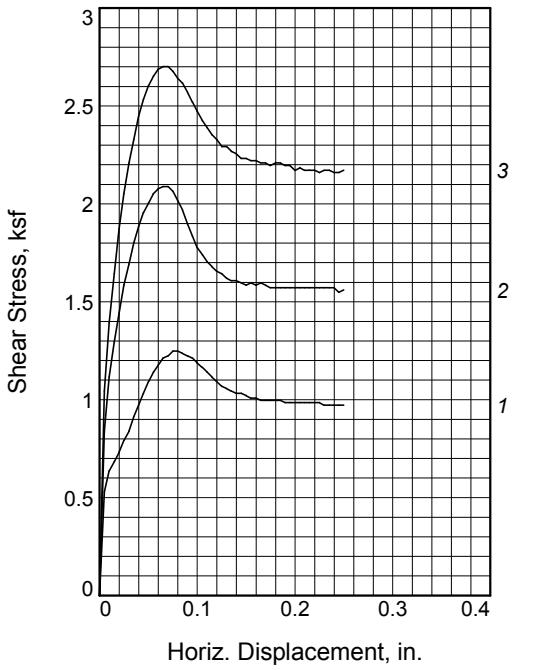
**Sample Number:** B-4 **Depth:** 7-8.5'

**Proj. No.:** H01602.01 **Date Sampled:**

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA




| Sample No. | | 1 | 2 | 3 |
|---|---|---|---|---|
| Initial | Water Content, % | 9.7 | 9.1 | 9.1 |
| | Dry Density, pcf | 122.9 | 125.1 | 125.5 |
| | Saturation, % | 74.0 | 75.0 | 76.1 |
| | Void Ratio | 0.3461 | 0.3222 | 0.3184 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 1.00 | 1.00 | 1.00 |
| At Test | Water Content, % | 12.0 | 10.9 | 10.1 |
| | Dry Density, pcf | 125.2 | 127.7 | 129.5 |
| | Saturation, % | 99.4 | 97.3 | 96.9 |
| | Void Ratio | 0.3210 | 0.2955 | 0.2770 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 0.98 | 0.98 | 0.97 |
| Normal Stress, ksf | | 1.00 | 2.00 | 3.00 |
| Peak Stress, ksf | | 1.25 | 2.09 | 2.70 |
| Displacement, in. | | 0.08 | 0.07 | 0.07 |
| Ultimate Stress, ksf | | | | |
| Displacement, in. | | | | |
| Strain at peak, % | | 3.1 | 2.7 | 2.7 |

**Sample Type:**
**Description:** Silty sand

**Specific Gravity=** 2.65
**Remarks:**

**Figure** _____

**Client:** City of Riverside

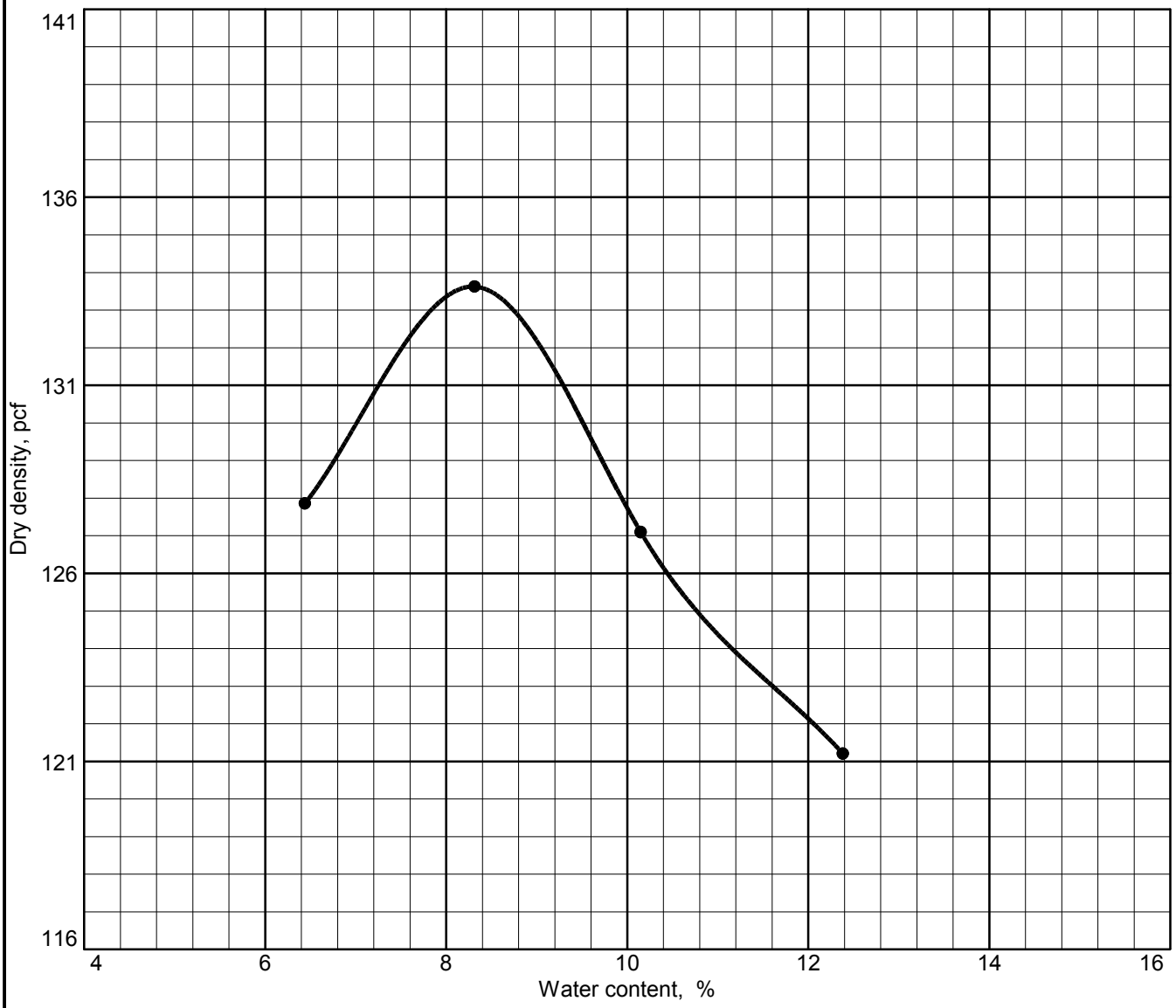
**Project:** Riverside PD Replacement HQ

**Sample Number:** B-2 **Depth:** 20-21.5'

**Proj. No.:** H01602.01 **Date Sampled:** 8/20/25

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
Fresno, CA

# COMPACTION TEST REPORT




Test specification: ASTM D 1557-12 Method A Modified

| Elev/ Depth | Classification | | Nat. Moist. | Sp.G. | LL | PI | % > No.4 | % < No.200 |
|---|---|---|---|---|---|---|---|---|
| | USCS | AASHTO | | | | | | |
| 0-5' | SC | | | | | | 0.0 | |

| TEST RESULTS | MATERIAL DESCRIPTION |
|---|---|
| Maximum dry density = 133.6 pcf<br>Optimum moisture = 8.3 % | Clayey sand |

**Project No.** H01602.01 **Client:** City of Riverside
**Project:** Riverside PD Replacement HQ

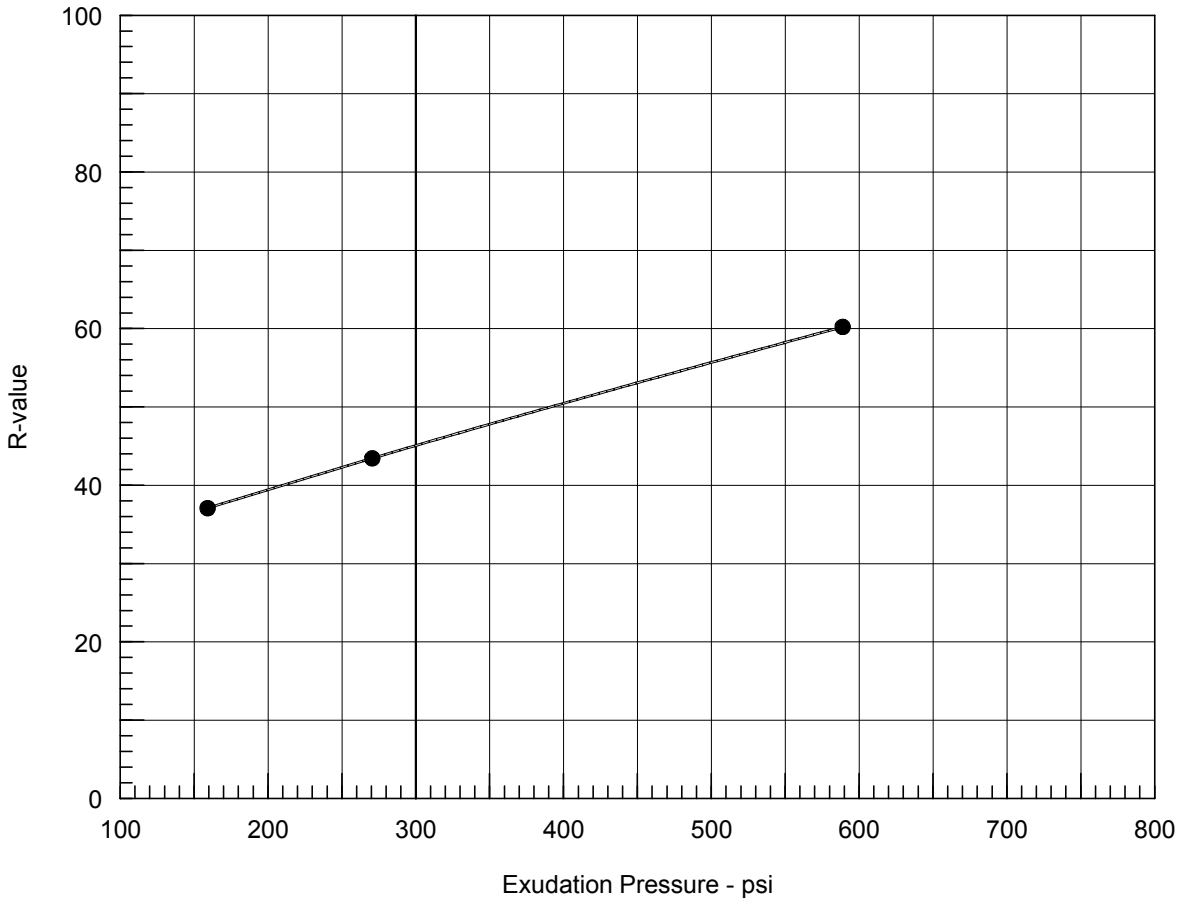
● **Source:** **Sample No.:** B-5 **Elev./Depth:** 0-5'

**Moore Twining Associates, Inc.**
**Fresno, CA**

**Remarks:**

**Figure**

# R-VALUE TEST REPORT




**Resistance R-Value and Expansion Pressure - ASTM D 2844**

| No. | Compact. Pressure psi | Density pcf | Moist. % | Expansion Pressure psi | Horizontal Press. psi @ 160 psi | Sample Height in. | Exud. Pressure psi | R Value | R Value Corr. |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 140 | 120.2 | 13.0 | 0.00 | 60 | 2.43 | 271 | 45 | 43 |
| 2 | 110 | 120.3 | 14.0 | 0.00 | 73 | 2.48 | 159 | 37 | 37 |
| 3 | 350 | 123.2 | 11.9 | 0.00 | 40 | 2.36 | 589 | 64 | 60 |
| | | | | | | | | | |

| Test Results | Material Description |
|---|---|
| **R-value at 300 psi exudation pressure =** 45 | Silty sand |

**Project No.:** H01602.01
**Project:** Riverside PD Replacement HQ
**Sample Number:** B-6 / P-1 **Depth:** 0.5'-7'
**Date:** 9/25/2025

**Tested by:** CT
**Checked by:** MS
**Remarks:**

R-VALUE TEST REPORT

**Moore Twining Associates, Inc.**

**Figure** N/A

MOORE TWINING ASSOCIATES, INC.
*California ELAP Certificate #1371*

**2527 Fresno Street**
**Fresno, CA 93721**
**(559) 268-7021 Phone**
**(559) 268-0740 Fax**

September 17, 2025

Work Order #: **LI03001**

Shaun Reich
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

**RE: Riverside PD Replacement HQ**

Enclosed are the analytical results for samples received by our laboratory on **09/03/25** . For your reference, these analyses have been assigned laboratory work order number **LI03001**.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, Moore Twining Associates, Inc. (MTA) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

**Moore Twining Associates, Inc.**



Lauren Cox
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
*California ELAP Certificate #1371*

**2527 Fresno Street**
**Fresno, CA 93721**
**(559) 268-7021 Phone**
**(559) 268-0740 Fax**

| | | |
|---|---|---|
| MTA Geotechnical Division | **Project:** Riverside PD Replacement HQ | **Reported:** |
| 2527 Fresno Street | **Project Number:** H01602.01 | 09/17/2025 |
| Fresno CA, 93721 | **Project Manager:** Shaun Reich | |

## Analytical Report for the Following Samples

| Sample ID | Notes | Laboratory ID | Matrix | Date Sampled | Date Received |
|---|---|---|---|---|---|
| B-3 @ 1-5 | | LI03001-01 | Soil | 08/21/25 00:00 | 09/03/25 08:42 |

MOORE TWINING ASSOCIATES, INC.
*California ELAP Certificate #1371*

**2527 Fresno Street**
**Fresno, CA 93721**
**(559) 268-7021 Phone**
**(559) 268-0740 Fax**

| | | |
|---|---|---|
| MTA Geotechnical Division | **Project:** Riverside PD Replacement HQ | **Reported:** |
| 2527 Fresno Street | **Project Number:** H01602.01 | 09/17/2025 |
| Fresno CA, 93721 | **Project Manager:** Shaun Reich | |

## B-3 @ 1-5

LI03001-01 (Soil)

| Analyte | Result | Reporting Limit | Units | Batch | Prepared | Analyzed | Method | Flag |
|---|---|---|---|---|---|---|---|---|
| **Inorganics** | | | | | | | | |
| Chloride | ND | 0.00060 | % by Weight | [CALC] | 09/08/25 | 09/08/25 | [CALC] | |
| Chloride | ND | 6.0 | mg/kg | B5I0502 | 09/05/25 | 09/08/25 | Cal Test 422 | |
| pH | **8.1** | 0.10 | pH Units | B5I0502 | 09/05/25 | 09/05/25 | Cal Test 643 M | |
| Sulfate as SO4 | **0.0011** | 0.00060 | % by Weight | [CALC] | 09/08/25 | 09/08/25 | [CALC] | |
| Sulfate as SO4 | **11** | 6.0 | mg/kg | B5I0502 | 09/05/25 | 09/08/25 | Cal Test 417 | |

## Notes and Definitions

| | |
|---|---|
| PREP | Modified preparation by pulverizing sample to pass #40 sieve and soaked for a minimum of 12 hours using a minimum dilution ratio of 1:10 |
| ND | Analyte NOT DETECTED at or above the reporting limit |
| mg/kg | milligrams per kilogram (parts per million concentration units) |



*The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.*

CI0300 1

# MOORE TWINING — CHAIN OF CUSTODY / ANALYSIS REQUEST

2527 FRESNO STREET • FRESNO, CA 93721 • PHONE (559) 268-7021 • FAX: (559) 268-0740

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

**WORK ORDER #:** LI0210 1
**PAGE** 1 OF 2

| REPORT TO: | ☐ INVOICE TO: | ☐ REPORT COPY TO: | REPORTING: |
|---|---|---|---|
| ATTENTION: Shaun R | ATTENTION: | | ☐ STANDARD FORMAT ☐ PDF<br>☐ EDT (SWRCB) ☐ EXCEL<br>☐ GEOTRACKER/COELT (LUFT)<br>GLOBAL ID: |
| COMPANY NAME: Geotechnical Division | COMPANY NAME: | | ☐ COUNTY ENVIRONMENTAL HEALTH: |
| ADDRESS: | ADDRESS: | | ☐ STATE WATER RESOURCES CONTROL BOARD: |
| PHONE: | PHONE: | | ☐ OTHER: |
| EMAIL / FAX: | EMAIL / FAX: | | |

**SAMPLE INFORMATION**

SAMPLED BY (PRINT):

SIGNATURE:

☐ PUBLIC SYSTEM ☐ ROUTINE
☐ PRIVATE WELL ☐ REPEAT
☐ OTHER ☐ REPLACEMENT

TURN AROUND TIME
☒ STANDARD
☐ RUSH, DUE ON: 10 pay TAT

**SAMPLE TYPES**

SOLID:
BS - BIOSOLID
CR - CERAMIC
SL - SOIL/SOLID
LIQUID:
DW - DRINKING WATER
GW - GROUND WATER
OL - OIL
SF - SURFACE WATER
ST - STORM WATER
WW - WASTEWATER

**PROJECT INFORMATION**

CONTRACT / P.O. NO.:

PROJECT: Riverside PD Replacement HQ.

PROJECT NUMBER: H01602.01

PROJECT MANAGER: Shaun R.

**ANALYSIS REQUESTED**

LAB USE

NOTES ON RECEIVED CONDITION:
☐ CUSTODY SEAL(S) BROKEN ☐ SAMPLES(S) DAMAGED
☐ ON ICE ☐ AMBIENT TEMP. ☐ INCORRECT PRESERVATION

| CLIENT SAMPLE ID | DATE | TIME | TYPE | pH | Chlorite | Sulfate | | | | | | | | STATION CODE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B-3 @ 1-5 | 8-21-25 | | SL | X | X | X | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
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| | | | | | | | | | | | | | | |

COMMENTS / ADDITIONAL INSTRUCTIONS:

| RELINQUISHED BY | COMPANY | DATE | TIME | RECEIVED BY | COMPANY |
|---|---|---|---|---|---|
| [signature] | | 9-3-25 | 8:12 | [signature] | |
| | | | | | |

Payment for services rendered as noted herein are due in full within 30 days from the date invoiced. If not so paid, account balances are deemed delinquent. Delinquent balances are subject to monthly service charges and interest specified in MTA's current Standard Terms and Conditions for Laboratory Services. The person signing for the Client/Company acknowledges that they are either the Client or an authorized agent to the Client, that the Client agrees to be responsible for payment for the services on this Chain of Custody and agrees to MTA's terms and conditions for laboratory services unless contractually bound otherwise. MTA's current terms and conditions can be obtained by contacting our accounting department at (559) 268-7021.

Sample Integrity — Page 2 of 2 — Moore Twining Associates WO# C103001 — MTA Bottles: Yes or No

**COC Info**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Was temperature within range? Chemistry ≤6°C Micro <10°C Temp_____°C | Yes No N/A | Did all bottle labels agree with COC? | Yes No N/A | Were there bubbles in VOA vials? (Volatiles Only) | Yes No N/A |
| If samples were taken today, is there evidence that chilling has begun? Recvd_____C° | Yes No N/A | Was a sufficient amount of sample received? | Yes No N/A | Was PM notified of discrepancies? PM: By/Time: | Yes No N/A |
| Did all bottles arrive unbroken and intact? | Yes No N/A | Were correct containers and preservatives received for the tests requested? | Yes No N/A | | |
| Do samples have a hold time <72 hours? | Yes No N/A | | | | |

**Bottles Received**

| 125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V) | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Bacti $Na_2S_2O_3$ Thio 120mL | | | | | | | | | | | | | | | | | | | | | | | |
| None (P) | | | | | | | | | | | | | | | | | | | | | | | |
| None (AG) | | | | | | | | | | | | | | | | | | | | | | | |
| None (CG) 500ml | | | | | | | | | | | | | | | | | | | | | | | |
| Cr6 Buffer (P) Borate Carbonate Buffer-EPA 7196A | | | | | | | | | | | | | | | | | | | | | | | |
| Dissolved Oxygen 300ml (P) | | | | | | | | | | | | | | | | | | | | | | | |
| $HNO_3$ (P) | | | | | | | | | | | | | | | | | | | | | | | |
| HCl (AG) | | | | | | | | | | | | | | | | | | | | | | | |
| $H_2SO_4$ (P) | | | | | | | | | | | | | | | | | | | | | | | |
| $H_3PO_4$ (AG) - TOC | | | | | | | | | | | | | | | | | | | | | | | |
| NaOH (P), cyanide | | | | | | | | | | | | | | | | | | | | | | | |
| NaOH + ZnAc (P) sodium hydroxid+zinc acetate | | | | | | | | | | | | | | | | | | | | | | | |
| $Na_2S_2O_3$ (AG) Thio EPA 547, 538, TTHM | | | | | | | | | | | | | | | | | | | | | | | |
| $Na_2S_2O_3$ (CG) Thio EPA , 504.1 | | | | | | | | | | | | | | | | | | | | | | | |
| $Na_2S_2O_3$ 250ml (Brown P) EPA 549 | | | | | | | | | | | | | | | | | | | | | | | |
| Thio/K Citrate(AG) EPA 531.2 | | | | | | | | | | | | | | | | | | | | | | | |
| $NH_4Cl$ (AG) 552- ammonium chloride EPA HAA5 | | | | | | | | | | | | | | | | | | | | | | | |
| Other: | | | | | | | | | | | | | | | | | | | | | | | |
| Client Own | | | | | | | | | | | | | | | | | | | | | | | |
| Low Level Hg/Metals Double Bag | | | | | | | | | | | | | | | | | | | | | | | |
| Plastic Bag | 1 | | | | | | | | | | | | | | | | | | | | | | |
| Glass Jar: 125 / 250 / 500 | | | | | | | | | | | | | | | | | | | | | | | |
| Soil Tube: Brass / Steel / Plastic | | | | | | | | | | | | | | | | | | | | | | | |
| 5g Encore | | | | | | | | | | | | | | | | | | | | | | | |
| Cubitaner 1G / 2.5 G | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |

**Comments**

**Filter or Split**

| | Container | Preservative | Date/Time/Initials |
|---|---|---|---|
| S P F | | | |
| S P F | | | |
| S P F | | | |
| S P F | | | |
| S P F | | | |

Labeled by:_____@_____ Labels checked by: lm @ 849

FL-SC-0003-07

MOORE TWINING ASSOCIATES, INC.

| | | | |
|---|---|---|---|
| Project Name: | Riverside PD Replacement HQ | Report Date: | 9/25/2025 |
| | | Sample Date: | 8/20/2025 |
| Project Number: | H01602.01 | | |
| | | Sampled By: | SR |
| Subject: | Minimum Resistivity, ASTM G187 | Tested By: | RS |
| Material Description: | Silty sand | Test Date: | 9/4/2025 |
| Location: | B-3 @ 1-5' | | |

**Laboratory Test Results, Minimum Resistivity - ASTM G187**

| Total Water Added, mls | Resistivity, Ohm-cm |
|---|---|
| 25 mls | 28,000 |
| 50 mls | 23,000 |
| 75 mls | 20,000 |
| 100 mls | 1,050 |
| 125 mls | 9,300 |
| 150 mls | 9,600 |

Remarks: Min. Resistivity is 9,300 Ohm-cm



PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

# APPENDIX D

## FIELD PERCOLATION TESTS

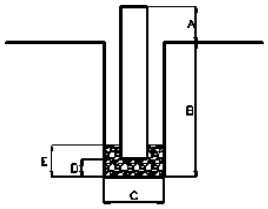
This appendix contains the field percolation tests. The field tests represents the results of the field percolation tests.

MOORE TWINING ASSOCIATES, INC.

**PERCOLATION TEST**
**No. P-1**

**Project:** Riverside PD Replacement HQ
**Location:** 4102 Orange Street
**Coordinates:**

**Project No.** H01602.01
**Test Date:** 8/22/2025




| | |
|---|---|
| A. Top of Pipe Above Ground | -4 Inches |
| B. Depth of Hole | 124 Inches |
| C. Diameter of Hole | 8 Inches |
| D. Depth of Gravel Below Pipe | 3 Inches |
| E. Total Gravel Layer Thickness | 73.5 Inches |
| F. Pipe Length | 120 Inches |
| G. Pipe Diameter | 2 Inches |

Pre-saturated: to 72 inches from bottom on 8/21/25 at 3:50 pm
Checked 0 inches from bottom on 8/22/25 A 8:00 am

Gravel Correction Factor: 2.6

| Trial | Date | Time | Depth To Water* (feet) | Time Interval (min) | Water Drop (inches) | Unfactored Percolation Rate, (minutes per inch) | Unfactored Infiltration Rate, (Inches per hour) |
|---|---|---|---|---|---|---|---|
| 1 | 8/22/2025 | 8:24:00 | 7.91 | | | | |
| Criteria Test | 8/22/2025 | 8:34:00 | 8.6 | 10.00 | 8.28 | 3.1 | 1.7 |
| 2 | 8/22/2025 | 8:34:00 | 8.6 | | | | |
| Criteria Test | 8/22/2025 | 8:44:00 | 8.98 | 10.00 | 4.56 | 5.6 | 1.3 |
| 3 | 8/22/2025 | 8:44:00 | 8.98 | | | | |
| | 8/22/2025 | 8:54:00 | 9.23 | 10.00 | 3 | 8.5 | 1.1 |
| 4 | 8/22/2025 | 8:54:00 | 9.23 | | | | |
| | 8/22/2025 | 9:04:00 | 9.38 | 10.00 | 1.8 | 14.2 | 0.8 |
| Refill 5 | 8/22/2025 | 9:08:00 | 7.37 | | | | |
| | 8/22/2025 | 9:18:00 | 8.1 | 10.00 | 8.76 | 2.9 | 1.4 |
| 6 | 8/22/2025 | 9:18:00 | 8.1 | | | | |
| | 8/22/2025 | 9:28:00 | 8.54 | 10.00 | 5.28 | 4.8 | 1.1 |
| 7 | 8/22/2025 | 9:28:00 | 8.54 | | | | |
| | 8/22/2025 | 9:38:00 | 8.89 | 10.00 | 4.2 | 6.1 | 1.1 |
| 8 | 8/22/2025 | 9:38:00 | 8.89 | | | | |
| | 8/22/2025 | 9:48:00 | 9.14 | 10.00 | 3 | 8.5 | 1.0 |
| 9 | 8/22/2025 | 9:48:00 | 9.14 | | | | |
| | 8/22/2025 | 9:58:00 | 9.3 | 10.00 | 1.92 | 13.3 | 0.8 |
| Refill 10 | 8/22/2025 | 10:01:00 | 8.22 | | | | |
| | 8/22/2025 | 10:11:00 | 8.61 | 10.00 | 4.68 | 5.5 | 1.0 |
| 11 | 8/22/2025 | 10:11:00 | 8.61 | | | | |
| | 8/22/2025 | 10:21:00 | 8.9 | 10.00 | 3.48 | 7.4 | 0.9 |
| 12 | 8/22/2025 | 10:21:00 | 8.9 | | | | |
| | 8/22/2025 | 10:31:00 | 9.13 | 10.00 | 2.76 | 9.3 | 0.9 |
| 13 | 8/22/2025 | 10:31:00 | 9.13 | | | | |
| | 8/22/2025 | 10:41:00 | 9.3 | 10.00 | 2.04 | 12.5 | 0.8 |

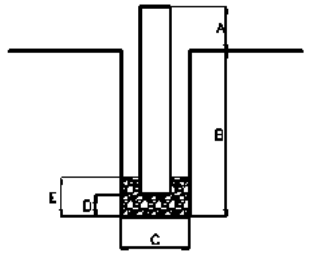
* Depth to water measured from top of pipe

MOORE TWINING ASSOCIATES, INC.

**PERCOLATION TEST**
**No. P-2**

**Project:** Riverside PD Replacement HQ
**Location:** 4102 Orange Street
**Coordinates:**

**Project No.** H01602.01
**Test Date:** 8/22/2025




| | |
|---|---|
| A. Top of Pipe Above Ground | -1 Inches |
| B. Depth of Hole | 119.5 Inches |
| C. Diameter of Hole | 8 Inches |
| D. Depth of Gravel Below Pipe | 3 Inches |
| E. Total Gravel Layer Thickness | 53 Inches |
| F. Pipe Length | 120 Inches |
| G. Pipe Diameter | 2 Inches |

Pre-saturated: to 72 inches from bottom on 8/21/25 at 3:45 pm
Checked 0 inches from bottom on 8/22/25 at 8:00 am

Gravel Correction Factor: 2.6

| Trial | Date | Time | Depth To Water* (feet) | Time Interval (min) | Water Drop (inches) | Unfactored Percolation Rate, (minutes per inch) | Unfactored Infiltration Rate, (Inches per hour) |
|---|---|---|---|---|---|---|---|
| 1 | 8/22/2025 | 8:30:00 | 7.79 | | | | |
| Criteria Test | | 8:40:00 | 8.45 | 10.00 | 7.92 | 3.2 | 1.6 |
| 2 | | 8:46:00 | 8.45 | | | | |
| Criteria Test | | 8:56:00 | 8.93 | 10.00 | 5.76 | 4.4 | 1.6 |
| Refill 3 | | 9:00:00 | 6.93 | | | | |
| | | 9:10:00 | 7.68 | 10.00 | 9 | 2.8 | 1.3 |
| 4 | | 9:10:00 | 7.68 | | | | |
| | | 9:20:00 | 8.2 | 10.00 | 6.24 | 4.1 | 1.1 |
| 5 | | 9:20:00 | 8.2 | | | | |
| | | 9:30:00 | 8.63 | 10.00 | 5.16 | 5.0 | 1.2 |
| 6 | | 9:30:00 | 8.63 | | | | |
| | | 9:40:00 | 8.95 | 10.00 | 3.84 | 6.7 | 1.2 |
| 7 | | 9:40:00 | 8.95 | | | | |
| | | 9:50:00 | DRY | 10.00 | -- | -- | -- |
| Refill 8 | | 9:54:00 | 7.1 | | | | |
| | | 10:04:00 | 7.68 | 10.00 | 6.96 | 3.7 | 1.0 |
| 9 | | 10:04:00 | 7.68 | | | | |
| | | 10:14:00 | 8.18 | 10.00 | 6 | 4.3 | 1.1 |
| 10 | | 10:14:00 | 8.18 | | | | |
| | | 10:24:00 | 8.6 | 10.00 | 5.04 | 5.1 | 1.2 |
| 11 | | 10:24:00 | 8.6 | | | | |
| | | 10:34:00 | 8.95 | 10.00 | 4.2 | 6.1 | 1.3 |
| Refill 12 | | 10:37:00 | 7.87 | | | | |
| | | 10:47:00 | 8.32 | 10.00 | 5.4 | 4.7 | 1.1 |
| 13 | | 10:47:00 | 8.32 | | | | |
| | | 10:57:00 | 8.72 | 10.00 | 4.8 | 5.3 | 1.2 |
| 14 | | 10:57:00 | 8.72 | | | | |
| | | 11:07:00 | 9.03 | 10.00 | 3.72 | 6.9 | 1.2 |

* Depth to water measured from top of pipe

# APPENDIX E

## LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

This appendix contains the graphical results of the liquefaction and seismic settlement analyses

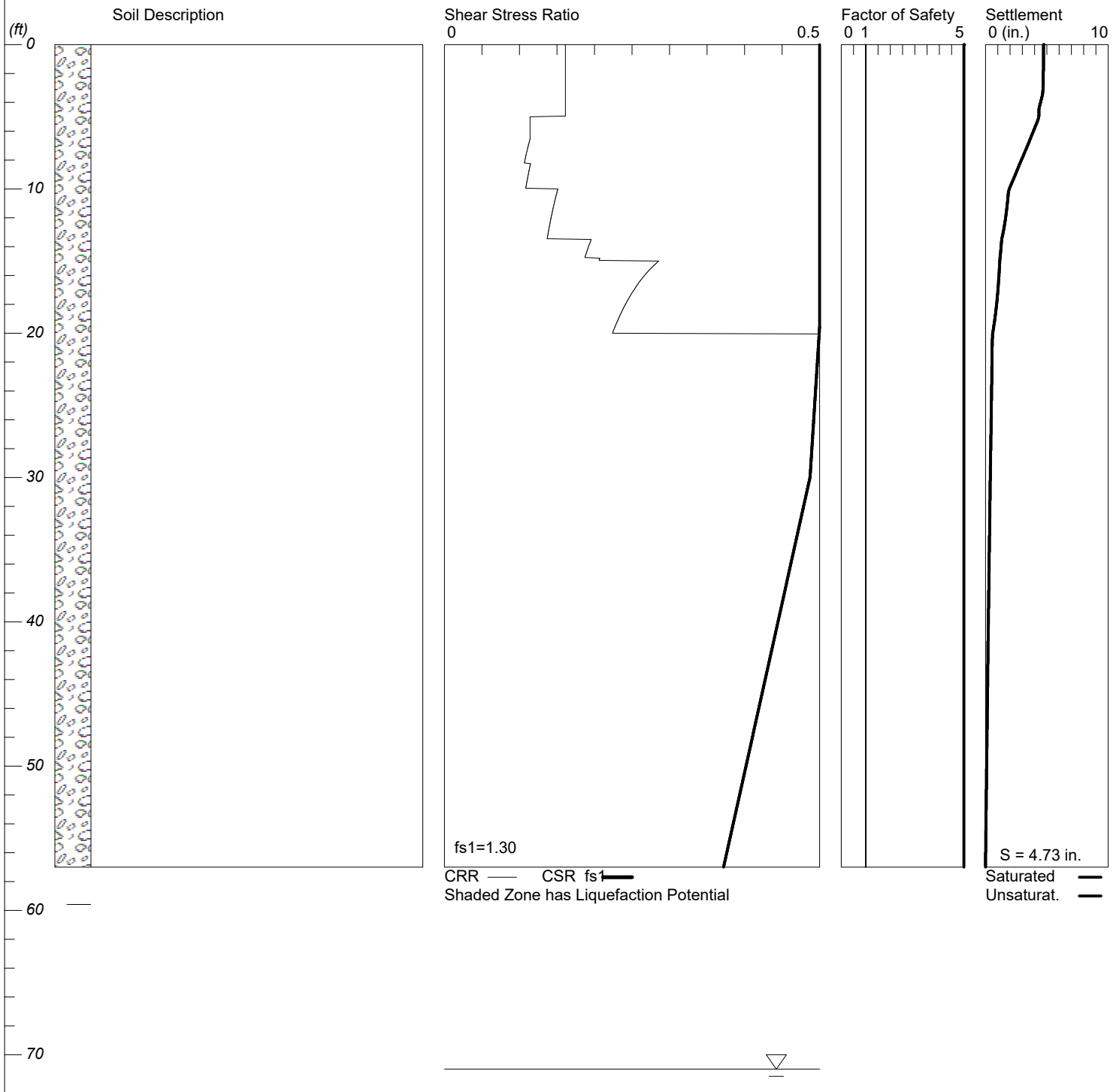
# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

### Parking Lot Area

***Hole No.=B-2  Water Depth=71 ft  Surface Elev.=850***

***Magnitude=8.2***
***Acceleration=0.62g***



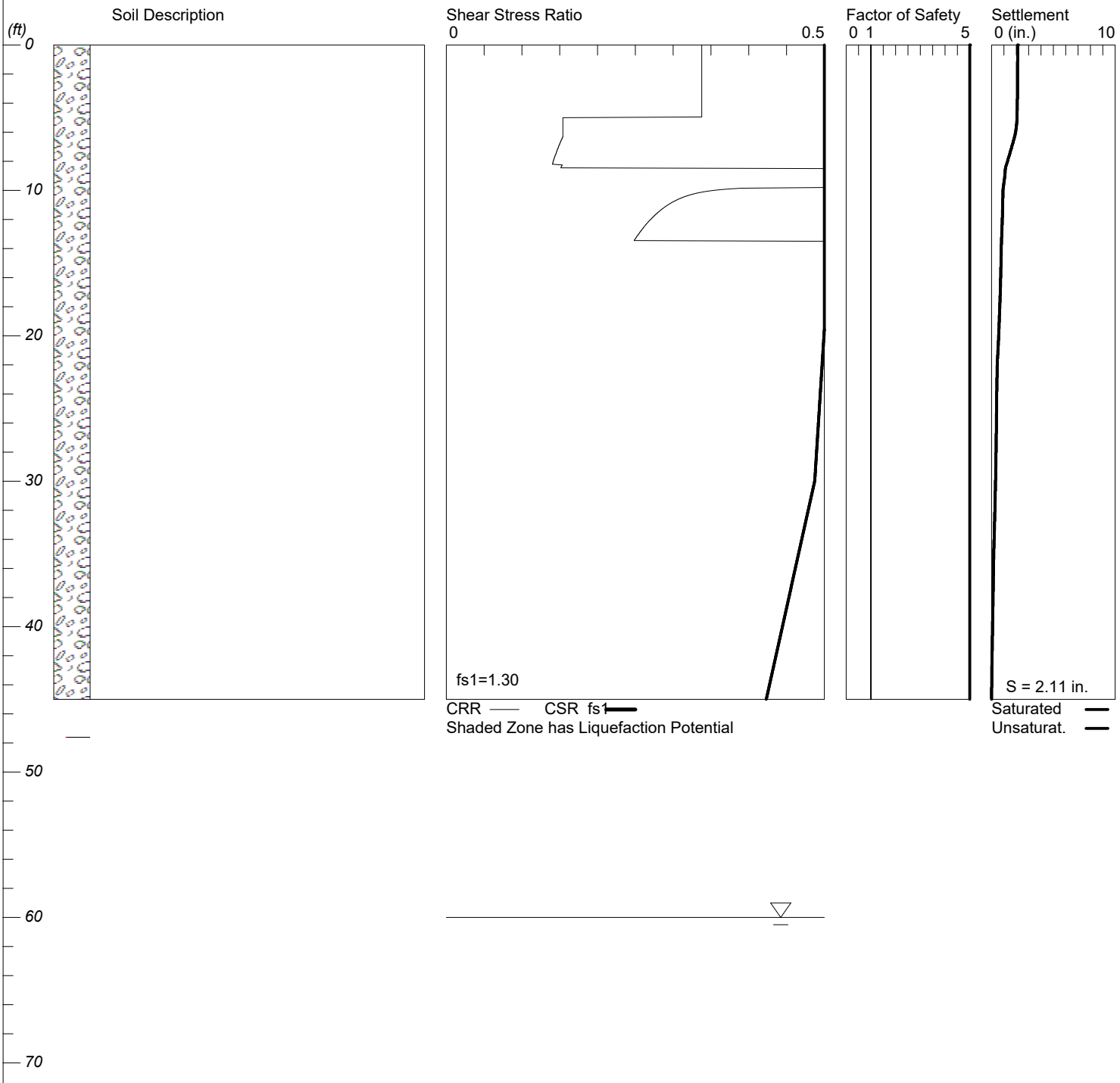

LiquefyPro CivilTech Software USA www.civiltech.com

# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

***Hole No.=B-3  Water Depth=60 ft  Surface Elev.=841.5***

***Magnitude=8.2***
***Acceleration=0.62g***



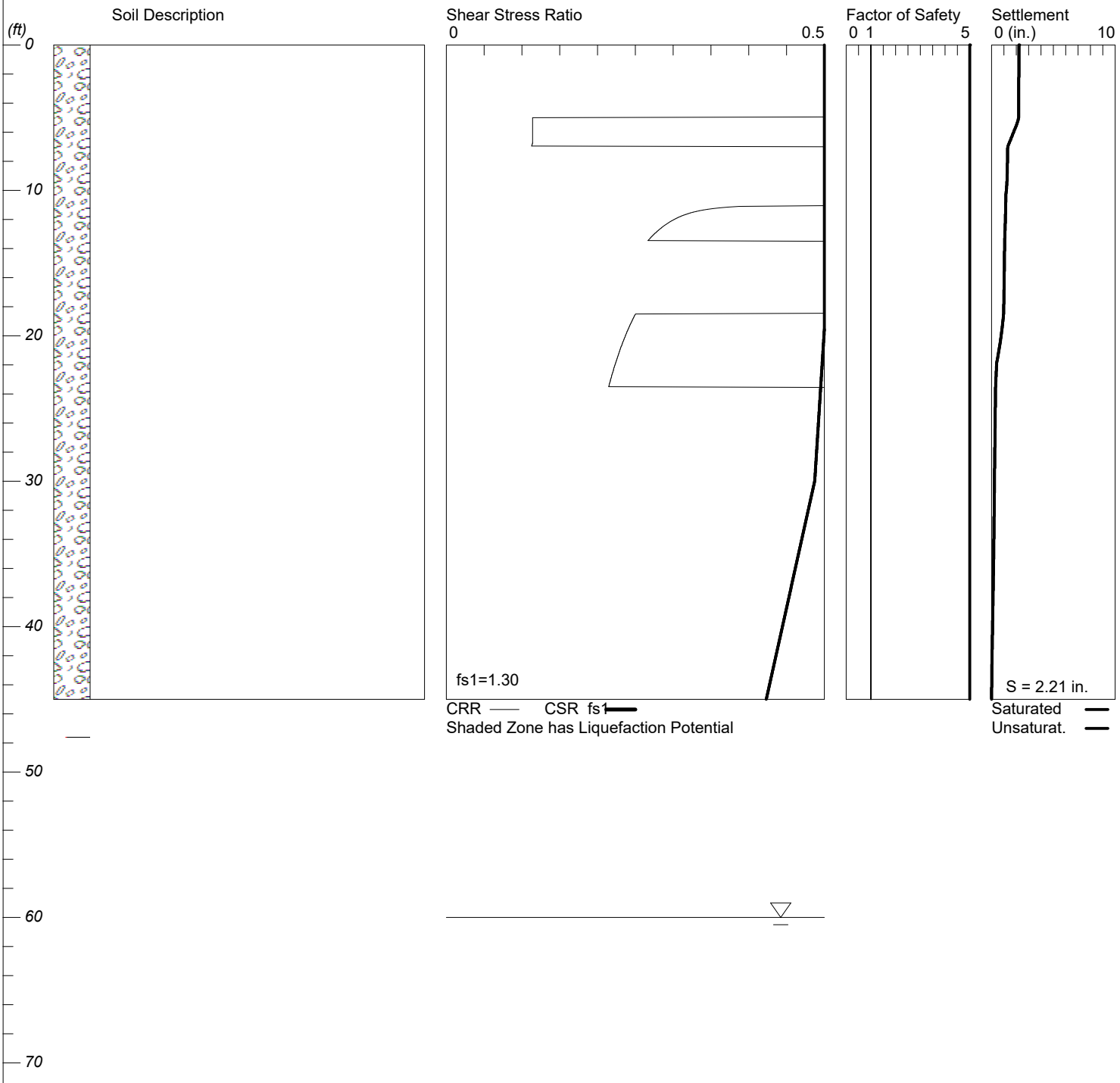

LiquefyPro CivilTech Software USA www.civiltech.com

# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

***Hole No.=B-4  Water Depth=60 ft  Surface Elev.=841.5***

***Magnitude=8.2***
***Acceleration=0.62g***



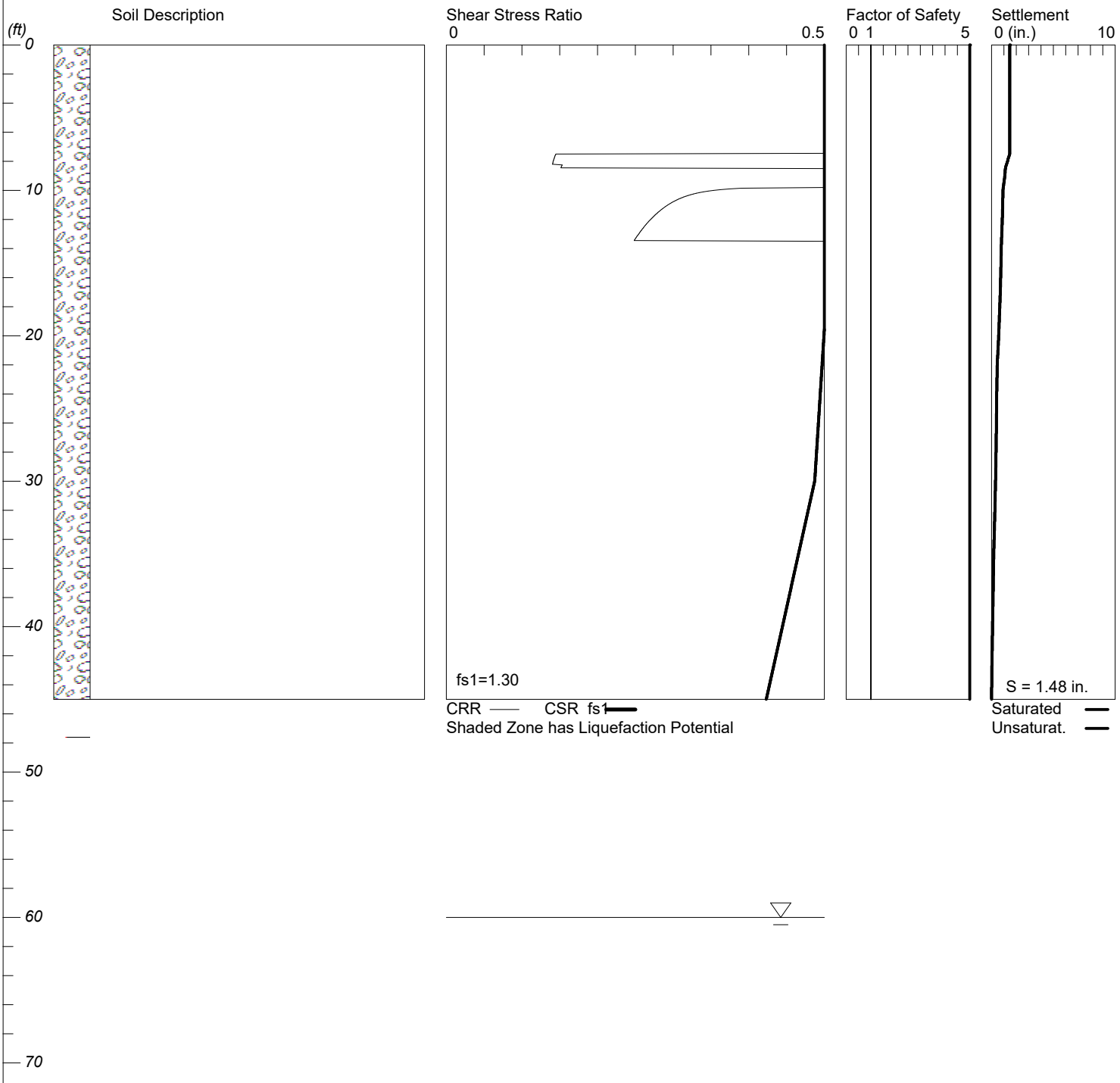

LiquefyPro CivilTech Software USA www.civiltech.com

# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg; 7-ft Over-Excavation

***Hole No.=B-3  Water Depth=60 ft  Surface Elev.=841.5***

***Magnitude=8.2***
***Acceleration=0.62g***



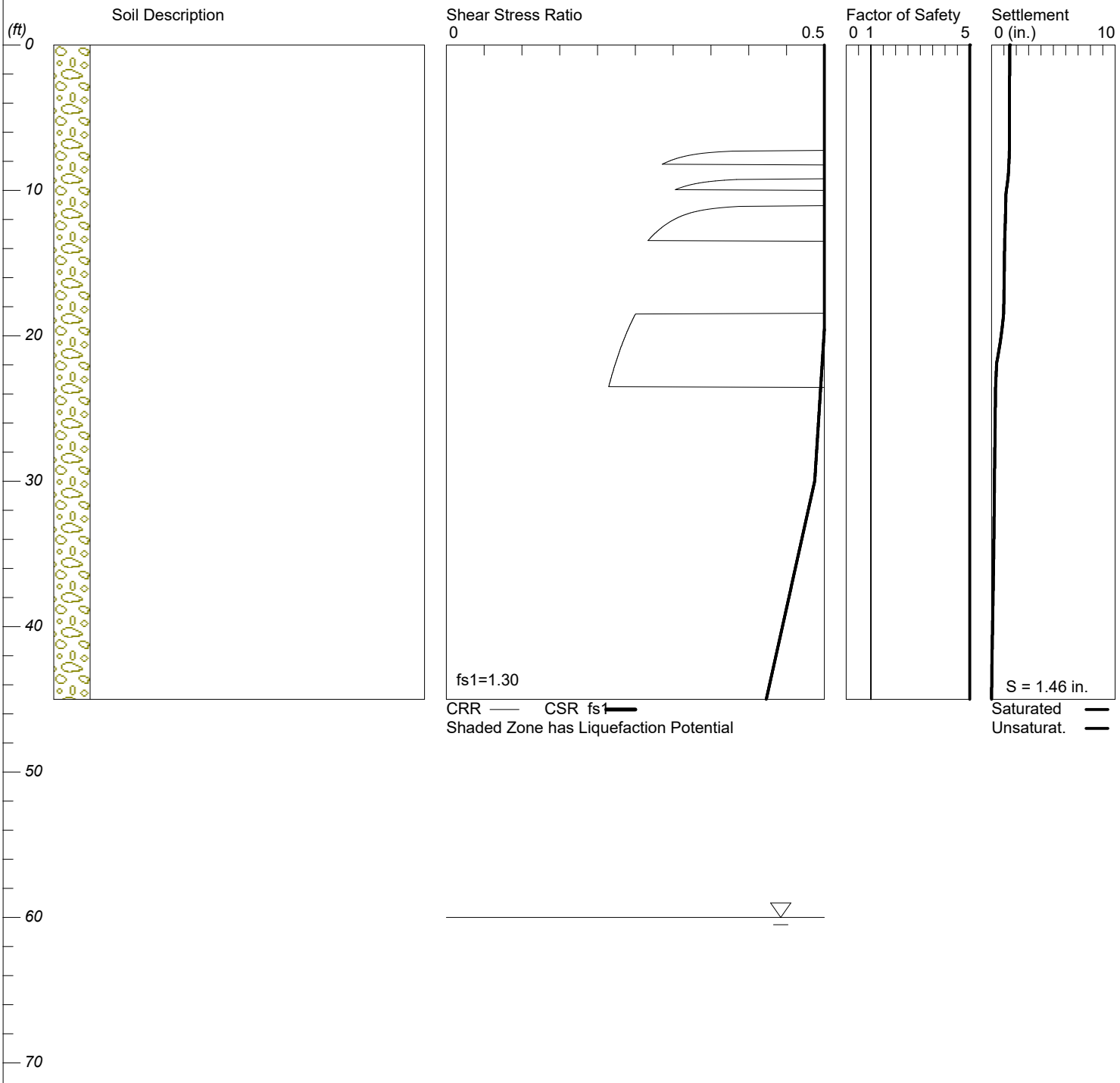

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# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg; 7-ft Over-Excavation

***Hole No.=B-4  Water Depth=60 ft  Surface Elev.=841.5***

***Magnitude=8.2***
***Acceleration=0.62g***




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MOORE TWINING ASSOCIATES, INC.

June 16, 2026 DRAFT H01602.01

City of Riverside
8095 Lincoln Avenue
Riverside, CA 95204

Attention: Mr. Joseph Lopez, Capital Projects Manager

Subject: **Addendum No. 1 to Geotechnical Engineering and Geohazard Investigation**
**Proposed Replacement Headquarters Building**
**Riverside Police Department**
**4102 Orange Street**
**Riverside, California**

Reference: Geotechnical Engineering and Geologic/Seismic Hazard Investigation, Proposed Replacement Headquarters Building, Riverside Police Department, 4102 Orange Street, Riverside, California, dated December 01, 2025, prepared by Moore Twining Associates, Inc., Project No. H01602.01

Dear Mr. Lopez:

We are pleased to submit this addendum to our above referenced Geotechnical Engineering and Geologic/Seismic Hazard Investigation report for the proposed replacement Riverside Police Department headquarters building project in Riverside, California. This addendum was prepared based upon a supplemental geotechnical investigation conducted to further assess the potential for dry seismic settlement.

## 1.0 PURPOSE AND SCOPE OF SUPPLEMENTAL INVESTIGATION

**1.1 Purpose:** As background, Moore Twining Associates, Inc. (Moore Twining) prepared the referenced Geotechnical Engineering and Geologic/Seismic Hazard Investigation report for the proposed replacement Riverside Police headquarters building project, dated December 01, 2025. Based on excessive dry seismic settlement derived from standard penetration test data from drilled borings, the referenced 2025 report provided two (2) options for support of the foundation system: 1). over-excavation to remove a sufficient depth of soils to reduce the dry seismic settlement to a tolerable magnitude for design followed by compaction of engineered fill below foundations; or 2). use of specialty deep ground improvement (rigid inclusions) to allow a lesser depth of excavation. The purpose of this supplemental investigation was to conduct Cone Penetration Tests (CPTs) and conduct a more detailed analysis of dry seismic settlement.


PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

This addendum is not a "stand-alone document" and should be used in conjunction with the Geotechnical Engineering and Geologic/Seismic Hazard Investigation report prepared by our firm, dated December 01, 2025. Where noted, the recommendations included in this report supersede the recommendations of the December 01, 2025 report.

**1.2 Scope:** Our contract amendment, dated April 22, 2026, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

1.2.1 A "Shaded Site Plan - OPT B", prepared by Holt Architecture, dated August 10, 2024, and provided via email by Mr. Joseph Lopez, Capital Projects Manager for the City of Riverside, on January 8, 2025, was reviewed for general project information.

1.2.2 A supplemental investigation including cone penetration testing was conducted.

1.2.3 Mr. Joseph Lopez, Capital Projects Manager for the City of Riverside, Mr. Thomas Howell of Holt Architecture, Mr. Dennis Pham of VCA Engineers, and Mr. Marty Greenwood of Tilden-Coil Constructors, were consulted during the investigation.

1.2.4 The report prepared by our firm entitled, "Geotechnical Engineering and Geologic/Seismic Hazard Investigation Proposed Replacement Headquarters Building, Riverside Police Department, 4102 Orange Street, Riverside, California," dated December 01, 2025, was reviewed.

1.2.5 This report was prepared to present the findings of the supplemental investigation and provide updated recommendations.

## 2.0 SUPPLEMENTAL FIELD EXPLORATION

The supplemental field exploration conducted for this investigation is summarized in the following subsections.

**2.1 Field Exploration:** The field exploration consisted of advancing three (3) Cone Penetration Test (CPT) soundings.

**2.1.1 Cone Penetration Test (CPT) Soundings:** On May 13, 2026, three (3) CPT soundings were advanced at the site to depths ranging from 61 to 75 feet below site grade (BSG). The CPT locations in relation to the existing building footprint and future building footprint, are shown on Drawing No. 1, attached within Appendix A of this addendum. Prior to advancing the CPT soundings, the concrete walkway pavement at CPT-01 was cored using a 3½-inch diameter diamond tip core barrel and advanced

to a depth of 5 feet BSG using a 3-inch diameter hand auger to provide utility clearance and CPT access. CPT soundings CPT-02 and CPT-03 were advanced from the pavement surface by pushing the cone through the asphalt pavement. It should be noted that CPT-01 was advanced within the higher elevation of the site within the Portland cement concrete pedestrian walkway to the north of the existing building. CPT-02 and CPT-03 were advanced within the lower elevation of the asphalt concrete alley on the south side of the existing building. It should be further noted that the surface elevation difference between CPT-01 and CPT-02/CPT-03, is about 11 feet. CPT methods were used to obtain generally continuous soil behavior type and penetration resistance information.

The CPT soundings were performed by Kehoe Testing and Engineering using an electronic piezocone with a 60-degree apex angle and a diameter of 35.7 millimeters (about 1½ inches). The CPT soundings were hydraulically advanced using a 30-ton CPT rig in accordance with ASTM Test Method D3441. CPT measurements of cone tip resistance, sleeve friction, and dynamic pore water pressure were recorded at 0.25 foot intervals during penetration to provide nearly continuous logs of the soil behavior types. The CPT logs and test data are attached to this report.

The CPT sounding holes were backfilled in accordance with the guidelines of Riverside County Department of Environmental Health.

## 3.0 FINDINGS, RESULTS AND UPDATED EVALUATIONS

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the field investigation and our understanding of the proposed construction.

**3.1 Surface Conditions:** At the time of our supplemental field investigation, the site was generally similar to the surface conditions observed during our previous investigation at the site. Refer to the descriptions in the referenced December 01, 2025 report for descriptions of the surface conditions. The concrete walkway at the location of CPT-01 was measured to be about 3½ in thickness. The asphalt concrete at locations CPT-02 and CPT-03 was not measured due to the CPT pushing through the surface. The previous borings advanced in this area indicated an asphalt thickness of 4 to 4½ inches.

In general, the subgrade encountered within CPT-01 below the 3½ inch thick concrete walkway was observed to consist of a silty sand fill extending to a depth of 3 feet BSG. The silty sand fill was underlain by a native clayey sand extending to the depth of hand auger exploration, 5 feet BSG. The CPT analysis data indicated soil behavior types resembling the native subgrade encountered within boring B-2, advanced in the original geotechnical investigation. CPT-01 notably encountered soil behavior types resembling granular sand & silty sand with alternating lenses of silty sand & sandy silt with lenses of clay & silty clay between the depths of about 35 feet BSG and 46 feet BSG. Sand & silty sand was generally further encountered extending to the maximum depth of exploration, 75 feet BSG.

CPT-02 and CPT-03 generally encountered similar soil behavior types which were generally similar to boring B-3 and B-4, which were advanced during the original geotechnical investigation.

It should be further noted that groundwater was not encountered to the depth explored. See Appendix A, Test Boring and CPT Location Map, and Appendix B, CPT graphic logs, for further details.

**3.2 Supplemental Seismic Settlement Evaluations:** Due to the depth of groundwater being greater than 50 feet below site grade, the potential for liquefaction to occur is low. However, there is a potential for dry seismic settlement to occur above the design groundwater depth. Accordingly, dry seismic settlement estimates were estimated. Dry seismic settlement analysis was conducted using the computer program LIQUEFYPRO by Civiltech, and the methodology of Tokimatsu and Seed, 1987. Due to the low potential for liquefaction settlement, a factor of safety of 1.1 was used for the dry seismic settlement analysis.

As part of this report, a supplemental analysis was conducted using the recently obtained CPT data, which indicates that the maximum considered earthquake would induce dry seismic settlement in granular soils between depths of about 3 feet to 20 feet below site grade.

The analysis extended to a depth of 50 feet BSG where CPT-02 and CPT-03 were advanced in the lower portion of the site in the asphalt concrete alley on the south side of the existing building. The analysis for CPT-01 within the Portland cement concrete pedestrian walkway on the north side of the existing building was extended to a depth of 61 feet BSG since it was advanced in an area that is 11 feet high in elevation that the areas of where CPT-02 and CPT-03 were advanced. Based on the analysis, the data from CPT-01 indicated a seismic settlement of about 0.7 inches, while CPT-02 indicated a seismic settlement estimate of about 1½ inches, and CPT-03 indicated a seismic settlement estimate of about 2½ inches.

Graphical results of the seismic settlement analysis are presented in Appendix C.

## 4.0 CONCLUSIONS

Based on the findings of this supplemental investigation, the seismic settlement estimates based on the Cone Penetration Test (CPT) data indicated dry seismic settlements ranging from about 0.7 inches at the location of CPT-01 on the north side of the site (higher elevation) to up to about 2½ inches at the location of CPT-03 on the south side of the site (lower elevation). In order to limit the differential seismic settlement to ¾ inches over a horizontal distance of 30 feet as indicated in the referenced December 01, 2025 Geotechnical Engineering and Geohazards Investigation report, and considering the existing higher ground surface elevation within the northern and western portions of the site, the minimum depth of over-excavation within the higher elevation area of the building pad could be reduced. Thus, this addendum includes revised recommendations for the minimum depth of over-excavation within a portion of the building pad area.

## 5.0 REVISED RECOMMENDATIONS

The site preparation for the building pad recommended in Section 9.5.5 of the referenced December 01, 2025 Geotechnical and Geohazard Investigation report shall be replaced with the following:

**9.5.5R** Following stripping and removal of existing surface and subsurface improvements, two options are provided to mitigate the potential for static and seismic settlement.

Option 1 - over-excavate and place engineered fill to support foundations and reduce seismic settlement: With exception as noted below and as indicated on the attached Drawing No. 1 in Appendix A of this addendum, the building pad area should be over-excavated to a depth of 7 feet below the basement finish floor elevation, to at least 1 foot below subsurface structures to be removed, to at least 2 feet below the bottom of foundations, and to the depth required to remove all soils which are disturbed from demolition work, whichever is greater. **However, the minimum depth of over-excavation within the northern and western portions of the site as indicated on Drawing No. 1 in Appendix A of this addendum shall meet all of the following requirements: a minimum depth of 4½ feet below the finish floor elevation, to at least 1 foot below subsurface structures to be removed, to at least 12 inches below the bottom of the foundations, and to the depth required to remove all soils which are disturbed from demolition work, whichever is greater.**

Option 2 - use of deep ground modification to mitigate seismic settlement and for direct support spread foundations: The over-excavation should extend to the depth required for placement of load transfer platform materials, as required for the deep ground modification design, to at least 1 foot below subsurface structures to be removed, to the depth required to remove all soils which are disturbed from demolition work, and to the depth required to excavate all undocumented fill, whichever is greater. **The rigid inclusions would be required within the entire footprint of the proposed structure to provide uniform support.**

As a minimum, the horizontal limit of the excavation for preparation of the building pad subgrade should include the entire footprint of the building, all foundations, adjacent walkways, **to a minimum of 4 feet horizontally** beyond the edges of all foundations, and to the horizontal extent of the deep ground modification limits, whichever is greater. Upon approval of the excavation by Moore Twining based on visual observations and the survey data provided by the contractor, the bottom of the excavation should be scarified to a depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to a minimum of 92 percent relative compaction prior to placement of fill soils. It should be noted that the above recommendations are not intended for preparation of a working surface to support any specific construction equipment such as may be used for ground modification work. The contractor should determine the preparation and fill material requirements for support of construction equipment. The materials proposed for fill should be submitted to Moore Twining for review and approval prior to use.

If Option 2 above is selected, due to the soil disturbance caused by the deep ground modification installation process, after the deep ground modification work, the subgrade soils should be re-prepared by compacting the soils disturbed from the deep ground modification work. The recompaction work and final documentation/verification of the compaction of the building pad area should be verified by Moore Twining by observation and testing.

## 6.0 CLOSING

This supplemental report was prepared in accordance with generally-accepted engineering principles and practices and is subject to the limitations and notifications included in the referenced December 01, 2025 report. With exception of the recommendations modified herein, the recommendations included in the Geotechnical Engineering and Geologic/Seismic Hazard Investigation Report prepared by our firm, dated December 01, 2025, remain applicable for the project

We appreciate the opportunity to be of service to the City of Riverside. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

**MOORE TWINING ASSOCIATES, INC.**
**Geotechnical Engineering Division**

***DRAFT***

Shaun Reich, EIT
Staff Engineer

***DRAFT***

Allen Harker, CEG
Certified Engineering Geologist

***DRAFT***

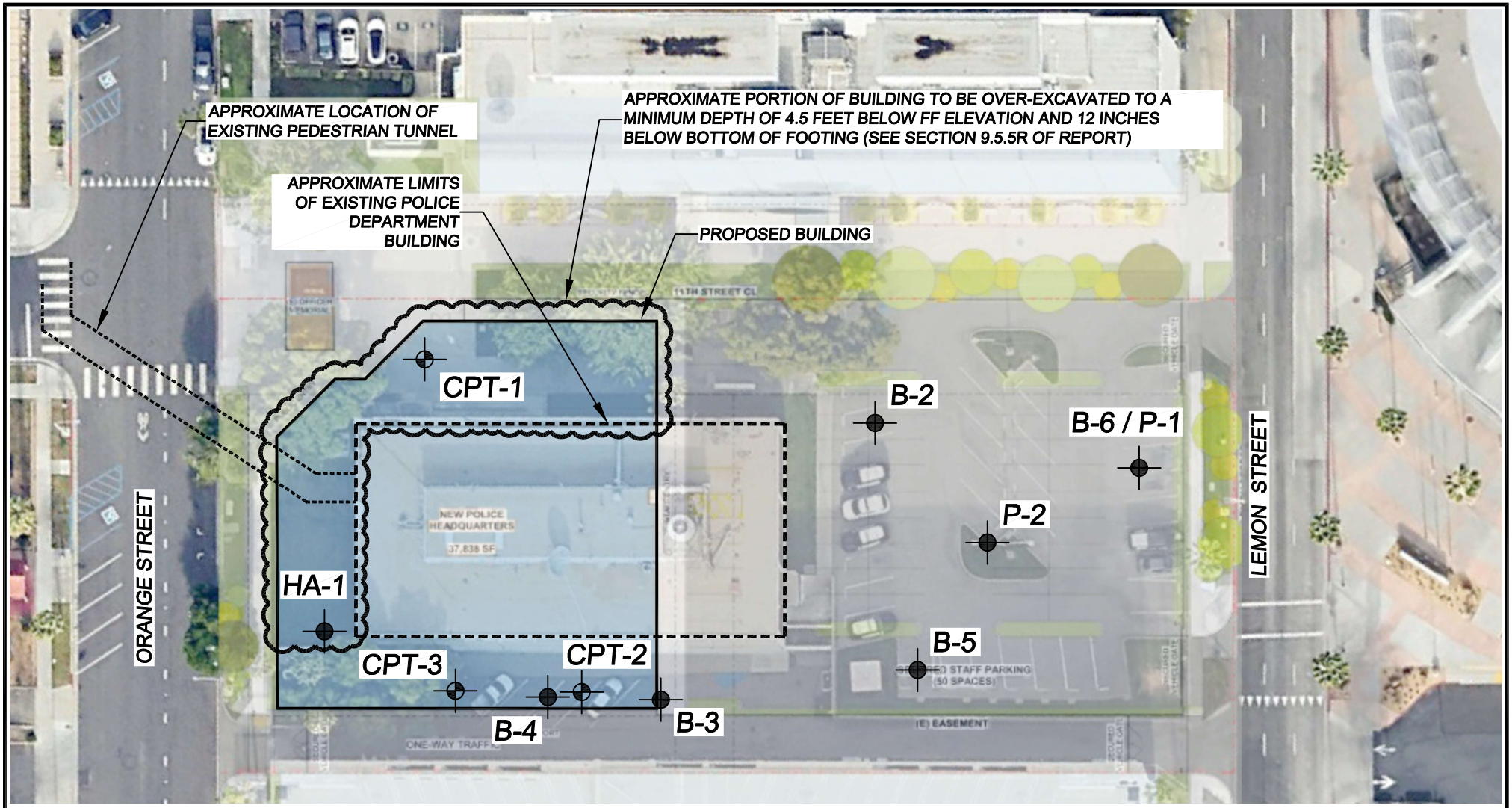
Read L. Andersen, RGE
Geotechnical Division Manager

Attachments: Appendix A: Test Boring and CPT Location Map
Appendix B: Logs of CPT Soundings
Appendix C: Seismic Settlement Graphical Analysis

# APPENDIX A

## DRAWINGS

Drawing No. 1 - Test Boring and CPT Location Map




B-1 — APPROXIMATE TEST BORING LOCATION

HA-1 — APPROXIMATE HAND AUGER BORING LOCATION

CPT-1 — APPROXIMATE CPT LOCATION

P-1 — APPROXIMATE PERCOLATION TEST LOCATION

N

0 50

APPROXIMATE SCALE IN FEET

TEST BORING AND CPT LOCATION MAP
PROPOSED REPLACEMENT OF POLICE DEPARTMENT HEADQUARTERS BUILDING
4102 ORANGE STREET
RIVERSIDE, CALIFORNIA

| | |
|---|---|
| FILE NO. 01602-01-02 | DATE DRAWN: 06/04/2026 |
| DRAWN BY: RM | APPROVED BY: |
| PROJECT NO. H01602.01 | DRAWING NO. 1 |

MOORE TWINING ASSOCIATES, INC.
Since 1898

# APPENDIX B

# LOG OF CPT SOUNDINGS

# SUMMARY

# OF

# CONE PENETRATION TEST DATA

Project:

**Riverside Police Department**
**4102 Orange Street**
**Riverside, CA**
**May 13, 2026**

Prepared for:

**Mr. Shaun Reich**
**Moore Twining Associates, Inc.**
**2527 Fresno Street**
**Fresno, CA 93721**
**Office (800) 268-7021 / Fax (559) 268-7126**

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

# TABLE OF CONTENTS


1. **INTRODUCTION**
2. **SUMMARY OF FIELD WORK**
3. **FIELD EQUIPMENT & PROCEDURES**
4. **CONE PENETRATION TEST DATA & INTERPRETATION**

**APPENDIX**

- CPT Plots
- CPT Classification/Soil Behavior Chart
- CPT Data Files (sent via email)

# SUMMARY
# OF
# CONE PENETRATION TEST DATA

## 1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Riverside Police Department project located at 4102 Orange Street in Riverside, California. The work was performed by Kehoe Testing & Engineering (KTE) on May 13, 2026. The scope of work was performed as directed by Moore Twining Associates, Inc. personnel.

## 2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at three locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

| LOCATION | DEPTH OF CPT (ft) | COMMENTS/NOTES: |
|---|---|---|
| **CPT-1** | 75 | |
| **CPT-2** | 61 | Refusal |
| **CPT-3** | 61 | Refusal |

**TABLE 2.1 - Summary of CPT Soundings**

## 3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone with a cone net area ratio of 0.83. The following parameters were recorded at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

## 4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING



Steven P. Kehoe
President

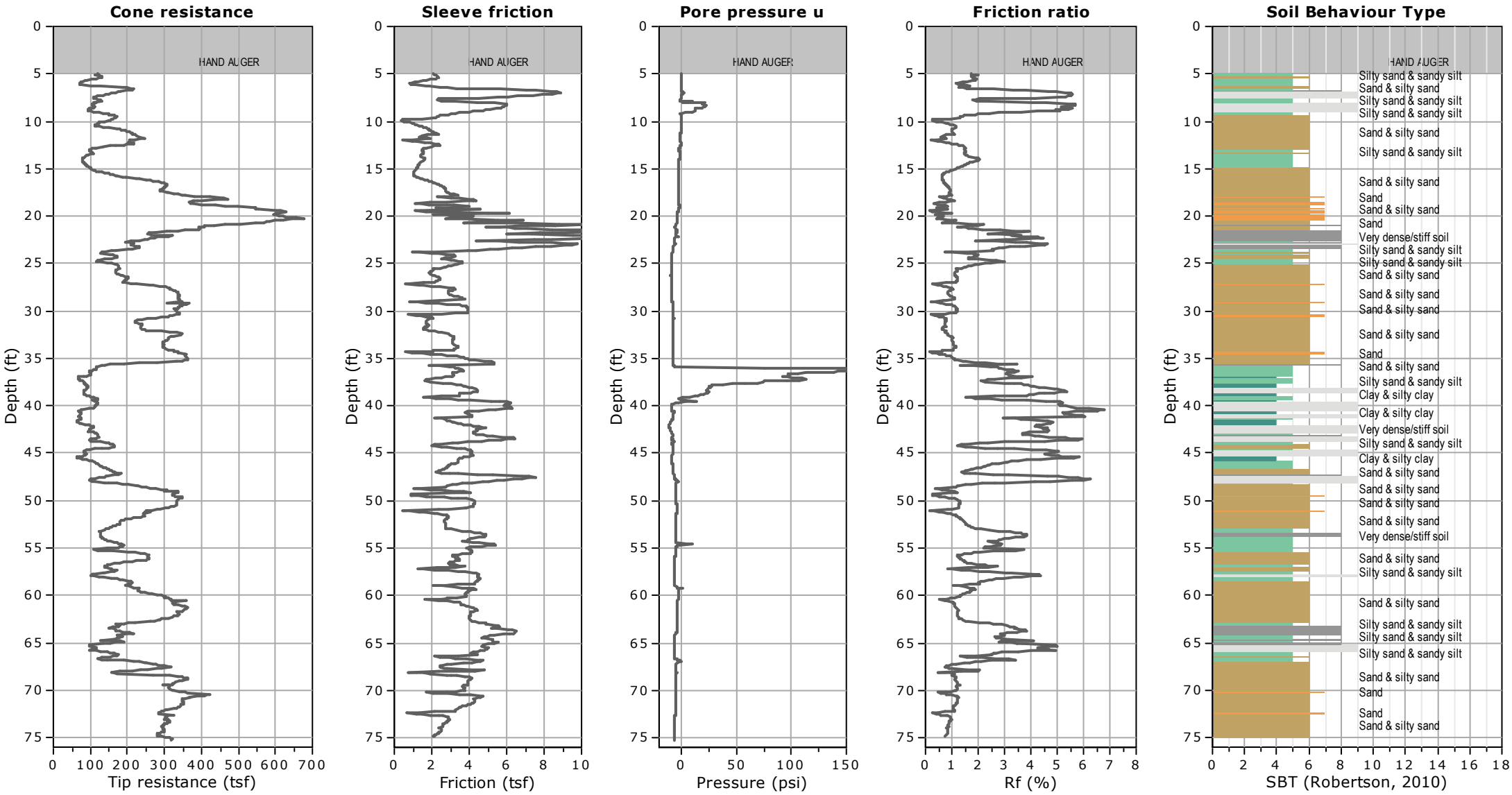
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# APPENDIX

**Kehoe Testing and Engineering**
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

**Project:** **Moore Twining Associates / Riverside Police Department**
**Location:** **4102 Orange St, Riverside, CA**

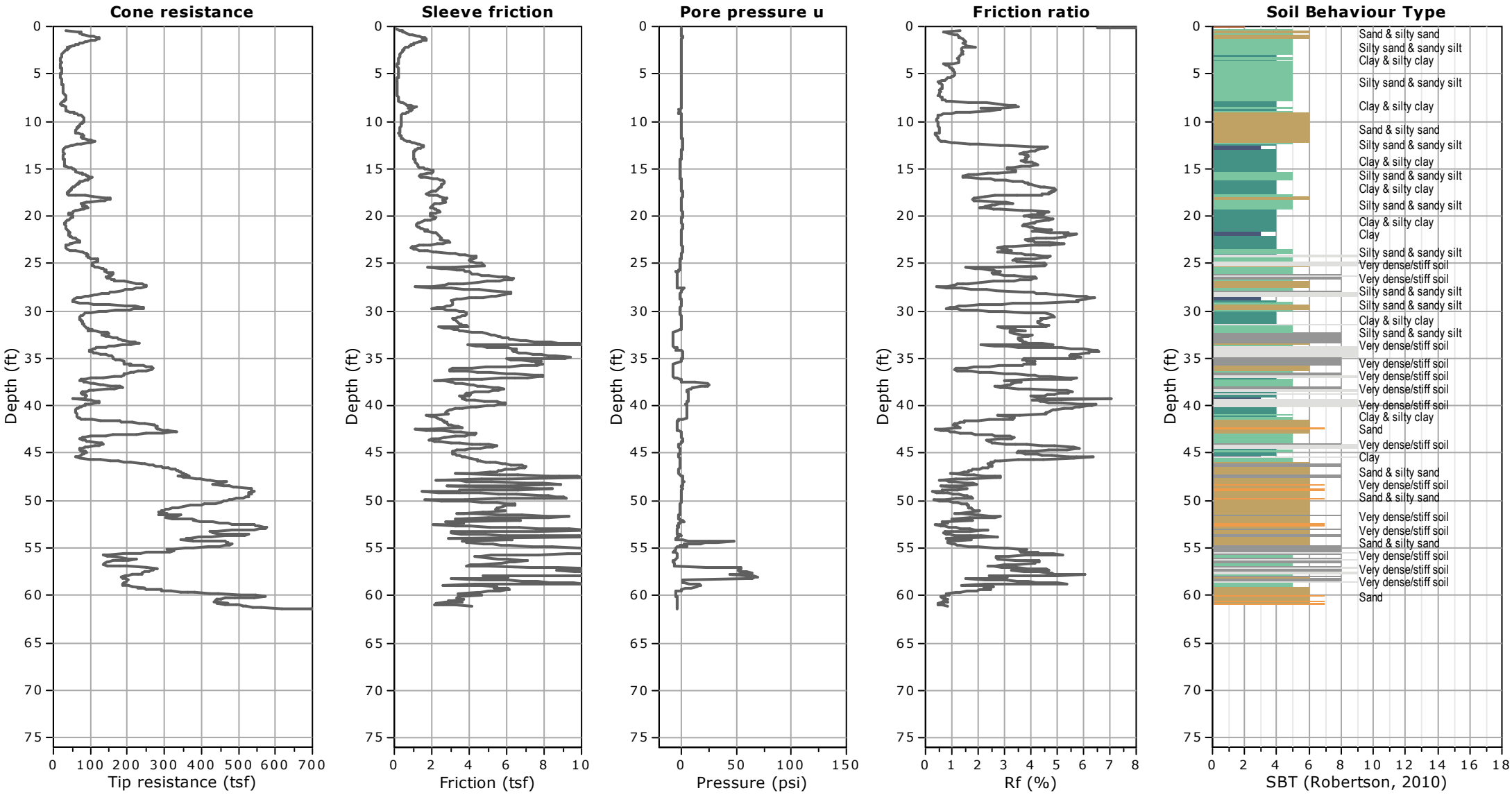
**CPT-1**
Total depth: 75.26 ft, Date: 5/13/2026



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**Location:** **4102 Orange St, Riverside, CA**

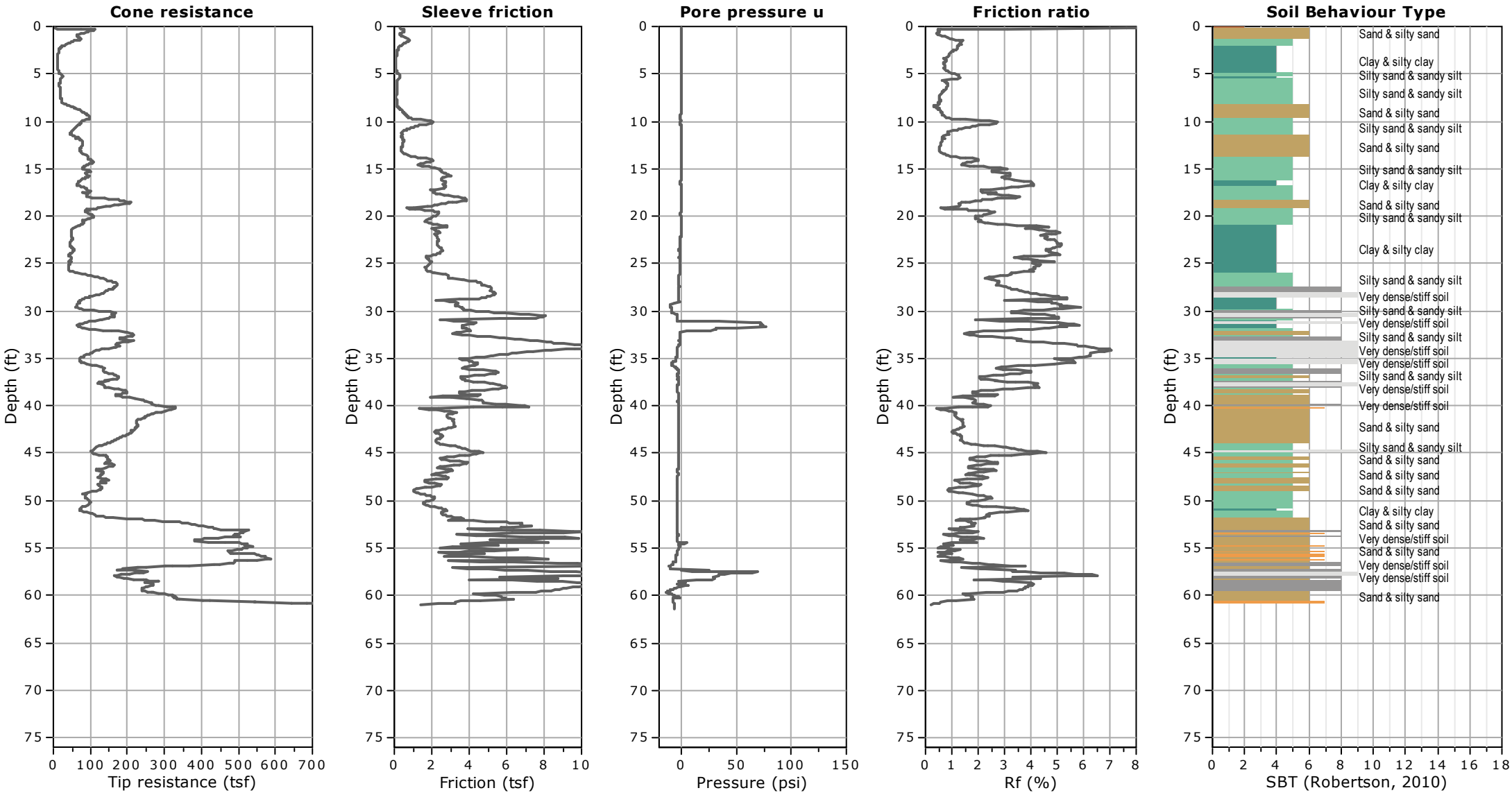
**CPT-2**
Total depth: 61.48 ft, Date: 5/13/2026



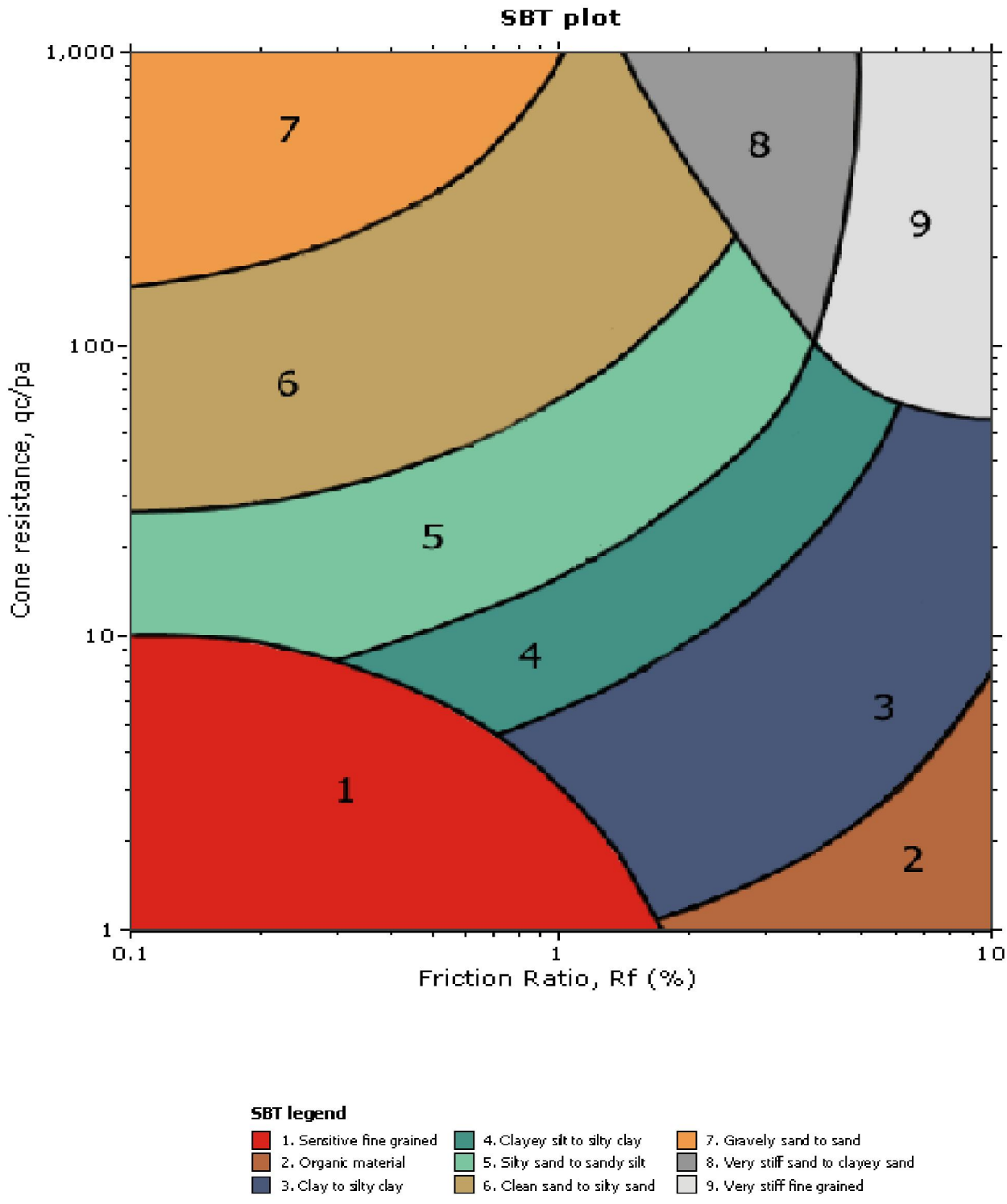
**Kehoe Testing and Engineering**
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

**Project:** **Moore Twining Associates / Riverside Police Department**
**Location:** **4102 Orange St, Riverside, CA**

**CPT-3**
Total depth: 61.36 ft, Date: 5/13/2026



**Kehoe Testing and Engineering**
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com



# APPENDIX C

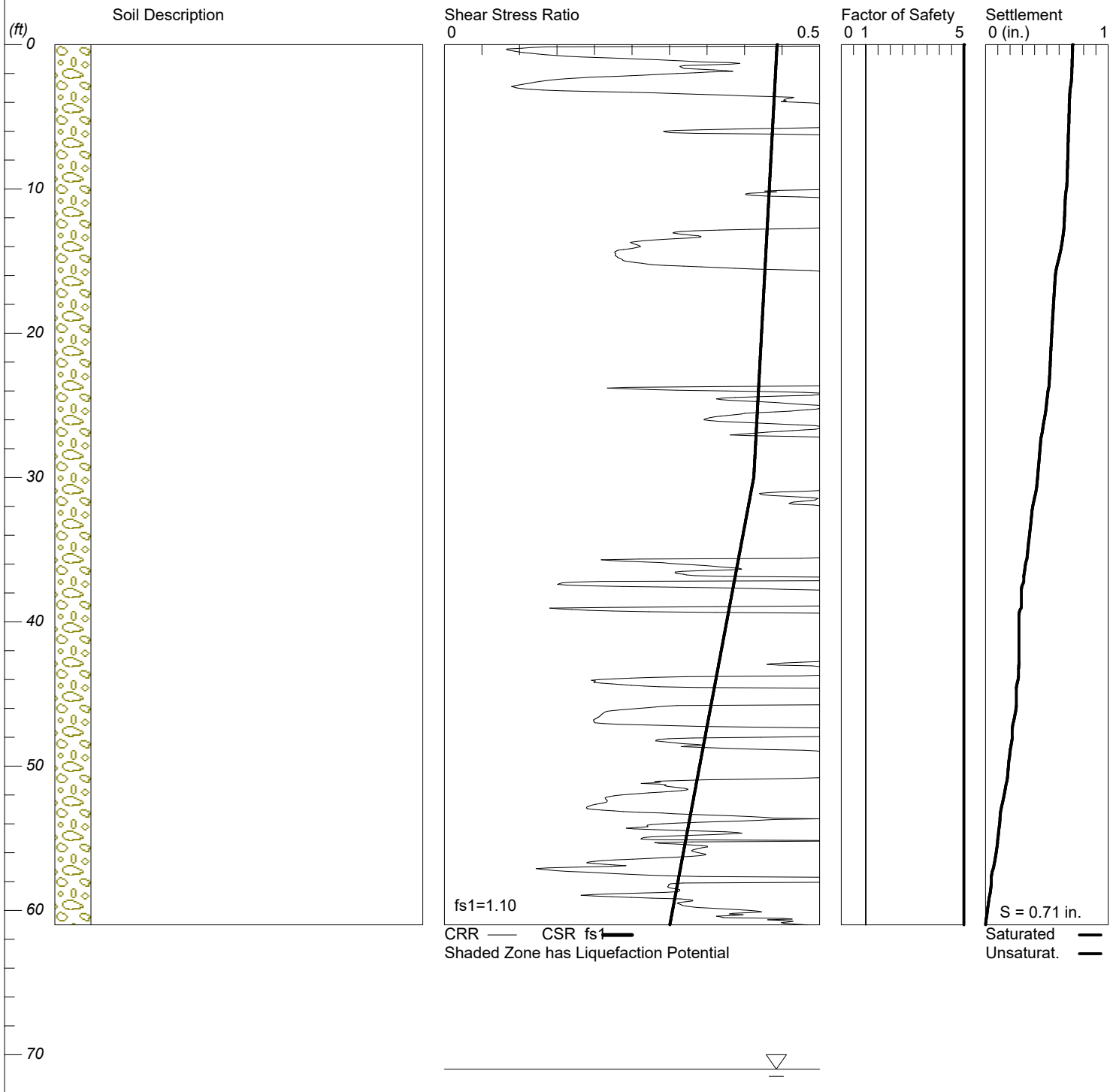
## SEISMIC SETTLEMENT GRAPHICAL ANALYSIS

# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

***Hole No.=CPT-01  Water Depth=71 ft***

***Magnitude=8.2***
***Acceleration=0.62g***



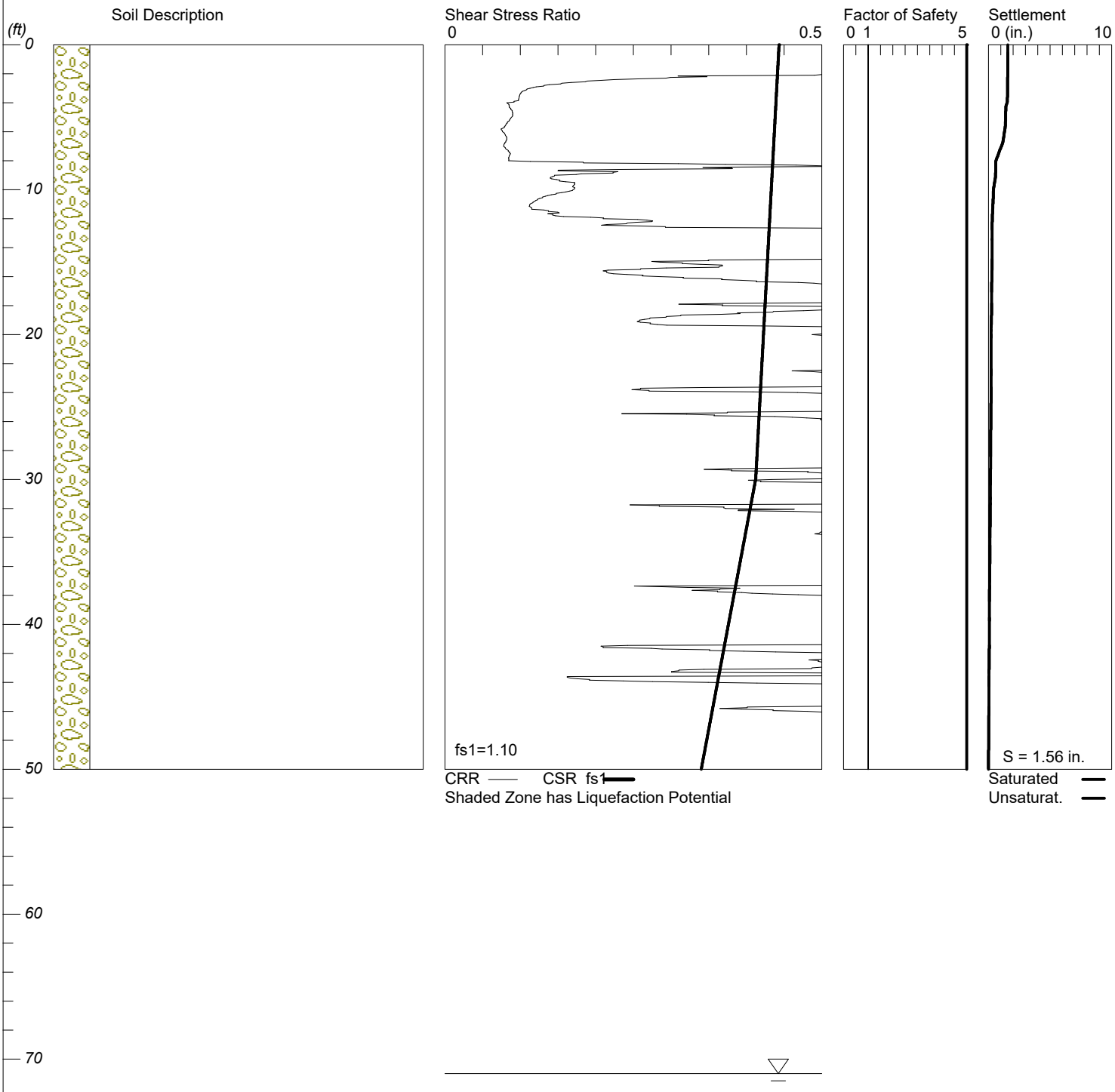

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# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

***Hole No.=CPT-02  Water Depth=71 ft***

***Magnitude=8.2***
***Acceleration=0.62g***



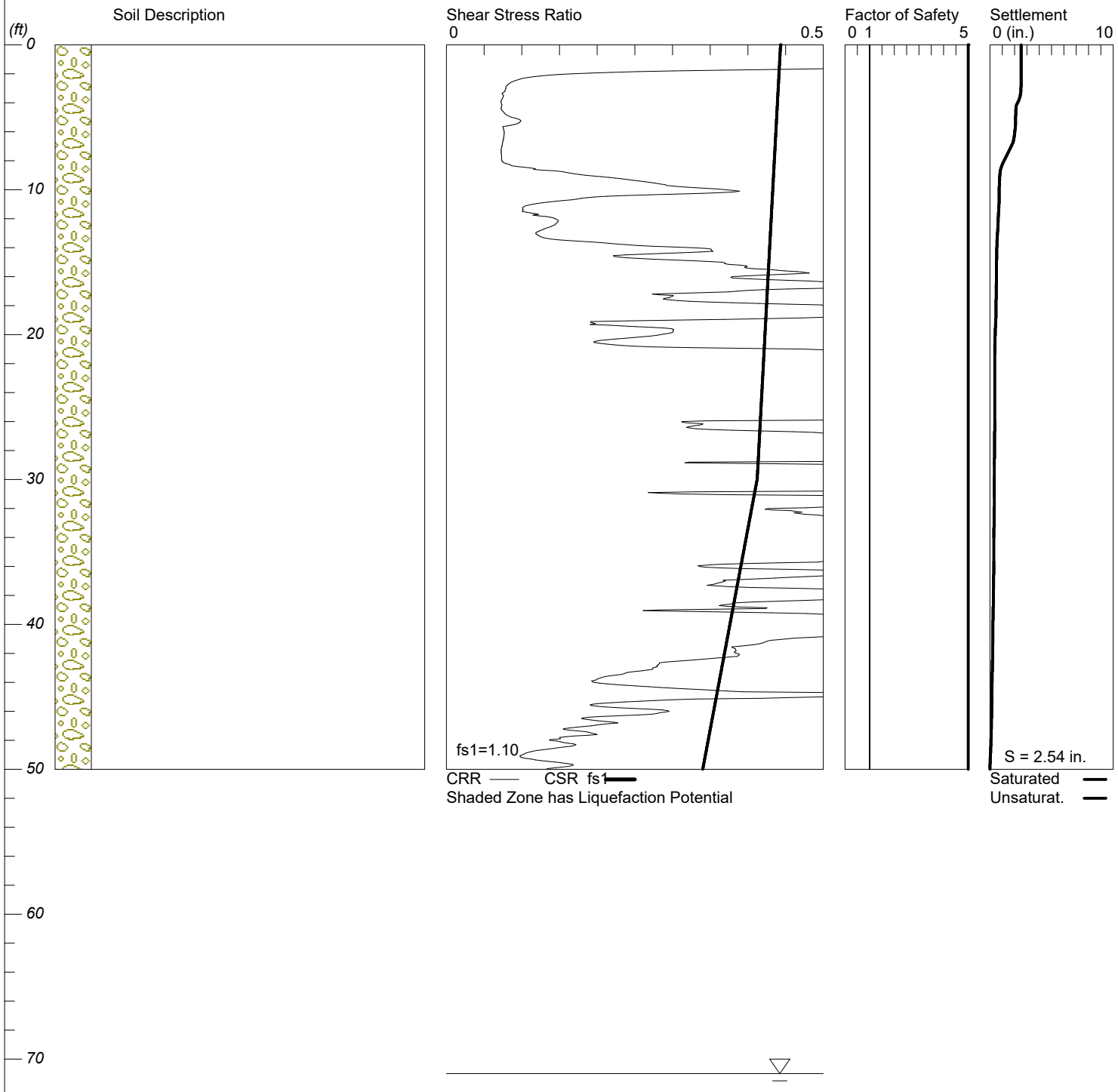

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# LIQUEFACTION ANALYSIS

## Proposed Replacement Police Dept HQ Bldg

***Hole No.=CPT-03*** ***Water Depth=71 ft***

***Magnitude=8.2***
***Acceleration=0.62g***




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