GEOTECHNICAL ENGINEERING INVESTIGATION

THE NEIGHBORHOODS AT LUGONIA VILLAGE

NORTHWEST CORNER OF WEST LUGONIA AVENUE

AND KARON STREET

REDLANDS, CALIFORNIA

Project Number: H02901.01

For:

The Planning Associates Group
9880 Irvine Center Drive, Suite 100
Irvine, CA 92618

February 11, 2022

February 11, 2022 H02901.01

Redlands Summit, LLC
Attn: Darin Zhang, Manager
P.O. Box 80458
San Marino, CA 91118

Subject: **Geotechnical Engineering Investigation**
The Neighborhoods at Lugonia Village
Northwest Corner of West Lugonia Avenue and Karon Street
Redlands, California

Dear Mr. Zhang:

We are pleased to submit this geotechnical engineering investigation report prepared for the Neighborhoods at Lugonia Village to be located at the northwest corner of West Lugonia Avenue and Karon Street in Redlands, California.

The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations. Moore Twining should be retained to review those portions of the plans and specifications that pertain to earthwork, pavements, 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.

In addition, it is recommended that Moore Twining be retained to provide inspection and testing services for the excavation, earthwork, pavement, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the construction complies with our recommendations. These services are not, however, part of this current contractual agreement. A representative with our firm will contact you in the near future regarding these services.

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

We appreciate the opportunity to be of service to Redlands Summit, LLC. 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.

DRAFT

Allen H. Harker, PG
Project Geologist
Geotechnical Engineering Division

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for the proposed Neighborhoods at Lugonia Village to be located at the northwest corner of West Lugonia Avenue and Karon Street in Redlands, California.

The Land Use/Conceptual Site Plan for The Neighborhoods at Lugonia Village, prepared by The Lindom Company, dated April 5, 2021, indicate the proposed development will be divided into four neighborhoods, identified as Neighborhoods A, B, C and D. The developments are anticipated to consist of 2-story and 3-story buildings with carports, and various larger Club/Leasing and Day Care/Gymnasium buildings. The developments are anticipated to include a swimming pool and associated cabana/restroom and pool equipment facilities.

At the time of our investigation, the site was generally vacant land covered by scattered dead grasses and weeds. Some scattered concrete debris was also noted throughout the site. Where existing vegetation and landscaping is present, these areas should be stripped of all vegetation and top soil, and removal of vegetation should remove all roots greater than ¼ inch in diameter. Debris such as concrete debris should be removed from the site and not mixed with on-site soils.

Drainage/irrigation structures were noted on the north and west sides of the site. A debris pit was noted in the northeastern corner of the site, and a dry well was noted in the southwestern portion of the site. In addition, an electrical easement exists at the site. These on-site features should be completely removed and any piping or underground utilities (if any) should be removed from the site and not mixed with soils to be used as engineered fill. Recycled materials including asphalt, concrete and brick should not be mixed in with soils to be used as engineered fill below buildings; however, these materials may be processed to less than 6 inches in size and mixed in with soils to be used as engineered fill outside of building areas. Rodent burrows were also noted throughout the site, some of which extended about 12 inches in depth. The rodent burrows should be over-excavated until undisturbed soils are encountered.

Between November 15 and 18, 2021, thirty-one (31) test borings (B-1 through B-31) were drilled at the site in the proposed building areas to depths ranging from about 15 to 50 feet below site grades (BSG). In addition, four (4) percolation test borings were drilled at the site near the four (4) corners of Neighborhood A to depths ranging from about 10 to 15 feet BSG. On November 17, 2021, five (5) Cone Penetration Tests (CPTs) were advanced at the site to depths of about 50 feet BSG. CPT-3 encountered refusal due to high tip resistance at a depth of about 28 feet BSG. CPT-3 was advanced a second time about five (5) feet away from the initial location and was advanced to a depth of 50 feet.

The soils encountered generally consisted of silty sands extending to varying depths and overlying interbedded layers of poorly graded sands, poorly graded sands with silt and additional silty sand layers extending to the maximum depth explored, about 50 feet BSG. The Cone Penetration Test soundings generally encountered a soil behavior type described as sand to silty sand extending to the maximum depth explored (50 feet BSG). The soil behavior types described from the CPT soundings were generally similar to the soils encountered in the borings.

EXECUTIVE SUMMARY (continued)

Groundwater was not encountered in the test borings drilled at the time of our November 15 through 18, 2021 field exploration to the maximum depth explored, about 51½ feet BSG.

Based on the field and laboratory investigation, the near surface soils tested possess a very low expansion potential, low to moderate compressibility characteristics, slight collapse potential, moderate to high shear strength characteristics and excellent pavement support characteristics when compacted as engineered fill.

In order to limit the potential for excessive differential static settlement of the building foundations, over-excavation and compaction of the near surface soils is recommended to support new foundations on engineered fill. Static settlements of 1 inch total and ½ inch differential should be anticipated for foundations supported on subgrade soils prepared in accordance with the recommendations of this report.

However, this investigation identified a significant potential for "dry" seismic settlement at the site. The seismic settlements were estimated to range from about 3 to 6 inches total and about 1½ to 3 inches differential in 40 feet. These estimated seismic settlements should be considered by the building designer (structural engineer) to determine whether a conventional spread foundation system or reinforced mat/slab foundation system can tolerate this magnitude of settlement for the proposed structures. Based on a conference call on February 4, 2022 to discuss this issue with the design team, a rigid post-tensioned slab is expected to be the preferred approach to be provide foundation design that can tolerate the seismic settlements noted in this report.

In the event that the predicted differential seismic settlement cannot be resisted by the foundation system, alternative methods of site preparation could be utilized to mitigate or reduce differential seismic settlements. Discussions during this investigation concluded that ground modification, or other means to mitigate or reduce the anticipated seismic settlement, are not feasible for the numerous smaller residential structures planned. So recommendations for ground modification were not included in the scope of this initial geotechnical investigation. If mitigation of seismic settlements needs to be evaluated for the larger structures, Moore Twining should be contacted to provide supplemental investigations of those locations to further evaluate subsurface conditions and provide recommendations for ground modification.

It is understood that the project may include construction of onsite infiltration system(s). The location of the proposed infiltration system(s) were not known at the time of preparation of this report. For feasability purposes, percolation tests were conducted. Two (2) tests conducted at a depth of about 10 feet BSG for the proposed infiltration systems indicated unfactored infiltration rates of 0.9 inches per hour for percolation test P-3 in the southwestern portion of the site and 7.1 inches per hour for P-2 in the northeastern portion of the site (both tests conducted in poorly graded sand with silt soils). In addition, percolation tests conducted at a depth of about 15 feet BSG for the proposed infiltration systems indicated unfactored infiltration rates of 2.6 inches per hour for P-4 in the northwestern portion of the site and 15.9 inches per hour for P-1 in the southeastern portion of the site. The results are considered preliminary and confirmation tests will be needed by conducting double-ring infiltration tests when the location and depth of the infiltration systems are known.

Chemical testing of soil samples indicated the soils exhibit a “mildly corrosive” to “essentially non-corrosive” corrosion potential.

Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318-14, the sulfate concentration from chemical testing of soil samples falls in the S0 classification (less than 0.10 percent by weight) for concrete.

The site is not located in an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture on the site is estimated to be low.

This Executive Summary should not be used for design or construction and should be reviewed in conjunction with the attached report.

TABLE OF CONTENTS

Page

TABLE OF CONTENTS

Page

GEOTECHNICAL ENGINEERING INVESTIGATION

THE NEIGHBORHOODS AT LUGONIA VILLAGE

NORTHWEST CORNER OF WEST LUGONIA AVENUE

AND KARON STREET

REDLANDS, CALIFORNIA

Project Number: H02901.01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the Neighborhoods at Lugonia Village to be located at the subject property in Redlands, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Redlands Summit, LLC to perform this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, site description, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, 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 percolation tests (Appendix D), and photographs (Appendix E).

The Geotechnical Engineering Division of Moore Twining, headquartered in Fresno, California, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

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

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations, exterior slabs-on-grade, and pavements with regard to the anticipated foundation and traffic loads;

2.1.2 Recommendations for 2019 California Building Code seismic coefficients and earthquake spectral response acceleration values;

2.1.3 Geotechnical engineering parameters for use in design of foundations and slabs-on-grade, (e.g., soil bearing capacity and settlement);

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

2.1.5 Recommendations for the design and construction of new asphaltic concrete (AC) and Portland cement concrete (PCC) pavements;

2.1.6 Results of percolation tests, estimated infiltration rates, and general recommendations for infiltration systems;

2.1.7 Recommendations for temporary excavations and trench backfill; and

2.1.8 Conclusions regarding soil corrosion potential.

This report is provided specifically for the project described in the Anticipated Construction section of this report. This investigation did not include a full geologic/seismic hazards evaluation, flood plain investigation, compaction tests, environmental investigation, nor an environmental audit.

2.2 **<u>Scope</u>:** Our revised proposal, dated September 24, 2021, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A Land Use/Conceptual Site Plan for The Neighborhoods at Lugonia Village, prepared by The Lindom Company, dated April 5, 2021, was reviewed. An updated Site Plan for The Neighborhoods at Lugonia Village, prepared by AO Architects, dated November 9, 2021, was also provided for review to gain an understanding of the proposed structures.

2.2.2 An ALTA/NSPS Land Title Survey, dated August 2, 2021, prepared by On Point Land Surveying, Inc., was reviewed.

2.2.3 An undated utility map, provided by Mr. Wayne Pena (DRC Engineering, Inc.) was reviewed to locate planned boring locations prior to the field investigation.

2.2.4 A visual site reconnaissance and subsurface exploration were conducted.

2.2.5 Satellite images of the site between the years 1985 and 2021 from online sources were reviewed.

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

2.2.7 Y.Y. Lin (Lindom Company), Mr. Mark Van Gaale (VCA Structural) and Mr. Wayne Pena (DRC Engineering, Inc.) were consulted during the investigation.

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

2.2.9 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

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

3.1 Site Description: The Neighborhoods at Lugonia Village is to be located on a rectangular-shaped property located northwest of the intersection of West Lugonia Avenue and Karon Street in Redlands, California. The overall site has a total gross area of about 24.4 acres. The general site location is noted on Drawing No. 1 in Appendix A of this report.

The site is bounded by West Lugonia Avenue to the south, Karon Street to the east, a future extension of Pennsylvania Avenue to the north, and vacant land to the west with the existing Tennessee Street beyond.

The site is generally vacant land covered by scattered dead grasses and weeds. Some scattered concrete debris was also noted throughout the site. Also some remnant elements of past structures/improvements were noted. A rectangular-shaped open concrete structure, about 6 feet by 9.3 feet in lateral dimension by about 1 foot in height and filled with debris consisting of vegetation, wood, a car tires, was noted in the northeast corner of the site. On the north side of this concrete structure is a concrete drainage structure that measured to be about 4.5 feet wide on the inside and about 12 inches in height. The drainage structure trends from the center line of future Pennsylvania Avenue at the northeast corner of the site and extends about 320 feet to the west where the channel turns to the north and continues off-site.

Another concrete drainage/irrigation structure was located along the western property line and trends from north to south across the majority of the western property boundary line. Portions of the concrete structure consists of an open channel structure that is about 1.5 feet wide, while other portions include a round shallow concrete pipe.

An open cylindrical excavation (thought to be a dry well) was noted in the southwestern portion of the site. The dry well was about 5 feet in diameter and the sides of the structure was lined with brick and concrete. The dry well was filled with concrete debris; thus, the depth of the hole could not be determined. The depth to the top of the debris was about 4.5 to 5 feet below grade. An approximate 12 to 18-inch diameter pipe was noted as extending perpendicular away from the shaft near the bottom just above the debris. A smaller, approximate 4-inch diameter pipe was noted as extending perpendicular away from the dry well near the top of the well. It is assumed that the dry well may be associated with some of the past improvements noted at the site.

The ALTA/NSPS Land Title Survey, dated August 2, 2021, prepared by On Point Land Surveying, Inc., identifies a 5-foot wide electrical easement that trends in a north to south orientation in the middle portion of the site. Based on our site observations, the southern portion of the electrical easement includes a concrete structure with approximate 6-inch high concrete sidewalls on both side of the 2-foot wide easement. Mr. Wayne Pena (DRC) indicated that this is a Southern California Edison (SCE) easement and did not think there was any underground line present within the easement.

Rodent burrows were also noted throughout the site, some of which extended about 12 inches, and deeper, below site grade.

Some of the features described above are shown in pictures included in Appendix D and are also shown on Drawing No. 2 in Appendix A of this report.

The grading of the site slopes gently down to the west. The ALTA/NSPS Land Title Survey indicates that the site ranges in elevation from about 1,289 feet above mean sea level (AMSL) in the western portion of the site to about 1,307 feet AMSL in the eastern portion of the site, adjacent to Karon Street. Portions of the eastern side of the site include a west-facing slope that descends away from Karon Street and has a maximum height of about 5 feet.

3.2 **Site History:** Satellite images of the site between the years 1985 and 2021 from online sources were reviewed. The site has been vacant land dating back to 1985. About 18 trees were noted in the western-third of the site as shown in satellite images through December 2005. The next satellite image in January 2006 shows all but one of these trees was removed. Some of the trees in the western-third of the site grew back but then were removed again sometime between 2016 and 2018.

3.3 **Previous Studies:** No previous geotechnical engineering, geological, compaction reports, 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.

3.4 <u>Anticipated Construction</u>: The Land Use/Conceptual Site Plan for The Neighborhoods at Lugonia Village indicates the development will be divided into four neighborhoods, identified as Neighborhoods A, B, C and D.

Neighborhood A was shown on the Land Use/Conceptual Site Plan to include 2-story and 3-story apartment buildings. However, the updated site plan, prepared by AO Architects, dated November 9, 2021, shows the these apartment buildings will have adjacent covered garages. The latest site plan shows sixteen (16) apartment buildings (buildings 1 through 16) planned for Neighborhood A with 92 units across about 14.49 acres. In addition, smaller buildings 5A, 7A, 9A, 12A, and 14A appear to be additional buildings with ground floor parking and apartments above. In addition, ten (10) six-stall carports are planned throughout the apartment building layout, and seventeen (17) eight-stall carports are planned around the perimeter of the apartment building layout. The carports are anticipated to be supported on cast-in-drilled-hole pier foundations. Neighborhood A also includes a 5,932 square foot Club and Leasing building, a Kid Care/Gymnasium building, a swimming pool, and a cabana/restroom and pool equipment building. Other improvements for Neighborhood A include at 1,637 square foot mail room building, a tot lot and a dog park.

Neighborhood B is planned north of Neighborhood A, and according to the Land Use/Conceptual Site Plan will include 2-story and 3-story townhomes with 2-car side-by-side garages. The updated site plan shows fifty-seven (57) townhomes planned for Neighborhood B. A club house building, swimming pool, and dog park are also planned in Neighborhood B. Neighborhood B is estimated to occupy about 4.7 acres.

Neighborhood C is planned on the east side of both Neighborhoods A and B and according to the Land Use/Conceptual Site Plan will include detached single family homes. The latest site plan shows twenty (20) three-story and four-story detached single family homes on 7,200 square foot lots across to occupy about 3.49 acres.

Neighborhood D was originally planned as a Day Care building and play yard in the southeast portion of the site. However, the updated Site Plan for The Neighborhoods at Lugonia Village, prepared by AO Architects, dated November 9, 2021, indicates that the these improvements have been incorporated into Neighborhood A.

In addition, it is understood that the project may include construction of onsite infiltration system(s). The location, types and details of proposed infiltration system(s) were not known at the time of preparation of this report.

For the purpose of evaluating foundation support for this report, preliminary maximum column loads of about 40 kips and maximum perimeter wall loads of 3 kips per linear foot were assumed. However, since the residential structures may be supported on a rigid mat/slab foundation system

Mr. Mark Van Gaale (VCA Structural) reported the maximum loading would be around 270 pounds per square foot for a typical three-story residential structure. This maximum loading includes dead plus live loads but does not include the load of any slab/foundations. The actual design foundation loads should be provided to Moore Twining when available. In the event that the maximum foundation loads exceed those assumed for design, the recommendations of this report may not be applicable and may need to be revised.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, conducting standard penetration tests, cone penetration testing, soil sampling and percolation testing.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Yaman Al Ahmed of Moore Twining between November 15 and 18, 2021. The features noted are described in the background information section of this report.

4.1.2 Drilling Test Borings: Prior to drilling, the site was marked for Underground Service Alert for members to mark utility locations.

The depths and locations of the test borings were selected based on the size of the structures, type of construction, estimated depths of influence of the anticipated foundation loads, and the subsurface soil conditions encountered.

Between November 15 and 18, 2021, thirty-one (31) test borings (B-1 through B-31) were drilled at the site in the proposed building areas to depths ranging from about 15 to 50 feet below site grades (BSG). In addition, four (4) percolation test borings were drilled at the site near the four (4) corners of Neighborhood A to depths ranging from about 10 to 15 feet BSG. The borings were drilled with a conventional truck-mounted CME-75 drill rig equipped with 6⅝ and 8-inch outside diameter (O.D.) hollow-stem augers.

During the drilling of the test borings, bulk samples of soil were obtained for laboratory testing. The test borings were drilled under the direction of a Moore Twining professional geologist. The soils encountered in the test borings were logged during drilling by a representative of our firm. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of the borings.

Test boring locations were determined with reference to existing site features shown on the site plan. The locations, as described, should be considered approximate. The locations of the test borings are shown on Drawing No. 2 in Appendix A. The test borings were loosely backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated at the boring locations.

4.1.3 **<u>Soil Sampling</u>:** Standard penetration tests were conducted in the test borings, 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 has a 2-inch O.D. and a 1⅜-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 pushing or driving a California modified split barrel ring 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, airtight 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 **<u>Cone Penetration Testing</u>:** On November 17, 2021, five (5) Cone Penetration Tests (CPTs) were advanced at the site to depths of about 50 feet BSG. CPT-3 encountered refusal due to high tip resistance at a depth of about 28 feet BSG. CPT-3 was advanced a second time about five (5) feet away from the initial location and was advanced to a depth of 50 feet. The CPTs were conducted at the locations shown on Drawing No. 2 in Appendix A.

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. Measurements of cone tip resistance and sleeve friction data were recorded at approximate 2 inch intervals during penetration to provide nearly continuous data for interpreting the engineering properties of the soils. The CPT logs are presented in Appendix B of this report. The 50-foot deep CPTs were backfilled with bentonite granules.

4.1.5 **<u>Percolation Testing</u>:** Percolation tests were conducted in the four (4) borings (P-1 through P-4) where shown on Drawing No. 2 in Appendix A of this report. The locations and depths of the percolation tests were provided by Mr. Wayne Pena (DRC). The percolation test borings were drilled to depths of approximately 10 to 15 feet below site grade with a truck-mounted CME-75 drill rig equipped with 8-inch outside diameter (O.D.) hollow-stem augers. The percolation tests were conducted within the boreholes and infiltration rates were estimated using the percolation test data.

The percolation tests were conducted on November 18 and 19, 2021 in accordance with the percolation test procedure noted in the San Bernardino County Water Quality Management Plan Technical Guidance Document (TGD), dated June 21, 2013. The percolation tests were constructed in conformance with Section VII.3.8.1 "Shallow Percolation Test (Less than 10 feet)" of the TGD. The holes were cylindrical with a diameter of 8 inches. A gravel layer about 2 inches thick was placed at the bottom to limit washout during refilling. A 2-inch diameter perforated PVC pipe was placed in the boreholes and used to transmit poured water to the bottom of the holes. Each test was pre-soaked with 5 gallons of water or to at least 5 times the hole radius (20 inches) until all water had percolated; or, to a duration of at least 15 hours. Details of the percolation test construction and pre-soak times and conditions are indicated on the enclosed data sheets for each percolation test.

After the presoak, the pre-test trials were conducted to determine if the "sandy soil" procedures were applicable. The initial percolation at all of the locations indicated that two consecutive measurements showed at least 6 inches of water seeped away in less than 25 minutes, so the tests were conducted utilizing the "sandy soil" procedure with a minimum of six ten (10) minute readings. At the start of each test "trial", the water level was refilled to a minimum height of 20 inches. These initial 25 minute measurements and the six 10 minute measurements are included on the enclosed data sheets for each percolation test.

4.2 **<u>Laboratory Testing</u>:** The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface soils encountered.

The results of laboratory tests are summarized 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 **<u>Soil Profile</u>:** The soils encountered in the borings conducted for this investigation generally consisted of silty sands extending to varying depths and generally overlying interbedded layers of poorly graded sands, poorly graded sands with silt and additional silty sand layers extending to the maximum depth explored, about 51½ feet BSG. However, in one portion along the west portion of the site (B-30) the silty sands extended to the maximum depth explored of 20 feet BSG. Also, in a central area where the deepest boring was drilled (B-22), poorly graded sands with silt were encountered at the surface to a depth of 3½ feet BSG, and were underlain by more silty sands from 3½ to 28½ feet BSG, which were underlain by poorly graded sands with silt extending to the maximum depth explored of 51½ feet BSG. The soil layers encountered typically included small amounts of gravel (about 11 percent or less). The soils survey maps, prepared by the U.S. Department of Agriculture, do not indicate the presence of any cobbles or boulders in the upper 5 feet BSG.

The Cone Penetration Test soundings generally encountered a soil behavior type described as sand to silty sand extending to the maximum depth explored (50 feet BSG). However, interbeds described as silty sand to sandy silt were also encountered and were generally up to about 1 to 2 feet in thickness; however, silty sand to sandy silt soils were encountered in the upper 7 feet of CPT-1 located in the northeast portion of the site . Very thin interbeds of gravelly sand to sand that were less than 1 foot in thickness were also encountered between the depths of about 17 to 20 feet BSG and at about 47½ to 48 feet BSG in CPT-3 in the southwest portion of the site. In addition, occasional very thin interbeds (6 inches thick or less) of clayey silt to silty clay were also encountered at depths of about 31 feet in CPT-1, 33 feet in CPT-5 and at the ground surface in some of the CPT's. The soil behavior types described from the CPT soundings were generally similar to the soils encountered in the borings.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring location are presented in the logs of borings in Appendix B. The stratification lines in the logs represent the approximate boundary soil types; the actual in-situ transition may be gradual.

5.2 Soil Engineering Properties: The following is a description of the soil engineering properties as determined from our field exploration and laboratory testing.

Silty Sands: The silty sands with varying amounts of gravel (about 8 percent or less) were described as very loose to dense, as determined by standard penetration resistance, N-values, ranging from 2 to 17 blows per foot, and equivalent N-values (estimated by driving a California Modified split barrel sampler) ranging from 4 to 34 blows per foot. The moisture content of the samples tested ranged from about 1 to 7 percent. Nineteen (19) relatively undisturbed samples revealed dry densities ranging from 94.3 to 113.9 pounds per cubic foot. Seven (7) Atterberg Limits tests conducted on silty sand samples at various depths indicated that all of the samples were non-viscous and non-plastic. Ten (10) consolidation tests conducted on near surface samples collected in the upper 6½ feet BSG indicated low to moderate compressibility characteristics (ranging from 2.9 to 7.0 percent consolidation under a load of 8 kips per square foot). Upon inundation, the consolidation test samples exhibited slight collapse potential (0.0, 0.4, 0.1, 0.4, 0.1, 0.7, 1.0, 0.0, and 0.1 percent collapse when wetted under loads of either 0.5 or 1 kip per square foot). Four (4) direct shear tests conducted on a sample collected from borings B-5, B-14, B-22 and B-24 in the upper 6½ feet BSG indicated internal angles of friction of 39, 39, 31 and 31 degrees and 160, 0, 260 and 180 pounds per square foot of cohesion, respectively.

Poorly Graded Sands and Poorly Graded Sands with Silt: The poorly graded sands and poorly graded sands with silt and varying amounts of gravel (about 11 percent or less) were described as very loose to dense, as determined by standard penetration resistance, N-values ranging from 4 to greater than 35 blows per foot, and equivalent N-values (estimated by driving a California Modified split barrel sampler) ranging from 3 to 31 blows per foot. The moisture content of the samples tested ranged from 1 to 3 percent. Three (3) relatively undisturbed samples revealed dry densities of 101.9, 111.8, and 105.1 pounds per cubic foot. One (1) Atterberg Limits test conducted on a poorly graded sand with silt sample collected from depths of 0 to 1½ feet from boring B-22 indicated the sample was non-viscous and non-plastic.

Expansion Index Tests: Three (3) expansion index tests conducted on bulk samples of silty sand collected in the upper 3½ feet from borings B-3, B-7 and B-25 all indicated expansion index values of 0.

Maximum Density/Optimum Moisture Content Determination: The results of a maximum density/optimum moisture content determination from a silty sand sample collected at depths of 0 to 3½ feet BSG from boring B-10 indicated a maximum dry density of 121.9 pounds per cubic foot at an optimum moisture content of 9.8 percent.

R-value Tests: Four (4) R-value tests conducted on a near surface silty sand samples collected from depths of about 0 to 3½ feet BSG in borings B-1, B-14, B-22 and B-28 indicated R-values of 73, 75, 75 and 72.

Chemical Tests: Chemical tests performed on a near surface silty sand soil sample collected at depths of 0 to 3½ feet BSG from borings B-5, B-16 and B-31 indicated pH values of 7.3, 7.2 and 7.8; and a minimum resistivity values of 19,000; 25,000 and 28,000 ohms-centimeter, respectively. In addition, the same chemical tests performed on the near surface silty sand soil sample collected at depths of 0 to 3½ feet BSG from borings B-5, B-16 and B-31 indicated each indicated less than 0.00060 percent by weight concentrations of sulfate; and the samples from boring B-5 and B-31 each indicated less than 0.00060 percent by weight concentrations of chloride while the sample from boring B-16 indicated 0.00074 percent by weight concentration of chloride.

5.3 Groundwater Conditions: Groundwater was not encountered in the test borings drilled at the time of our November 15 through 18, 2021 field exploration to the maximum depth explored, about 51½ feet BSG. A well located about 1,000 feet northeast of the site indicated groundwater depths ranging from about 165 to 236 BSG for data collected between the years 2011 and 2019. The most recent measurement in June 2019 indicated a groundwater depth of about 227 feet BSG. Another well with more historical data located about ¾ mile southwest of the site indicated groundwater depths ranging from about70 feet BSG in 1928 to about 161 feet BSG in 2005 for sporadic groundwater data collected between the years 1928 and 2008.

It should be recognized, however, that groundwater elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level 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. The evaluation of such factors was beyond the scope of this investigation and report.

5.4 **Percolation Test Results:** The results of the percolation tests are summarized in Table No. 1 below. For the proposed infiltration systems, the percolation tests were conducted at a depth of about 10 and 15 feet BSG within silty sand and poorly graded sand with silt layers. The results of the percolation tests are presented in Appendix D.

It should be noted that the field tests do not take into account the long term effects of subgrade saturation, silt accumulation, groundwater influence, nor vegetation. In general, the infiltration rate of the soils will decrease when the soils are saturated and the reduction in the infiltration rate increases the longer the soils are saturated. Published studies indicate 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 infiltration rate, both short and long term.

It should also be noted that the unfactored infiltration rates shown in Table No. 1 below should be considered preliminary data. When the locations of the underground infiltration systems are known, additional testing will need to be conducted. Based on other projects that we have conducted in the City of Redlands, "Double ring infiltrometer infiltration testing will be required to determine the design infiltration rate. This has been indicated to be a requirement of the final SQMP and is mandatory for all underground storage systems. The tests must be at the same depth as the basin bottom and as near the center of the basin as possible. A minimum of two tests will be required for the basin unless the soils engineer determines that the soils on the site are uniform and homogeneous and then 1 test per basin will be required. Percolation testing is allowed for preliminary purposes, but the values used for final design must come from a double ring infiltrometer infiltration test."

Table No. 1
Results of Percolation Testing

Location and Depth	Percolate Rate (Minutes per Inch)[1]	Unfactored Infiltration Rate (Inches per Hour)[1]	Subgrade Soil Type
P-1 at 15 feet BSG	2.1	15.9	Silty Sand
P-2 at 10 feet BSG	4.8	7.1	Poorly Graded Sand with Silt
P-3 at 10 BSG	4.1	0.9	Poorly Graded Sand with Silt
P-4 at 15 feet BSG	3.2	2.6	Silty Sand

Notes:
BSG - Below site grade
[1] - Includes no factor of safety

6.0 EVALUATION

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 this investigation 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.

6.1 Existing Surface and Subsurface Improvements: At the time of our investigation, the site was generally vacant land covered by scattered dead grasses and weeds. Some scattered concrete debris was also noted throughout the site. Where existing vegetation and landscaping is present, these areas should be stripped of all vegetation and top soil, and removal of vegetation should remove all roots greater than ¼ inch in diameter. Over sized debris such as large chunks of concrete or bricks should be removed from the site and not mixed with on-site soils.

Remnant elements of past structures and irrigation improvements were noted during the field investigation, and there may be additional buried and subsurface structures not noted during this investigation. These elements and any associated fill soils will not provide uniform support of the proposed building and pavement improvements, and should be entirely removed and backfilled as engineered fill as part of demolition and earthwork for site preparation.

The approximate location of the identified features for removal are illustrated on Drawing No. 2 in appendix A. These elements include the following.

A rectangular-shaped open concrete structure was noted in the northeast corner of the site. This particular structure was noted to be about 6 feet by 9.3 feet in lateral dimension with a height of about 1 foot, and had a bottom filled with vegetation, wood, a car tire and other debris. An open cylindrical excavation (possibly a deeper gravel filled dry well), about 5 feet in diameter and lined with brick and concrete along the sides of the hole, was noted in the southwestern portion of the site. The vertical dry well had two horizontal pipes extending away from the shaft. Other features on the site included a concrete drainage structure that measured to be about 4.5 feet wide on the inside and about 12 inches in height and extended along a portion of the northern property boundary (center line of Pennsylvania Avenue) and then continues northward off-site. Another concrete drainage structure was located along the western property line and trends from north to south across the majority of the western property boundary line. Portions of this concrete drainage structure consists of a U-shaped structure that is about 1.5 feet wide, while other portions include a round concrete drainage pipe. A 5-foot wide electrical easement trends in a north to south orientation in the middle portion of the site. Based on our site observations, the southern portion of the electrical easement includes a concrete structure with approximate 6-inch high concrete sidewalls on both side of the 2-foot wide easement.

These on-site features, and any other subsurface structures identified during demolition and earthwork should be completely removed and any piping or underground utilities (if any) should be removed from the site and not mixed with soils to be used as engineered fill. The suspected dry well should be abandoned per state and local requirements as discussed in recommendations section 8.4.12 of this report. Recycled materials including asphalt, concrete and brick should not be mixed in with soils to be used as engineered fill below buildings; however, these materials may be processed to less than 6 inches in size and mixed in with soils to be used as engineered fill outside of building areas.

Rodent burrows were also noted throughout the site, some of which extended about 12 inches in depth. These voids will not provide uniform support of the proposed building and pavement improvements. Thus the burrows should be over-excavated until undisturbed soils are encountered, then the resulting excavations backfilled as engineered fill.

6.2 **<u>Expansive Soils</u>:** In evaluation of the potential for expansive soils at the site, expansion index testing was performed on representative samples 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. The soils tested were classified by expansion potential in accordance with Table 1 of ASTM D4829 and are summarized in Appendix C of this report. The results of expansion index testing indicated that the near surface samples tested are granular in nature and have expansion index values of 0. Therefore, special procedures to address expansive soils concerns are not anticipated for the project.

6.3 **<u>Static Settlement and Bearing Capacity of Shallow Foundations:</u>** 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.

Due to the very loose to loose near surface soils encountered at the site, the noted rodent burrows and possibility of more buried structures, this report recommends that footings for the proposed buildings be supported on two feet of engineered fill soils in order to limit total and differential static settlements of foundations to 1 inch total and ½ inch differential in 40 feet. A net allowable soil bearing pressure of 3,000 pounds per square foot, for dead-plus-live loads, may be used for design of shallow spread foundations. For continuous rigid mat slab foundations, a net allowable soil bearing pressure of 1,000 pounds per square foot, for dead-plus-live loads, may be used.

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.

6.4 **<u>Seismic Ground Rupture and Design Parameters</u>:** The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active fault with known surface rupture is the Live Oak Canyon Fault (part of the Crafton Hills Fault Zone), which is located approximately 3¼ miles southwest of the site. It should be noted that the active San Andreas Fault is located about 4¼ miles northeast of the site. Accordingly, the potential for ground rupture at the site is considered low.

It is our understanding that the 2019 CBC will be used for structural design, and that seismic site coefficients are needed for design.

Based on the 2019 CBC, a Site Class D represents the on-site soil conditions with standard penetration resistance, N-values averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A table providing the recommended seismic coefficient and earthquake spectral response acceleration values for the project site is included in the Foundation Recommendations section of this report. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.844g was determined for the site using the Ground Motion Parameter Calculator provided by the United States Geological Survey (http://earthquake.usgs.gov/designmaps/us/application.php).

6.5 <u>Liquefaction and Seismic Settlement</u>: 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. One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils; however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

The analyses were conducted using the computer program LIQUEFYPRO by Civiltech. A peak horizontal ground acceleration, PGA_M, of 0.844g, a maximum considered earthquake magnitude of 8.09 and a groundwater depth of 70 feet were used in the analysis of the soils encountered in the CPTs that extended to a depth of about 50 feet BSG. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

Since groundwater is anticipated to be much deeper than 50 feet BSG and has historically not been encountered in the upper 50 feet BSG, liquefaction is not considered a concern. However, the analyses indicated that the granular soil layers encountered in the CPTs would be subject to dry seismic settlement. The dense soils (N-values of 30 or greater) from the CPT soundings were not considered to be subject to dry seismic settlement in the analyses. In general, the seismic settlements were estimated to range from about 3 to 6 inches total and about 1½ to 3 inches differential in 40 feet (about half of the computed total seismic settlement). The specific values of the calculated seismic settlement estimates are noted in Table No. 2 below. The majority of the seismic settlement was noted to occur between the depths of about 12 to 42 feet BSG. However, CPT-1 and CPT-4 (both conducted on the east side of the site) also indicated large percentages of seismic settlement in the upper 7 to 8 feet.

Table No. 2
Summary of Dry Seismic Settlement from CPT Soundings

CPT Number	Total Dry Seismic Settlement	Differential Dry Seismic Settlement in 40 feet	Layers with Largest Percentages of Seismic Settlement
CPT-1	5.7 inches	2.8 inches	53% in Upper 8 feet 42% from 18-41 feet
CPT-2	2.7 inches	1.3 inches	85% from 21-39 feet
CPT-3	4.6 inches	2.3 inches	88% from 12-42 feet
CPT-4	3.1 inches	1.6 inches	40% in Upper 7 feet 56% from 21-34 feet
CPT-5	3.0 inches	1.5 inches	79% from 18-41 feet

Based on our experience with other projects, the estimated seismic settlements may be excessive for support of wood-frame structures on conventional shallow spread foundations. These estimated seismic settlements should be considered by the building designer (structural engineer) to determine whether a conventional spread foundation system or reinforced mat/slab foundation system can tolerate this magnitude of settlement for the proposed structures. Based on a conference call on February 4, 2022 to discuss this issue with the design team, a rigid post-tensioned slab is expected to be the preferred approach to be provide foundation design that can tolerate the seismic settlements noted in this report.

In the event that the predicted differential seismic settlement cannot be resisted by the foundation system, alternative methods of site preparation could be utilized to mitigate or reduce differential seismic settlements. Discussions during this investigation concluded that ground modification, or other means to mitigate or reduce the anticipated seismic settlement, are not feasible for the numerous smaller residential structures planned. Thus, recommendations for ground modification were not included in the scope of this initial geotechnical investigation. If mitigation of seismic settlements needs to be evaluated for the larger structures, Moore Twining should be contacted to provide supplemental investigations of those locations to further evaluate subsurface conditions and provide recommendations for ground modification.

6.6 Asphaltic Concrete (AC) Pavements: Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphaltic concrete (AC) pavements. 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 7.0. 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 buildings, 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.

Four (4) R-value tests were conducted on near surface samples, which indicated R-values of 73, 75, 75 and 72. Based on the results of the testing, the procedures of the Caltrans Highway Design Manual and considering the extent of grading planned for the project, an R-value of 50 was used to determine the pavement section thickness recommendations.

6.7 **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 Resistance or R-value 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).

In evaluation of the pavement design for this project, a sample of the near surface soils anticipated to be representative of the soils which will support pavements was obtained and R-value testing performed in accordance with ASTM D2844. The R-value test result is summarized in Appendix C of this report. The R-value testing was used to estimate a modulus of subgrade reaction for the pavement design.

The recommendations provided in this report for PCC pavements are based on a trash truck accessing the trash enclosure area twice a week and daily and the design procedures contained in the Portland Cement Association "Thickness Design of Highway and Street Pavements."

The PCC pavement sections were designed for a life of 20 years, a load safety factor of 1.1, a single axle weight of 20,000 pounds, and a tandem axle weight of 35,000 pounds. A modulus of subgrade reaction, K-value, for the pavement section, of 230 psi/in was used for the pavement design considering the pavements to be underlain by 4 inches of aggregate base.

6.8 **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 National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 3 below.

Table No. 3
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 “mildly corrosive” to “essentially non-corrosive” potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the “mildly 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.

6.9 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. 4
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.

7.0 CONCLUSIONS

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

7.1 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 clearing, and earthwork activities by Moore Twining are integral to this conclusion.

7.2 The soils encountered in the borings conducted for this investigation generally consisted of loose to dense silty sands extending to varying depths and overlying interbedded layers of loose to dense poorly graded sands, poorly graded sands with silt and additional silty sand layers extending to the maximum depth explored, about 51½ feet BSG. However, at isolated locations the silty sands extended to the maximum depth explored of 20 feet BSG and poorly graded sands with silt were encountered at the surface to a depth of 3½ feet BSG, and were underlain by the typical soil profile described above. The soil layers encountered typically included small amounts of gravel (about 11 percent or less).

7.3 Based on our field and laboratory investigation, the near surface soils tested possess a very low expansion potential, low to moderate compressibility characteristics, slight collapse potential, moderate to high shear strength characteristics and excellent pavement support characteristics when compacted as engineered fill.

7.4 Groundwater was not encountered in the test borings drilled at the time of our November 15 and 18, 2021field exploration to the maximum depth explored, about 51½ feet BSG. Research of nearby well data site indicated groundwater depths ranging from about 165 to 236 BSG for data collected between the years 2011 and 2019; and historical data at another nearby well indicated groundwater depths ranging from about70 feet BSG in 1928 to about 161 feet BSG in 2005.

7.5 Percolation tests conducted at a depth of about 10 feet BSG for the proposed infiltration systems indicated unfactored infiltration rates of 0.9 inches per hour for percolation test P-3 in the southwestern portion of the site and 7.1 inches per hour for P-2 in the northeastern portion of the site (both tests conducted in poorly graded sand with silt soils). In addition, percolation tests conducted at a depth of about 15 feet BSG for the proposed infiltration systems indicated unfactored infiltration rates of 2.6 inches per hour for P-4 in the northwestern portion of the site and 15.9 inches per hour for P-1 in the southeastern portion of the site. It should also be noted that the unfactored infiltration rates listed above should be considered preliminary data. When the locations of the underground infiltration systems are known, additional testing will need to be conducted to meet agency site development requirements.

7.6 Seismic settlement analyses indicate that the loose to medium dense granular soil layers encountered would be subject to significant dry seismic settlement under the design earthquake. In general, the seismic settlements were estimated to range from about 3 to 6 inches total and about 1½ to 3 inches differential in 40 feet. These estimated seismic settlements may be excessive for support of the proposed residential buildings on lightly reinforced conventional shallow spread foundations. However, the project structural engineer has indicated a reinforced mat/slab foundation system can tolerate this magnitude of settlement for the proposed structures.

7.7 Based on the depth of groundwater liquefaction is not considered a concern for the proposed developments and improvements.

7.8 In order to limit the potential for excessive differential static settlement of the building foundations, over-excavation and compaction of the near surface soils is recommended to support new foundations on engineered fill. Static settlements of 1 inch total and ½ inch differential should be anticipated for foundations supported on subgrade soils prepared in accordance with the recommendations of this report.

7.9 Chemical testing of soil samples indicated the soils exhibit a “mildly corrosive” to “essentially non-corrosive” corrosion potential.

7.10 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318-14, the sulfate concentration from chemical testing of soil samples falls in the S0 classification (less than 0.10 percent by weight) for concrete.

7.11 The site is not located in an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture on the site is estimated to be low.

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, the following recommendations are presented 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, evaluation, and conclusions should be considered. The recommended design consultation and construction monitoring by Moore Twining are integral to the proper application of the recommendations. The Contractor is required to comply with the requirements and recommendations presented in this report.

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

8.1 General

8.1.1 Moore Twining should be provided the opportunity to review the final grading plans and foundation plans before the plans are released for bidding purposes so that any relevant recommendations can be presented.

8.1.2 This report was prepared based on assumed maximum column loads of about 40 kips and maximum perimeter wall loads of 3 kips per linear foot. Mr. Mark Van Gaale (VCA Structural) reported the maximum loading would be around 270 pounds per square foot for a typical three-story residential structure. This maximum loading includes dead plus live loads but does not include the load of any slab/foundations. When the actual foundation loads are known, this information should be provided to Moore Twining for review to confirm the recommendations for site preparation are suitable. In the event the foundation loads are different than assumed, the recommendations in this report may need to be revised.

8.1.3 A preconstruction meeting including, as a minimum, the owner, general contractor, earthwork contractor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project requirements and scheduling.

8.1.4 A demolition plan should be developed to identify the existing improvements shown on Drawing No. 2 in Appendix A (i.e., concrete pit in the northeastern portion of the site and concrete dry well in the southwestern portion of the site, concrete drainage structures along northern and western sides of the site, and concrete structure within the Southern California Edison electrical easement in the middle portion of the site) to be removed.

8.1.5 The Contractor(s) bidding on this project should determine if the information included in the construction documents are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect information as required to prepare accurate bids.

8.1.6 In the event that the predicted differential seismic settlement cannot be resisted by the foundation system, alternative methods of site preparation could be utilized to mitigate or reduce differential seismic settlements. Discussions during this investigation concluded that ground modification, or other means to mitigate or reduce the anticipated seismic settlement, are not feasible for the numerous smaller residential structures planned. So recommendations for ground modification were not included in the scope of this initial geotechnical investigation. If mitigation of seismic settlements needs to be evaluated for the larger structures, Moore Twining should be contacted to provide supplemental investigations of those locations to further evaluate subsurface conditions and provide recommendations for ground modification.

8.2 Site Grading and Drainage

8.2.1 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 should be sloped a minimum of two percent for a distance of at least ten 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 1 percent away from the foundations.

8.2.2 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. However as a minimum, perimeter landscaped areas should be sloped to rapidly drain surface water away from the buildings and limit irrigation to prevent water standing within 10 feet of foundations and saturating the soils supporting the foundations.

8.2.3 Trees should be setback from the proposed structures 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 planted at least 15 feet away (radius) from proposed or existing buildings.

8.2.4 Landscaping after construction should direct rainfall and irrigation runoff away from the structures and should establish positive drainage of water away from the structures. Care should be taken to maintain a leak-free sprinkler system.

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

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

8.2.7 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system. As an alternative, the roof drains should extend a minimum of 5 feet away from the structures and the resulting runoff directed away from the structures at a minimum of 2 percent.

8.3 Stormwater Infiltration Systems

The scope of this investigation only included installing and conducting percolation tests at general locations at depths of about 10 and 15 feet BSG to provide preliminary evaluation by the Civil Engineer for proposed infiltration systems. The results of these preliminary tests indicated unfactored infiltration rates of 0.9 inches per hour for percolation test P-3 in the southwestern portion of the site and 7.1 inches per hour for P-2 in the northeastern portion of the site (both tests conducted in poorly graded sand with silt soils). In addition, percolation tests conducted at a depth of about 15 feet BSG for the proposed infiltration systems indicated unfactored infiltration rates of 2.6 inches per hour for P-4 in the northwestern portion of the site and 15.9 inches per hour for P-1 in the southeastern portion of the site.

It should also be noted that the unfactored infiltration rates listed above should be considered preliminary data. When the locations of the underground infiltration systems are known, details regarding the location and depth of the underground infiltration systems will need to be provided to Moore Twining, and additional testing will need to be conducted. Based on other projects that we have conducted in the City of Redlands, it is expected that double ring infiltrometer infiltration testing will be required to determine the design infiltration rate for any underground stormwater. Percolation testing is allowed for preliminary purposes, but the values used for final design must come from a double ring infiltrometer infiltration test. Thus, additional recommendations for underground infiltration systems will be provided at a later time when details regarding the location and depth of the underground infiltration systems are provided and double ring infiltration testing is conducted.

Our experience with infiltration systems is that they have a limited life span. Thus, regular maintenance should be expected to maximize the useful life of these facilities and future expansion or modification of these systems should be anticipated to maintain functionality.

8.4 Site Preparation

One of the primary geotechnical engineering concerns identified in this report is the potential for dry seismic settlement that could occur from shaking from the maximum considered earthquake. The estimated seismic settlements should be reviewed by the building designer to determine the appropriate type of foundation system for the structures and whether special mitigation is required to address the estimated seismic settlements. As indicated in section 6.5 of this report, depending on the allowable settlement for the structures, site preparation could include: over-excavation and placement of engineered fill (see section 8.4.4 for site preparation recommendations) for support of a shallow spread or a rigid post-tensioned slab type foundation system which is engineered based upon the estimated seismic settlements (about 1½ to 3 inches of differential seismic settlement in 40 feet).

8.4.1 Stripping should be conducted in all areas of existing improvements to remove surface vegetation and root systems (if any). The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. The actual depth of stripping should be reviewed by our firm at the time of construction. Deeper stripping may be required in localized areas. Stripping and clearing of debris should extend laterally a minimum of 10 feet outside areas of proposed improvements (buildings, pavements and site flatwork work). 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.

8.4.2 Existing underground utilities within areas of proposed improvements (if any) should be removed and backfilled with engineered fill. A Southern California Edison 5-foot wide electrical easement trends in a north-south orientation through the middle portion of the site. It is unknown if an electrical utility exists within the easement. A concrete structure was noted within the 5-foot wide easement (see Drawing No. 2 in Appendix A of this report). The concrete structure within the electrical easement should be removed and not demolished in-place and mixed with on-site soils to be used as engineered fill. All utilities should be removed in their entirety and all loose backfill associated with these utilities should be over-excavated and backfilled as engineered fill. Utility materials to be removed should be completely removed and disposed of off-site and should not be crushed and buried in-place. Disturbed soils resulting from the removal of the utilities should also be over-excavated, moisture conditioned, and compacted as engineered fill. Prior to backfill of the excavations, the bottom of the excavations should be scarified to a depth of 8 inches, moisture conditioned and compacted as engineered fill.

8.4.3 During site preparation, the existing concrete pit in the northeastern portion of the site, the concrete dry well in the southwestern portion of the site, the concrete drainage structures along northern and western sides of the site and all other existing surface and subsurface structures will need to be removed (see general locations on Drawing No. 2 in Appendix A of this report). Refer to section 8.4.12 of this report regarding abandonment of the dry well in the southwestern portion of the site. Over-excavation should be conducted to remove all undocumented fills and all loose, disturbed soils associated with removal of surface and subsurface improvements and extend to at least 12 inches below the bottom of the surface and subsurface improvements that are removed.

8.4.4 After site stripping, removal of root systems, removal of existing surface and subsurface improvements, areas of proposed residential structures and all foundations should be over-excavated to at least 24 inches below preconstruction site grades, to the depth below 24 inches required to completely remove deeper rodent burrows, to a minimum of 12 inches below the bottom of reinforced mat/slab foundations designed to resist static settlements, to a minimum of 24 inches below the bottom conventional shallow spread of the footings (if used), and to at least 12 inches below the bottom of existing surface and subsurface improvements and associated fill soils to be removed, whichever is greater.

The over-excavation for the new structures should include the entire building footprints and all foundations, a minimum of 5 feet beyond the foundations and a minimum of 3 feet beyond all concrete slabs directly adjacent to the buildings such as walkways, etc., whichever is greater. The bottom of the excavation should be scarified 8 inches in depth, moisture conditioned to within (2) percent of the optimum moisture content and compacted as engineered fill.

8.4.5 The plans should show the limits of over-excavation for the building pads as described above in section 8.4.5.

8.4.6 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.

8.4.7 After site stripping, removal of root systems and removal of existing surface and subsurface improvements, areas of proposed carports should be over-excavated to at least 24 inches below preconstruction site grades, to the depth required to completely remove deeper rodent burrows, to at least 12 inches below the bottom of the footings (if shallow foundations are used), and to at least 12 inches below the bottom of existing improvements to be removed and associated fill soils to be removed, whichever is greater.

8.4.8 Where pool/spa excavations are made, a Moore Twining field representative should inspect and verify the resulting excavations are cleaned of all loose or organic material. After approval, the exposed native soils at the base of the excavation should be scarified to a depth of 8-inches, and moisture conditioned and compacted as engineered fill.

8.4.9 Following stripping and removal of surface and subsurface improvements, areas to receive fill outside the building pad over-excavation limits, pavements, and exterior slabs-on-grade should be prepared by over-excavation to a minimum of 12 inches below preconstruction site grade, to the depth required to remove rodent burrows, to the bottom of the proposed aggregate base section, and to at least 12 inches below the bottom of improvements to be removed, whichever is greater. The bottom of the over-excavation should be scarified to a minimum depth of 12 inches, moisture conditioned to within two (2) percent of the optimum moisture content and compacted as engineered fill. The upper 12 inches of subgrade beneath the pavement areas should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.4.10 Following stripping and removal of existing surface and subsurface improvements, areas to receive miscellaneous lightly (less than 1 kip per foot) loaded foundations such as site walls, trash enclosure walls and retaining walls, should be over-excavated to the bottom of foundations; to at least 12 inches below preconstruction site grades; and to at least 12 inches below subsurface improvements (structures, utilities, etc.) and any associated fill soils to be removed, whichever is greater. The over-excavation should extend to at least 3 feet beyond the edge of the foundations. If site walls are planned along property lines and over-excavation cannot extend beyond the property line, then the over-excavation should extend up to the property line. The bottom of the over-excavation should be scarified to a depth of at least 8 inches, moisture conditioned and compacted as engineered fill.

8.4.11 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.

8.4.12 The contractor should locate all on-site water wells or onsite septic system (if any) and the dry well in the southwestern portion of the site that is described in this report and shown on Drawing No. 2 in Appendix A. The debris and gravel (if any) in the dry well in the southwestern portion of the site should be removed (suggest drilling out with a bucket auger). All wells 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.

8.4.13 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.

8.4.14 Final grading shall produce building pads 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.

8.4.15 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) 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. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal.

8.5 Engineered Fill

8.5.1 The on-site near surface soils encountered are predominantly silty sands, poorly graded sands, and poorly graded sands with silt with varying small amounts of gravel. Recycled materials including asphalt, concrete and brick should not be mixed in with soils to be used as engineered fill below buildings; however, these materials may be processed to less than 6 inches in size and mixed in with soils to be used as engineered fill outside of building areas. The on-site soils are considered suitable for use as engineered fill below the recommended aggregate base section of. Interior and exterior slabs-on-grade and Portland cement concrete pavements are recommended in this report to be underlain by 4 inches of aggregate base. The aggregate base below the interior slabs-on-grade for the proposed building structures should consist of a non-recycled aggregate base. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.5.2 The compactability of the native soils is 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.

8.5.3 Import fill soil (if any) should be non-recycled, non-expansive and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 15
Organics	Less than 3 percent by weight
R-Value	Minimum 50*
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 10,000 ohms-cm

* for pavement areas only

Prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner and Moore Twining) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. In addition, Moore Twining should be requested to sample and test the material to determine compliance with the above geotechnical criteria. Contractors should provide a minimum of 7 working days to complete the testing.

8.5.4 Native and imported engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within two (2) percent of the 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. The upper 12 inches of fill and subgrade compacted in pavement areas should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

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

Table No. 5
Minimum Test Frequency

Area	Minimum Test Frequency
Building Pads	1 test per 5,000 square feet per compacted lift, but not less than two tests per building pad per lift
Pavement Subgrade and Mass Grading Outside Building Pads	1 test per 10,000 square feet per compacted lift
Utility Lines and Walkways	1 test per 150 feet per lift

8.5.6 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.

8.5.7 Aggregate base below the building slabs should comply with State of California Department of Transportation requirements for a non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. Alternatively, Crushed Miscellaneous Base (CMB), or a recycled Class 2 aggregate base, may be used for pavement areas outside the building and overbuild zones, provided that the recycled materials are accepted by the Owner and adequate quality control testing is conducted. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.6 General Recommendations for All Foundations

8.6.1 One of the primary geotechnical engineering concerns identified in this report is the potential for dry seismic settlement as a result of shaking from the maximum considered earthquake. The estimated seismic settlements presented below should be reviewed by the building design professionals to determine whether the proposed structures can be supported on a conventional spread foundation system - without the use of special mitigation measures.

8.6.2 The following settlements should be anticipated for design: 1) a total static settlement of 1 inch; 2) a differential static settlement of ½ inch in 40 feet; 3) an estimated differential seismic settlement of up to 3 inches over a distance of 40 feet.

If these magnitudes of settlements are tolerable for the planned structures, foundation design can follow the recommendations included in Section 8.7 of this report for design of shallow spread foundations. If these magnitudes of settlements are tolerable for the planned structures, recommendations are also included in Section 8.8 of this report for use in design of stiffened mat slab (post-tensioned type) foundations.

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

8.6.4 Foundation excavations or exposed soils should not be left uncovered and allowed to dry such that the moisture content of the soils is less than one percent above optimum moisture content or drying produces cracks in the soils. The exposed soils, such as sidewalls, excavation bottoms, etc. should be periodically moistened to maintain the moisture content within two (2) percent of the optimum moisture content until concrete is placed. It should be noted that the contractor should take precautions not to allow the exposed soils to dry, including weekends and holidays.

8.6.5 The following seismic factors were developed 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 Site Class D, a latitude of 34.071899 degrees and a longitude of -117.195862 degrees. The data provided in Table No. 6 are based upon the procedures of Sections 1613.2.1 through 1613.2.4 of the 2019 California Building Code and were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 6 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

TABLE NO. 6

Seismic Factor	2019 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.844g
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration (PGA)	0.767g
Spectral Response At Short Period (0.2 Second), Ss	1.828
Spectral Response At 1-Second Period, S_1	0.731
Site Coefficient (based on Spectral Response At Short Period), Fa	1.0
Site Coefficient (based on spectral response at 1-second period) Fv	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.828

TABLE NO. 6

Seismic Factor	2019 CBC Value
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note
Five percent damped design spectral response accelerations for short period, S_{DS}	1.219
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	See Note

Note: Requires ground motion hazard analysis per ASCE Section 21.2 (ASCE 7-16, Section 11.4.8), unless an Exception of Section 11.4.8 of ASCE 7-16 is applicable for the project design.

8.6.6 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete 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.

8.6.7 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 200 pounds per square foot may be used to resist axial loads. Lateral load resistance may be estimated using the 2019 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 300 pounds per square foot per foot of depth to a maximum of 3,000 pounds per square foot. The passive pressure may be assumed to act over twice the pier diameter. The passive resistance of the surface soils to a depth of 12 inches, or to the depth where the horizontal setback from the foundation to a descending slope is less than 3 feet, whichever is greater, should be neglected.

8.6.8 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.40 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

8.6.9 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 350 pounds per cubic foot. The upper 6 inches of subgrade in landscaped areas should be neglected in determining the total passive resistance.

8.7 Conventional Shallow Spread Foundations and Concrete Slabs on Grade

The following recommendations may be used for support of the structures, provided the design professional can conclude that the planned structures can tolerate the estimated static and seismic settlements recommended below.

8.7.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations and slabs on grade based on the estimated settlements. The following should be anticipated for design: 1) a total static settlement of 1 inch; and 2) a differential static settlement of ½ inch in 40 feet. In addition, shallow spread foundations would need to be designed for an additional differential seismic settlement of up to 3 inches over a distance of 40 feet.

8.7.2 Foundations 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 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.

8.7.3 All perimeter footings for the new buildings should have a minimum depth of 18 inches below the lowest adjacent grade. All interior foundations should have a minimum depth of 18 inches below the bottom of the floor slab or deeper to meet CBC minimum for 3 or 4-story structures. All footings for the new buildings should have a minimum width of 15 inches, regardless of load.

8.7.4 Structural loads for lightly (less than 1.5 kips per lineal foot) loaded miscellaneous foundations (such as screen walls for the proposed trash enclosures) should be supported on subgrade soils prepared in accordance with the "Site Preparation" section of this report. The screen walls for the trash enclosure may be supported by footings extending to a minimum depth of 12 inches below the lowest adjacent finished grade and a minimum width of 12 inches. These improvements may be designed for a maximum allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads for footings. This value may be increased by one-third for short duration wind or seismic loads. It should be noted the miscellaneous foundations (such as screen walls for the proposed trash enclosures) would be subject to the seismic settlements noted in this report.

8.7.5 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.38 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

8.7.6 For spread foundations, the allowable passive resistance of the engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. The upper 6 inches of subgrade in landscaped areas should be neglected in determining the total passive resistance.

8.8 Post-Tensioned Slab/Foundations

If the design professional conclude that the planned structures supported on conventional minium reinforced shallow spread foundations considering the combined estimated static and seismic settlements recommended herein, the following recommendations are and option to provide a more rigid mat/slab foundation (post-tensioned) system that can adequately resist the anticipated settlements of the subgrade soils.

8.8.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the rigid slab foundations based on a total static settlement of 1 inch, a differential static settlement of ½ inch in 40 feet, a total seismic settlement of up to about 6 inches, and a differential seismic settlement of up to about 3 inches over 40 feet.

8.8.2 Rigid slab foundations should be underlain by at least 4 inches of compacted, non-recycled Class 2 aggregate base over engineered fill extending to the depths recommended in the “Site Preparation” recommendations section of this report.

8.8.3 Rigid slab foundations consisting of a structurally engineered, nearly uniform thickness reinforced concrete slab-on-grade, may be designed for a maximum net allowable soil bearing pressure of 1,000 pounds per square foot for dead-plus-live loads. The dead load of the mat foundation may be neglected in design. These values may be increased by one-third for short duration wind or seismic loads.

8.8.4 A modulus of subgrade reaction of 7 psi/inch may be used in design of the post-tensioned slab.

8.8.5 The rigid slab foundation should incorporate perimeter thickened edges that extend at least 12 inches below the lowest adjacent finished grade.

8.8.6 Foundation excavations should be observed by Moore Twining prior to the placement of 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 reinforcement.

8.8.7 The moisture conditions of the subgrade soils for the building pad and foundation excavations should be maintained in accordance with the recommendations for engineered fill until placement of concrete for foundations or until aggregate base is placed for the building pad areas. If the subgrade is allowed to dry below the optimum moisture content, the subgrade soils below the slab should be wetted to achieve a moisture content within two (2) percent of the optimum moisture content prior to placement of the concrete slab.

8.9 Carport Foundations Supported on Cast-in-Drilled-Hole (CIDH) Pile Foundations

8.9.1 A structural engineer registered in the state of California should prepare structural details for the fuel canopy foundations to resist shear, moment, and axial (tension and compression) loads.

.

8.9.2 Skin friction in the upper portion of the piles, to a depth of 12 inches should be neglected for design. The allowable vertical downward load capacity of the CIDH pile foundations below a depth of 12 inches below site grade may be designed based on an allowable skin friction value of 200 pounds per square foot. The above stated values assume that the cast-in-drilled-hole foundations are placed into the existing undisturbed native soils. These values may be increased one-third (⅓) for short duration loading.

8.9.3 The allowable uplift resistance of the pile foundations may be assumed to be half of the skin friction value used for design.

8.9.4 Piles should be placed no closer to each other than three pile diameters, center-to-center. For alternate spacing, the capacity of piles in groups should be reduced using appropriate group reduction formulas.

8.9.5 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on a total static settlement of 1 inch and a differential static settlement of ½ inch between foundations. It should also be noted that the cast-in-drilled-hole pile foundations would be subject to the seismic settlements discussed in this report.

8.9.6 Passive resistance in the upper portion of the piles, to a depth of 1 foot should be neglected for design. The allowable passive resistance of the soils below a depth of 1 foot below site grade may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot to a maximum of 3,000 pounds per square foot. These values may be increased by one-third (1/3) for short duration wind or seismic loads. The passive pressure for drilled pile foundations spaced at three (3) pile diameters may be applied over a width equal to 2 pile diameters.

8.9.7 Piles should be placed no closer than three pile diameters, center-to-center. For alternate spacing, the capacity of piles in groups should be reduced using appropriate group reduction formulas.

8.10 Cast-In-Drilled-Hole Pile Construction

8.10.1 It is assumed the project structural engineer will prepare a specification for the construction of the deep foundations as part of the construction documents. The specifications should be consistent with the recommendations included in this report.

8.10.2 Concrete should be placed in the drilled shaft as soon as possible following drilling.

8.10.3 The on-site soils are granular in nature, some layers have low fines content and are anticipated to have limited standup capacity. If required, temporary casing should be used for temporary support of the excavations during construction. The casing should be slowly removed from the shaft excavation during placement of concrete to ensure the casing is not raised above the level of the concrete during shaft construction, to prevent sidewall soils from sloughing into the shaft excavation. As an alternative, it may be possible to utilize a drilling slurry for temporary support of the foundation excavations if unstable sidewalls occur. The Contractor will be required to provide temporary excavation support of the drilled pile excavations as necessary to construct the foundations.

8.10.4 Casing (if used) should be able to withstand the external pressures of the caving soils. The outside diameter of the casing should not be less than the diameter of the cast-in-drilled hole concrete pile.

8.10.5 Drilled holes for pile foundations should be drilled within 2 degrees of vertical. The rebar cage should be suspended within 2 degrees of vertical in the center of the excavation. This condition should be verified and documented during construction. Minimum concrete cover, as specified by the project design engineer, should be maintained throughout the length of the excavation.

8.10.6 Groundwater is not anticipated to be encountered during pile construction. However, in the event freewater seepage is encountered during excavation, the concrete should be placed from the bottom of the excavation by extending the tremie pipe or pump pipe to the bottom of the excavation and maintaining the outlet of the pipe below the wet concrete to prevent entrapment of freewater or slurry in the concrete. The concrete should be placed in a continuous manner to provide a seamless deep foundation element.

8.10.7 Casing should be lifted slowly as the concrete is deposited, while the bottom of the casing is kept at least two feet below the top of the concrete.

8.10.8 Moore Twining should inspect the drilling of the shafts to verify that the materials encountered are consistent with those evaluated during our geotechnical engineering investigation. This inspection should be conducted during drilling and prior to placement of reinforcing steel and concrete.

8.10.9 Loose soils should be removed from the drilled shaft excavation prior to placement of reinforcing steel and concrete.

8.11 Frictional Coefficient and Earth Pressures

8.11.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.38 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.

8.11.2 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 300 pounds per cubic foot. The upper 6 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.11.3 The active and at-rest pressures of the on-site or imported, non-expansive engineered fill may be assumed to be equal to the pressures developed by fluid with a density of 45 and 67 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 and roadways and hydrostatic water pressure.

8.11.4 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.

8.11.5 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a backdrain as recommended in this report.

8.11.6 The wall designer should determine if seismic increments are required. If seismic increments are required, Moore Twining should be contacted for recommendations for seismic geotechnical design considerations for the retaining structures.

8.12 Retaining Walls / Screen Walls

8.12.1 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.

8.12.2 Retaining wall / screen wall footings should be supported on engineered fill soils prepared as recommended in the Site Preparation section of this report. In the event retaining walls are planned, retaining walls should be supported on engineered fill soils as recommended for miscellaneous, lightly loaded foundations prepared as recommended in the Site Preparation section of this report. Spread and continuous footings for retaining walls with a minimum depth of 12 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. These values may be increased by one-third for short duration wind or seismic loads. It should be noted the retaining wall and screen wall foundations would be subject to the seismic settlements noted in this report.

8.12.3 Retaining walls should be constructed with imported or on-site granular backfill. The import fill material (if used) should be tested and approved as recommended under the subsection entitled "Engineered Fill" in the recommendations section of this report.

8.12.4 Granular wall backfill using the on-site soils or imported, non-expansive granular soils meeting the recommendations included in Section 8.5.3 of this report should be compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557 and should extend from the outer edge of the footing to the ground surface at a 1 Horizontal to 1 Vertical (1H:1V) inclination.

8.12.5 Segmented wall design (mechanically stabilized earth walls) 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 mechanically stabilized earth 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.

8.12.6 The earth pressures provided in this report (Section 8.11) assume that the retained materials behind the wall will be drained. A drain system should be provided. The drain system should be a minimum of 12 inches wide, and should consist of an open-graded rock (3/4 inch) encapsulated in a geotextile filter fabric such as Mirafi 140N. The gravel drain system should incorporate drain pipes at the base of the wall which are embedded in the open graded rock to carry seepage from behind the wall. Drainage should be directed to pipes which gravity drain to an approved outlet. 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 does not function properly. It is also recommended that inspection pipes and clean-outs be incorporated into the design.

8.12.7 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.

8.12.8 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 the exterior of the walls. Waterproofing systems should be designed and specified by a qualified professional.

8.12.9 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.

8.13 Interior Slabs-on-Grade

The slabs on the project that should be prepared as interior slabs include: the interior floor slab and all concrete slabs on grade directly adjacent to the buildings.

8.13.1 Interior slabs-on-grade should be constructed over 4 inches of non-recycled aggregate base over engineered fill placed for the building pad preparation in accordance with the Site Preparation section of this report.

8.13.2 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.

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

8.13.4 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 4 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 2,400 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 D1745 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.

8.13.5 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.

8.13.6 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.

8.13.7 The moisture retarding membrane is not required beneath exposed concrete floors, such as garages, provided that moisture intrusion into the structures are permissible for the design life of the structures.

8.13.8 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.

8.13.9 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.

8.13.10 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.

8.13.11 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.

8.14 Exterior Slabs-On-Grade and Concrete Pool / Spa Decking

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

8.14.1 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.

8.14.2 Subgrade soils for exterior slabs should be prepared as recommended in the “Site Preparation” section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the exterior slabs should be supported on 4 inches of aggregate base over the prepared subgrade soils. The aggregate base section may be omitted below exterior slabs provided an increased risk of subgrade instability and cracking of the concrete slabs is acceptable to the Owner.

8.14.3 The moisture content of the subgrade soils should be verified to be near optimum moisture content within 48 hours of placement of the slab-on-grade. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.14.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the slabs-on-grade.

8.14.5 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.

8.15 In-Ground Swimming Pool / Spa

8.15.1 The vertical walls of the pool/spa shells should be designed based on a minimum equivalent fluid pressure of 67 pounds per cubic foot. This value does not include any surcharge effects of construction equipment, foundations, slopes, or hydrostatic pressures, etc. The pool engineer should include the appropriate surcharges and design loads in addition to the above earth pressure. The pool / spa shells (bottom and walls) shall be designed for a potential differential settlement of ½ inch.

8.15.2 The bottom of the pool/spa excavations should be observed and approved by a Moore Twining representative prior to placement of reinforcing steel or forms. As recommended in the Site Preparation section of this report, after approval of the excavation by Moore Twining, the resulting excavations should be cleaned of all loose or organic material, the exposed native soils at the base of the excavation should be scarified to a depth of 8-inches, and moisture conditioned and compacted as engineered fill.

8.15.3 If the subgrade is prepared, and then disturbed by equipment workers, weather or other source, it is recommend that the exposed subgrade to receive slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to meet the recommendations of this report. This condition should be verified 48 hours prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

8.15.4 Due to the granular nature of the onsite soils, excavations for the pool/spa excavations should not be anticipated to stand unsupported vertical or near vertical. Caving or sloughing of steeply cut, unsupported, excavations should be anticipated. Thus, provisions for pool construction should address these conditions. Where caving occurs, all loose/disturbed soils should be removed to expose undisturbed native soils and the excavations should be backfilled with engineered fill.

8.15.5 The pool shell excavation should not encroach a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of any adjacent proposed (or existing) foundations.

8.16 Asphaltic Concrete (AC) Pavements

Recommendations are provided below for new asphaltic concrete pavements planned as part of the new construction.

8.16.1 The subgrade soils for asphaltic concrete pavements should be over-excavated and compacted as recommended in the “Site Preparation” section of the recommendations in this report.

8.16.2 The following pavement sections are based on an R-value of 50 and traffic index values ranging from 5.0 to 7.0, a minimum asphalt concrete thickness of 3 inches and a minimum aggregate base thickness of 4 inches. 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. 7
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0 to 6.0	3.0	4.0	12
6.5	3.5	4.5	12
7.0	4.0	4.5	12

AC - Asphaltic Concrete compacted as recommended in this report
AB - Class II Aggregate Base, Crushed Aggregate Base (CAB), or Crushed Miscellaneous Base (CMB) 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)

8.16.3 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.

8.16.4 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.

8.16.5 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.

8.16.6 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.

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

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

8.16.9 The asphaltic 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.

8.16.10 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.

8.17 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.

8.17.1 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.

8.17.2 The following pavement section designs are based on a design modulus of subgrade reaction, K-value of 230 psi/in over the native compacted soil. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) a design life of 20 years, 3) load transfer by aggregate interlock or dowels, 4) concrete shoulder, 5) a load safety factor of 1.1, and 6) truck loading consisting of 1 single axle load of 20 kips and two tandem axle loads of 35 kips each.

Table No. 8
Two-Layer Portland Cement Concrete Pavements

ADTT	PCC Layer Thickness (inches)	Aggregate Base Layer (inches)	Compacted Subgrade (inches)
0.29 trucks per day (2 trucks per week)	6.0	4.0	12.0
1 truck per day (7 trucks per week)	6.5	4.0	12.0

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)

8.17.3 The PCC pavement should be constructed in accordance with 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.

8.17.4 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 slabs.

8.17.5 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.

8.17.6 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 8.17.5, 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. The design engineer should decide if aggregate interlock is appropriate or specify joint reinforcement.

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

8.17.8 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.

8.17.9 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.

8.17.10 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.

8.17.11 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.

8.17.12 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.

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

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

8.18 Slopes and Temporary Excavations

8.18.1 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.

8.18.2 Temporary excavations should be constructed in accordance with 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.

8.18.3 In no case should excavations extend below a 2H to 1V zone below 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.

8.18.4 Shoring should be designed by an engineer with experience in designing shoring systems and registered in the State of California.

8.18.5 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 structures occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.19 <u>Utility Trenches</u>

8.19.1 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.

8.19.2 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 to between optimum and three (3) percent above optimum moisture content and compacted to a minimum of 92 percent relative compaction. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.19.3 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 No. 9 (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. 9
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 D2321-00
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80

8.19.4 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.

8.19.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned to within two (2) percent of the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. 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.

8.19.6 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

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

8.19.8 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.

8.19.9 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.

8.19.10 The plans should note that all utility trenches, including electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 92 percent per ASTM D-1557, except for the upper 12 inches below pavements, which should be compacted to at least 95 percent relative compaction.

8.19.11 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.

8.20 Corrosion Protection

8.20.1 The analytical results of sample analyses indicate the samples had resistivity values of 19,000; 25,000 and 28,000 ohms-centimeter, with pH values of 7.3, 7.2, and 7.8, respectively. Based on the resistivity values and the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 3 of section 6.8 of this report, the soils exhibit a "mildly corrosive" to "essentially non-corrosive" corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "mildly 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.

8.20.2 Based on Table 19.3.1.1 - Exposure categories and classes from Chapter 19 of ACI 318-14, the sulfate concentration from chemical testing of soil samples falls in the S0 classification (less than 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or 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 "Interior Slab on Grade" section of this report.

8.20.3 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.

9.0 DESIGN CONSULTATION

9.1 Moore Twining should be retained to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

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

9.3 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.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.

10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 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 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.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least $3,000,000 and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

10.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 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 between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

11.2 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.

11.3 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 (over 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.

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

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in 3.4, Anticipated Construction and Grading. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures 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.

11.6 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.

11.7 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.8 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.

11.9 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.

We appreciate the opportunity to be of service to Redlands Summit, LLC. 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

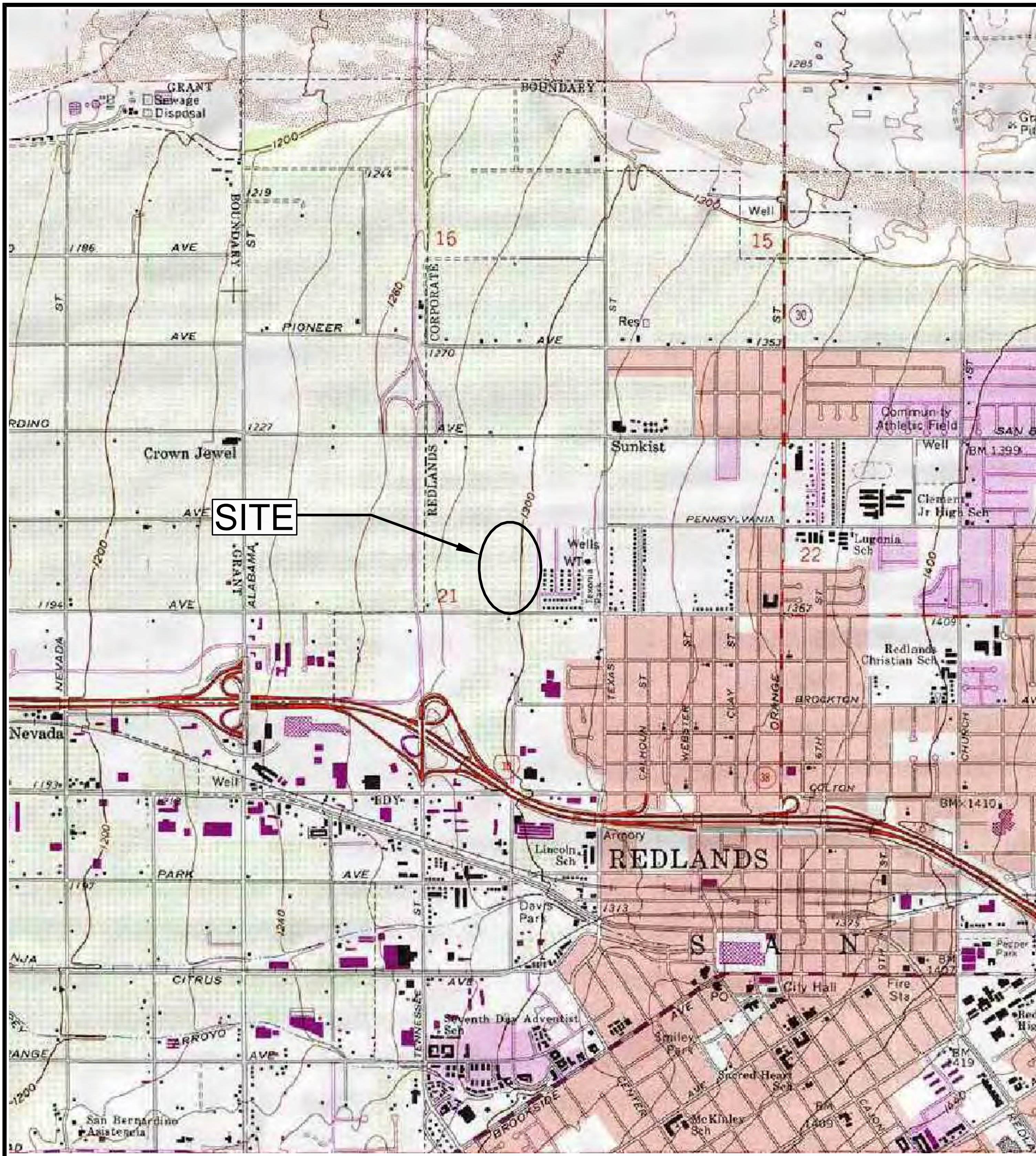
Allen H. Harker, PG
Professional Geologist

DRAFT

Scott W. Krauter, RGE
Assistant Manager

APPENDIX A

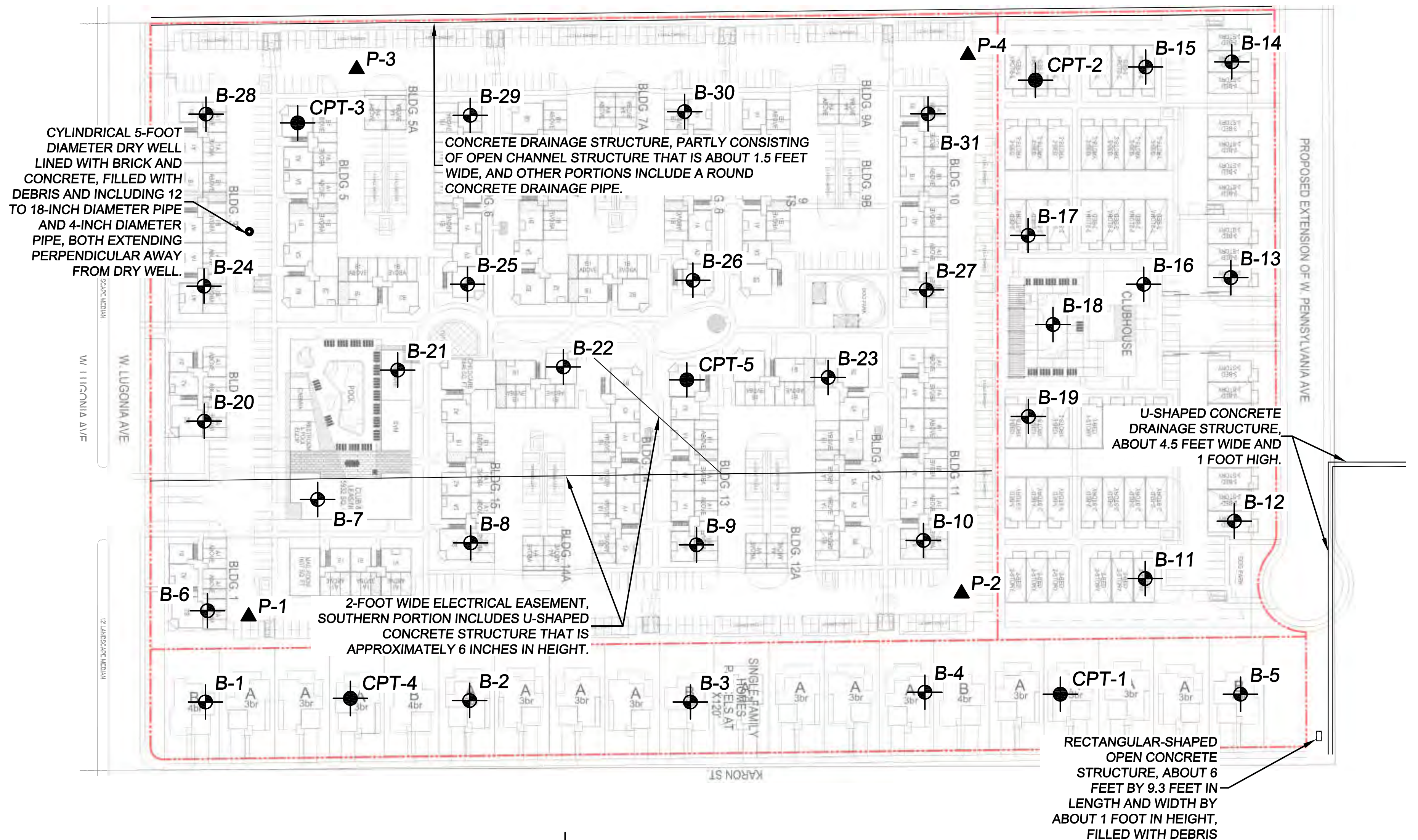
DRAWINGS



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
REDLANDS, CALIFORNIA QUADRANGLE 1996

0 2000
APPROXIMATE SCALE IN FEET

SITE LOCATION MAP THE NEIGHBORHOODS AT LUGONIA VILLAGE NORTHWEST CORNER OF WEST LUGONIA AVENUE AND KARON STREET REDLANDS, CALIFORNIA	FILE NO.: 02901-01-01	DATE: 01/06/2022	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H02901.01	DRAWING NO. 1	



APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

APPROXIMATE CONE PENETRATION TEST (CPT) SOUNDING LOCATION

N

0 160

APPROXIMATE SCALE IN FEET

TEST BORING, CPT AND PERCOLATION TEST BORING LOCATION MAP THE NEIGHBORHOODS AT LUGONIA VILLAGE NORTHWEST CORNER OF WEST LUGONIA AVENUE AND KARON STREET REDLANDS, CALIFORNIA	FILE NO. 02901-01-02	DATE DRAWN: 01/06/2022
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. H02901.01	DRAWING NO. 2

MOORE TWINING ASSOCIATES, INC.
Since 1898

APPENDIX B

LOGS OF BORINGS AND CONE PENETRATION TESTS

This appendix contains the final logs of borings and cone penetration tests (CPT). These logs represent 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 these test boring and CPT locations. Also, the passage of time may result in changes in the soil conditions at these test boring and CPT locations.

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: B-1

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	6/6 6/6 6/6	SM	SILTY SAND; medium dense, dry, fine to medium grained, light brown		12	0.9
	12/6 6/6 8/6		Loose, damp	DD = 111.7 pcf	14	1.2
5						
	11/6 13/6 22/6	SP	POORLY GRADED SAND; medium dense, damp, light brown		35	
10						
	10/6 10/6 5/6		Moist, increase in fines content, dark brown		15	
15			Bottom of Boring B-1 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	1/6 3/6 3/6	SM	SILTY SAND; loose, dry, fine to medium grained, light brown		6	0.6
	5/6 6/6 8/6		Decrease in fines content, increase in sand content	DD = 109.7 pcf	14	1.7
5						
	4/6 7/6 9/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, light brown		16	
10						
	4/6 7/6 9/6		Increase in moisture content		16	
15			Bottom of Boring B-2 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-3

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	4/6 3/6 4/6	SM	SILTY SAND; loose, dry, fine to medium grained, light brown	From 0-3.5': EI = 0	7	0.6
	4/6 5/6 9/6		Damp	Ring sample disturbed	14	1.2
5	3/6 6/6 9/6		Medium dense		15	
10	2/6 4/6 4/6	SP-SM	POORLY GRADED SAND WITH SILT; loose, moist, fine to medium grained, dark brown		8	
	6/6 8/6 13/6		Medium dense, decrease in fines content		21	
15			Bottom of Boring B-3 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-4

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Logged By: Y.A.

Drilled By: J.C.

Date: November 18, 2021

Drill Type: CME 75

Elevation:

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 Pound Auto Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 3/6 3/6	SM	SILTY SAND; loose, damp, fine to medium grained, light brown		5	
5	6/6 5/6 5/6	SP	POORLY GRADED SAND; loose, damp, fine to medium grained, light brown		10	
10	3/6 5/6 5/6		Increase in fines content		10	
15	6/6 3/6 6/6	SM	SILTY SAND; loose, moist, fine to medium grained, dark brown		9	
			Bottom of Boring B-4 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	1/6 2/6 2/6	SM	SILTY SAND; very loose, damp, fine to medium grained, brown, with trace gravel	From 0-3.5': pH = 7.3 SR = 19,000 ohm-cm Cl < 0.00060% SS < 0.00060%	4	
	2/6 3/6 4/6		Loose		7	3.3
5	11/6 16/6 26/6		Medium dense, increase in fines content	DD = 107.3 pcf ø = 39° c = 160 psf	42	2.9
10	6/6 10/6 8/6	SP	POORLY GRADED SAND; medium dense, moist, fine grained, light brown, with trace gravel		18	
	7/6 16/6 17/6		Dense, with some cobbles		33	
15			Bottom of Boring B-5 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-6

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	2/6 2/6 2/6	SM	SILTY SAND; very loose, dry, fine to medium grained, light brown		4	0.5
5	3/6 3/6 10/6		Medium dense, damp		13	
10	5/6 9/6 10/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, light brown, with trace gravel		19	
15	3/6 3/6 6/6	SM	SILTY SAND; loose, damp, fine grained, brown		9	
20	8/6 8/6 14/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown		22	
			Bottom of Boring B-6 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	1/6 2/6 2/6	SM	SILTY SAND; very loose, dry, fine to coarse grained, brown	From 0-3.5': EI = 0	4	0.6
5	22/6 20/6 21/6		Medium dense, damp, decrease in coarse grained sand, increase in fine grained sand and fines content, trace clay	DD = 114.3 pcf	41	2.8
10	3/6 5/6 7/6		Medium dense, light brown, with a little fine to coarse gravel	Gravel = 8.1% Sand = 74.9% -200 = 17.0% LL = Non-viscous PI = Non-plastic	12	
15	2/6 2/6 4/6		Loose, increase in fines, decrease in sand content		6	
20	7/6 8/6 9/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown, with trace gravel		17	
			Bottom of Boring B-7 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-8

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	2/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine to medium grained, brown, with trace gravel		5	0.7
	5/6 5/6 9/6		Damp, fine to coarse grained	DD = 103.8 pcf	14	1.4
5	4/6 1/6 2/6	SP	POORLY GRADED SAND; very loose, moist, fine to medium grained, brown, with trace gravel		3	
10	5/6 6/6 7/6		Medium dense, increase in gravel content		13	
	3/6 5/6 3/6		Loose, increase in fines content, decrease in gravel content		8	
15			Bottom of Boring B-8 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-9

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	4/6 6/6 3/6	SM	SILTY SAND; loose, damp, fine to medium grained, light brown		9	
	3/6 5/6 6/6		Medium dense		11	
5						
	5/6 8/6 10/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, brown		18	
10						
	5/6 7/6 10/6		With some gravel		17	
15						
	4/6 4/6 7/6	SM	SILTY SAND; medium dense, moist, fine grained, brown		11	
20			Bottom of Boring B-9 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-10

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 18, 2021

Elevation:

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	2/6 3/6 1/6	SM	SILTY SAND; very loose, dry, fine to medium grained, light brown		4	0.8
	2/6 5/6 8/6		Loose, damp	DD = 105.5 pcf	13	1.1
5	2/6 4/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown		11	
10	4/6 5/6 6/6		Increase in fines content		11	
	8/6 4/6 4/6	SP	POORLY GRADED SAND; loose, moist, fine to medium grained, brown		8	
15			Bottom of Boring B-10 at 15 feet			
20						
25						

Notes:

Figure Number

Test Boring: B-11

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	2/6 2/6 1/6	SM	SILTY SAND; very loose, dry, fine to medium grained, light brown		3	0.6
5	5/6 5/6 5/6		Loose, damp, fine grained, brown, trace fine gravel	Gravel = 0.5% Sand = 69.4% -200 = 30.1% LL = Non-viscous PI = Non-plastic	10	
10	7/6 8/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown		15	
15	9/6 10/6 11/6		Moist		21	
20	3/6 5/6 8/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		13	
25			Bottom of Boring B-11 at 20 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-12

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	1/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine to medium grained, light brown		5	0.7
	5/6 3/6 7/6		Damp, with trace gravel		10	
5	5/6 6/6 9/6		Moist, increase in fines content	DD = 111.5 pcf	15	4.3
10	3/6 5/6 6/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, brown		11	
15	6/6 10/6 10/6		Decrease in fines content		20	
20	5/6 6/6 8/6		Increase in fines content		14	
	5/6 5/6 5/6	SM	SILTY SAND; loose, moist, fine grained, brown		10	
25			Bottom of Boring B-12 at 25 feet			

Notes:

Figure Number

Test Boring: B-13

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	1/6 2/6 3/6	SM	SILTY SAND; loose, damp, fine grained, brown		5	
5	5/6 8/6 12/6		Medium dense, increase in moisture content and gravel content	DD = 110.7 pcf	20	1.8
10	6/6 7/6 8/6	SP-SM	POORLY GRADED SAND; medium dense, damp, fine to coarse grained, light brown, with trace gravel		15	
15	8/6 10/6 13/6		Medium dense, increase in gravel content		23	
20	10/6 8/6 9/6		Decrase in grain size		17	
			Bottom of Boring B-13 at 20 feet			
25						

Notes:

Figure Number

Test Boring: B-14

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	1/6 3/6 3/6	SM	SILTY SAND; loose, damp, fine grained, brown		6	
	1/6 3/6 3/6		Very loose, increase in fines content	DD = 107.4 pcf ø = 39° c = 0 psf	6	1.5
5	3/6 3/6 5/6		Loose, moist		8	
10	3/6 5/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, brown		12	
15	9/6 11/6 13/6		Increase in fines content		24	
20	7/6 11/6 13/6		With trace gravel		24	
25	8/6 10/6 12/6		Decrease in fines content		22	
			Bottom of Boring B-14 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-15

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Logged By: Y.A.

Drilled By: J.C.

Date: November 17, 2021

Drill Type: CME 75

Elevation:

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 Pound Auto Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 1/6 1/6	SM	SILTY SAND; very loose, damp, fine grained, brown		2	1.0
	2/6 2/6 2/6	SP-SM	POORLY GRADED SAND WITH SILT; very loose, damp, fine to medium grained, brown, with trace fine gravel		4	
5	6/6 2/6 3/6			DD = 101.9 pcf	5	2.4
10	4/6 8/6 11/6		Medium dense		19	
15	6/6 10/6 14/6		Increase in grain size, fine to coarse grained		24	
20	6/6 7/6 8/6	SP	POORLY GRADED SAND; medium dense, damp, fine grained, brown		15	
25	7/6 11/6 11/6		Moist		22	
			Bottom of Boring B-15 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-16

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

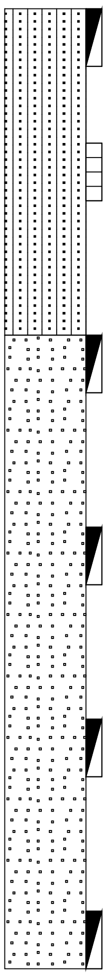
Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 16, 2021

Elevation:

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	2/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine grained, gray	From 0-3.5': pH = 7.2 SR = 25,000 ohm-cm Cl = 0.00074% SS < 0.00060% DD = 107.8 pcf	5	0.6
	5/6 9/6 7/6		Damp, increase in fines content		16	2.4
5						
	3/6 7/6 6/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, light brown, with trace gravel		13	
10						
	7/6 11/6 11/6		Increase in fine sand		22	
15						
	5/6 3/6 7/6		Loose, increase in fines content		10	
20						
	3/6 7/6 9/6		Decrease in fines content		16	
25			Bottom of Boring B-16 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-17

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Logged By: Y.A.

Drilled By: J.C.

Date: November 16, 2021

Drill Type: CME 75

Elevation:

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

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	2/6 4/6 6/6	SM	SILTY SAND; loose, dry, fine to medium grained, brown, with rootlets		10	0.8
5	10/6 17/6 30/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, fine to medium grained, brown, with trace gravel	DD = 111.8 pcf	47	2.2
10	5/6 5/6 10/6	SM	SILTY SAND; medium dense, damp, fine grained, brown, trace fine gravel	Gravel = 4.2% Sand = 78.5% -200 = 17.3%	15	
15	7/6 9/6 10/6	SP	POORLY GRAINED SAND; medium dense, moist, fine grained, brown		19	
20	8/6 8/6 8/6		With gravel		16	
25	9/6 6/6 6/6	SM	SILTY SAND; medium dense, moist, fine grained, brown		12	
			Bottom of Boring B-17 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-18

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 16, 2021

Elevation:

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	1/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine grained, gray, with trace gravel		5	0.5
	4/6 5/6 7/6		Medium dense, increase in sand content, decrease in fines content		12	
5						
	9/6 14/6 23/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, moist, fine to medium grained, brown	Rings disturbed	37	2.5
10						
	6/6 7/6 10/6		Medium dense, with gravel		17	
15			Bottom of Boring B-18 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-19

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	2/6 3/6 3/6	SM	SILTY SAND; loose, dry, fine to medium grained, brown		6	0.5
	4/6 5/6 6/6		Medium dense, moist		11	
5	7/6 8/6 10/6	SP	POORLY GRADED SAND WITH GRAVEL; medium dense, damp, fine to coarse grained, brown	Rings disturbed	18	1.0
	5/6 6/6 7/6		Increase in fines		13	
10						
	3/6 6/6 7/6		Decreaes in fines content, increase in sand content		13	
15						
	5/6 6/6 6/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, brown		12	
20						
	10/6 10/6 11/6		Moist		21	
25			Bottom of Boring B-19 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-20

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	5/6 3/6 3/6	SM	SILTY SAND; loose, damp, fine to medium grained, brown, with trace gravel		6	
	5/6 7/6 13/6		Medium dense, slight increase in moisture content, reddish brown	DD = 102.6 pcf	20	3.1
5						
	5/6 6/6 9/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, light brown		15	
10						
	3/6 5/6 8/6		Increase in silt content		13	
15			Bottom of Boring B-20 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-21

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	2/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine to medium grained, brown		5	
	5/6 6/6 10/6		Decrease in silt content, slight increase in sand content	DD = 107.7 pcf	16	0.8
5	3/6 5/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, brown, with trace gravel		12	
	4/6 5/6 9/6		Decrease in fines content, moist		14	
10						
	7/6 10/6 11/6		Increase in grain size		21	
15			Bottom of Boring B-21 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-22

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	2/6 2/6 2/6	SP-SM	POORLY GRADED SAND WITH SILT; very loose, damp, fine to medium grained, gray, with some fine to coarse gravel	Gravel = 10.0% Sand = 78.5% -200 = 11.5% LL = Non-viscous PI = Non-plastic	4	0.6
5	12/6 16/6 27/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace fine to coarse gravel	DD = 111.5 pcf Gravel = 2.4% Sand = 79.6% -200 = 18.0% LL = Non-viscous PI = Non-plastic ø = 31° c = 260 psf	43	6.9
10	2/6 3/6 5/6		Loose, sharp increase in fines content and decrease in sand content	Gravel = 1.1% Sand = 61.0% -200 = 37.9%	8	
15	10/6 3/6 6/6			Gravel = 0.2% Sand = 62.4% -200 = 37.4%	9	
20	3/6 5/6 6/6		Medium dense, increase in sand content		11	
25	2/6 4/6 5/6		Moist, increase in fines content and decrease in sand content	Sand = 54.7% -200 = 45.3%	9	
	4/6 6/6	SP-SM	POORLY GRADED SAND WITH	Gravel = 0.2% Sand = 88.6%	16	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-22

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

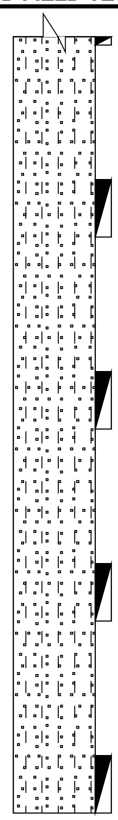
Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	10/6		SILT; medium dense, damp, fine to medium grained, brown, trace fine gravel	-200 = 11.2%		
35	4/6 6/6 11/6		Slight decrease in fines content, dark brown	Gravel = 0.9% Sand = 93.3% -200 = 5.8%	17	
40	4/6 6/6 10/6		With gravel		16	
45	10/6 10/6 10/6		Slight increase in fines content and decrease in sand content	Gravel = 0.5% Sand = 87.7% -200 = 11.8%	20	
50	7/6 9/6 11/6		Increase in grain size		20	
			Bottom of Boring B-22 at 50 feet			
55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-23

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	4/6 2/6 3/6	SM	SILTY SAND; loose, dry, fine grained, gray, with trace gravel		5	0.6
	2/6 2/6 3/6		Decrease in gravel, increase in fines content		5	
5						
	2/6 3/6 6/6	SP-SM	POORLY GRADED SAND WITH SILT; loose, damp, fine to coarse grained, brown		9	
10						
	4/6 5/6 6/6		Medium dense, decrease in fines content, increase in sand content		11	
15			Botom of Boring B-23 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-24

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 16, 2021

Elevation:

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	1/6 2/6 1/6	SM	SILTY SAND; very loose, dry, fine to medium grained, gray, with trace gravel		3	
	4/6 4/6 6/6		Loose, decrease in gravel content	DD = 107.9 pcf ø = 31° c = 180 psf	10	0.8
5	2/6 4/6 6/6	SP-SM	POORLY GRADED SAND; loose, damp, fine to medium grained, brown, with trace gravel		10	
10	6/6 7/6 11/6		Medium dense, decrease in fines content		18	
	4/6 5/6 6/6				11	
15			Bottom of Boring B-24 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-25

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 16, 2021

Elevation:

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	1/6 1/6 1/6	SM	SILTY SAND; very loose, dry, fine to medium grained, brown, trace fine gravel	Gravel = 0.8% Sand = 80.3% -200 = 18.9% EI = 0	2	0.5
	2/6 2/6 3/6		Loose	From 1.5-3': Gravel = 1.0% Sand = 79.7% -200 = 19.3%	5	0.7
5	7/6 14/6 30/6		Medium dense, gray, increase in fines content and decrease in sand content	From 5-6.5': DD = 111.3 pcf Gravel = 2.2% Sand = 67.5% -200 = 30.3%	44	1.7
10	7/6 6/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, moist, fine grained, light brown		13	
	2/6 5/6 7/6		Decrease in fines content		12	
15			Bottom of Boring B-24 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-26

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 16, 2021

Elevation:

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	1/6 2/6 2/6	SM	SILTY SAND; very loose, dry, fine to medium grained, brown, with trace gravel		4	0.3
	10/6 10/6 13/6		Medium dense, damp, decrease in fines content	DD = 94.3 pcf	23	1.3
5						
	3/6 5/6 8/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown, with trace gravel		13	
10						
	3/6 5/6 8/6		Decrease in silt content		13	
15			Bottom of Boring B-26 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-27

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Logged By: Y.A.

Drilled By: J.C.

Date: November 16, 2021

Drill Type: CME 75

Elevation:

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 Pound Auto Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6, 2/6, 1/6	SM	SILTY SAND; very loose, damp, fine to medium grained, brown		3	
	3/6, 5/6, 5/6		Loose, with gravel	DD = 109.0 pcf	10	1.0
5	2/6, 6/6, 5/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, brown		11	
10	5/6, 7/6, 10/6		Medium dense, decrease in fines content		17	
15	3/6, 5/6, 8/6		Moist		13	
	3/6, 5/6, 7/6	SM	SILTY SAND; medium dense, moist, fine grained, grayish brown		12	
20			Bottom of Boring B-27 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-28

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	1/6 2/6 2/6	SM	SILTY SAND; very loose, damp, fine to medium grained, brown		4	1.0
	2/6 2/6 2/6		Increase in fines content		4	
5	13/6 17/6 20/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine grained, brown	DD = 105.1 pcf	37	2.7
	3/6 3/6 7/6		Loose, decrease in fines content, with gravel		10	
10						
	5/6 7/6 9/6	SM	SILTY SAND; medium dense, moist, fine grained, brown		16	
15						
	11/6 5/6 11/6		Increase in moisture		16	
20			Bottom of Boring B-28 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-29

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	2/6 4/6 6/6	SM	SILTY SAND; loose, dry, fine grained, brown		10	0.5
	3/6 5/6 1/6		Moist		6	
5						
	2/6 8/6 5/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown		13	
10						
	3/6 5/6 7/6		Increase in fine grained sand, decrease in silt content		12	
15			Bottom of Boring B-29 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-30

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	2/6 1/6 2/6	SM	SILTY SAND; very loose, dry, fine to medium grained, brown		3	0.5
	11/6 18/6 35/6		Dense, moist, with weak cementation	DD = 113.9 pcf	53	6.4
5						
	3/6 5/6 6/6		Medium dense, no cemenation, with a little fine gravel	Gravel = 7.1% Sand = 68.2% -200 = 24.7%	11	
10						
	7/6 4/6 7/6		Decrease in fines content, with gravel		11	
15						
	7/6 8/6 10/6		Moist, fine grained, brown, increase in fines content		18	
20			Bottom of Boring B-30 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-31

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	2/6 1/6 1/6	SM	SILTY SAND; very loose, damp, fine to medium grained, brown	From 0-3.5': pH = 7.8 SR = 28,000 ohm-cm Cl < 0.00060% SS < 0.00060% DD = 106.6 pcf	2	
	3/6 5/6 7/6		Loose, increase in grain size, fine to coarse grained		12	1.8
5	8/6 6/6 2/6		Moist		8	
10	1/6 8/6 6/6		Medium dense, light brown		14	
15	2/6 5/6 10/6		Fine grained, brown, increase in fines content		15	
			Bottom of Boring B-31 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-1

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	3/6 4/6 8/6	SM	SILTY SAND; medium dense, damp, fine to medium grained, light brown		12	
5	5/6 5/6 7/6	SP	POORLY GRADED SAND; medium dense, damp, fine to medium grained, light brown, with trace fine gravel		12	
10						
15	3/6 4/6 5/6	SM	SILTY SAND; loose, damp, fine grained, brown, trace fine gravel, high fines content	Gravel = 0.2% Sand = 56.0% -200 = 43.8% LL = Non-viscous PI = Non-plastic	9	
			Bottom of Percolation Test Boring P-1 at 15 feet			
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-2

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 17, 2021

Elevation:

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	2/6 3/6 3/6	SM	SILTY SAND; loose, damp, fine to medium grained, brown		6	
5	3/6 5/6 6/6		Medium dense, increase in fines content		11	
	5/6 9/6 15/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, light brown, with a little fine gravel	Gravel = 6.1% Sand = 88.1% -200 = 5.8%	24	
10			Bottom of Percolation Test Boring P-2 at 10 feet			
15						
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-3

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Drilled By: J.C.

Drill Type: CME 75

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip

Logged By: Y.A.

Date: November 15, 2021

Elevation:

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	16, 2/6, 3/6	SM	SILTY SAND; loose, damp, fine to medium grained, brown		5	
5	1/6, 1/6, 1/6		Very loose		2	
10	2/6, 3/6, 7/6	SP-SM	POORLY GRADED SAND WITH SILT; Loose, damp, fine to medium grained, brown, with some fine to coarse gravel	Gravel = 11.2% Sand = 77.3% -200 = 11.5%	10	
			Bottom of Percolation Test Boring P-3 at 10 feet			
15						
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-4

Project: The Neighborhoods at Lugonia Village, Redlands

Project Number: H02901.01

Logged By: Y.A.

Drilled By: J.C.

Date: November 17, 2021

Drill Type: CME 75

Elevation:

Auger Type: 6-5/8" O.D. Hollow Stem Augers

Depth to Groundwater First Encountered During Drilling: N/E

Hammer Type: 140 Pound Auto Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	1/6 2/6 1/6	SM	SILTY SAND; very loose, damp, fine grained, brown		3	
5						
	8/6 7/6 10/6		Medium dense, increase in fines content		17	
10						
	3/6 3/6 4/6		Loose, with trace gravel	Gravel = 0.2% Sand = 68.4% -200 = 31.4%	7	
15			Bottom of Percolation Test Boring P-4 at 15 feet			
20						
25						

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Silty sand
	Poorly graded sand
	Poorly graded sand with silt
Misc. Symbols	
	Boring continues
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler

Notes:

1. Exploratory borings were drilled between November 15 and 18, 2021 using a CME 75 drill rig equipped with 6-5/8" and 8" outside diameter hollow stem augers.

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.

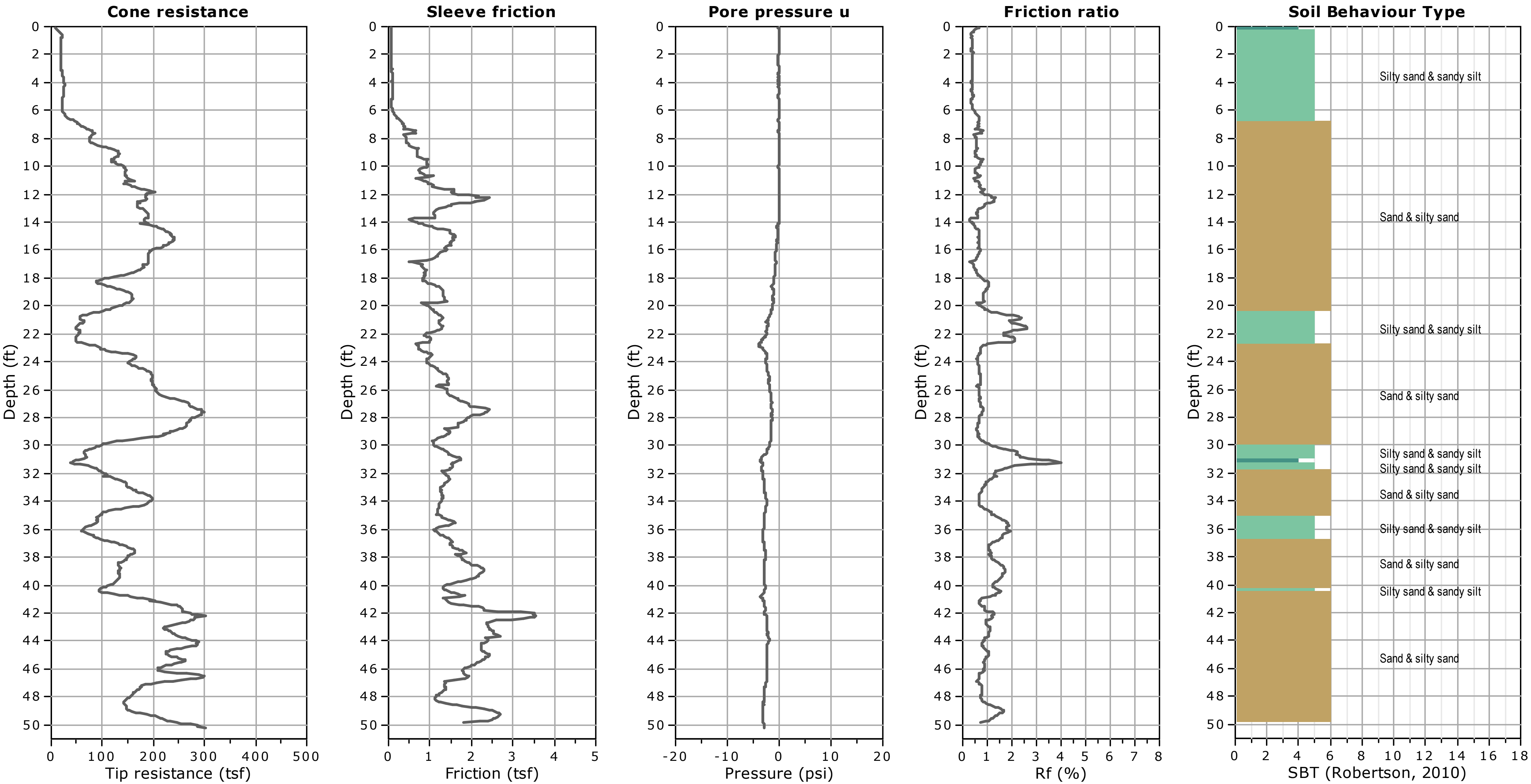
6. Results of tests conducted on samples recovered are reported on the logs.

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(%)
pH	= Soil pH	SR	= Soil resistivity (ohms-cm)
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

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

Project: Moore Twining Associates / The Neighborhoods at Lugonia Village **CPT-1**

Location: Redlands, CA Total depth: 50.21 ft, Date: 11/17/2021



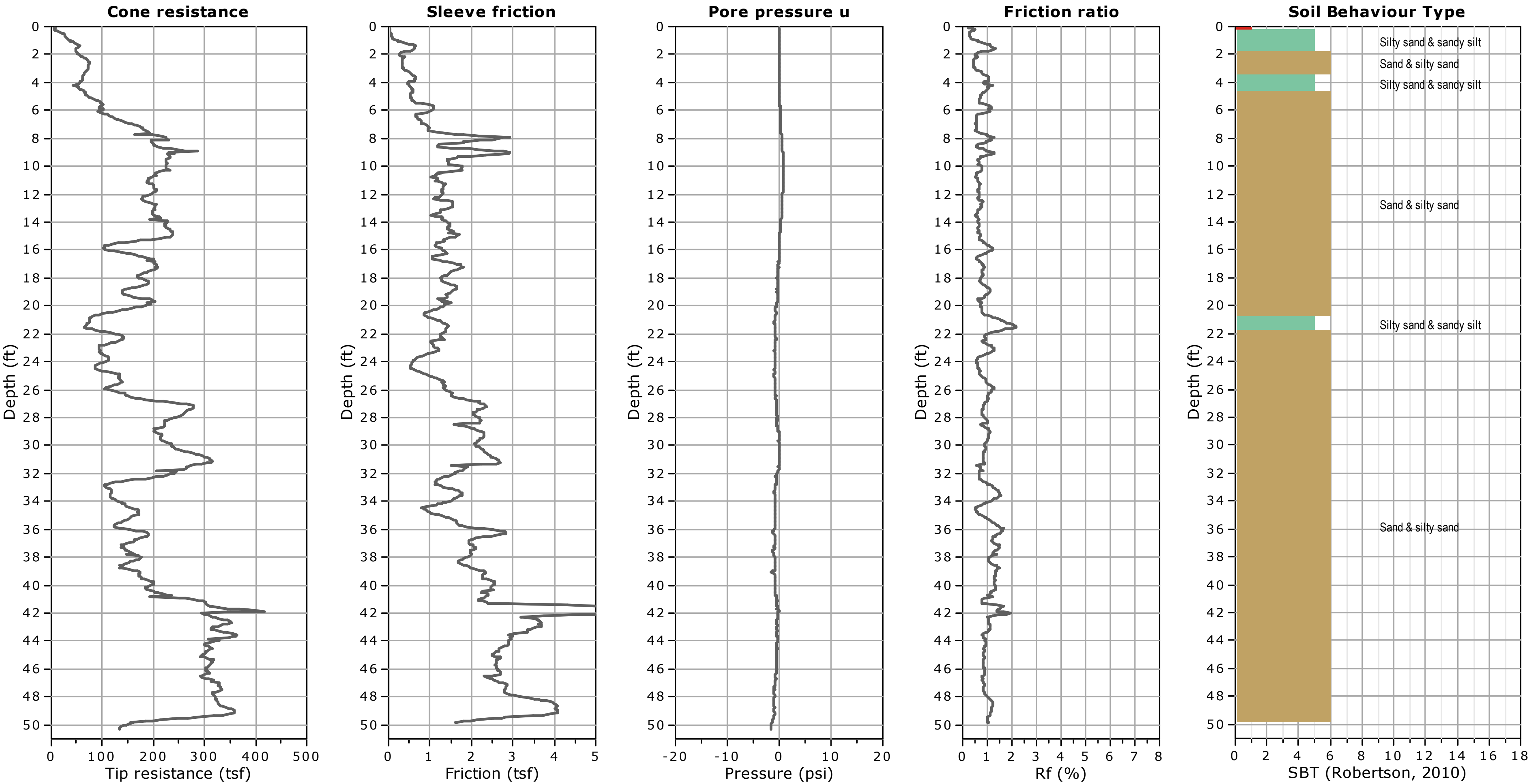
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Moore Twining Associates / The Neighborhoods at Lugonia Village**

Location: **Redlands, CA**

CPT-2

Total depth: 50.28 ft, Date: 11/17/2021



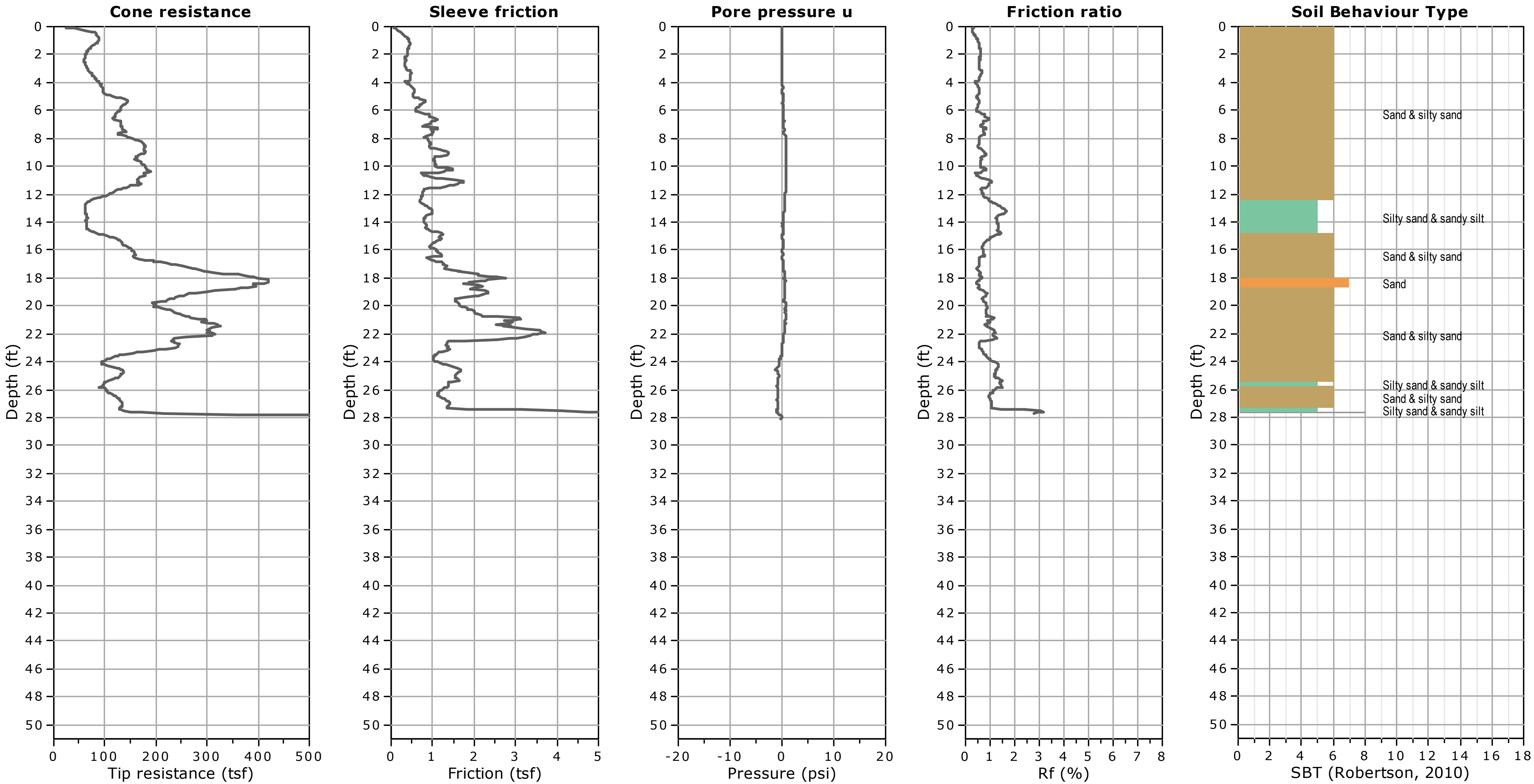
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Moore Twining Associates / The Neighborhoods at Lugonia Village**

Location: **Redlands, CA**

CPT-3

Total depth: 28.08 ft, Date: 11/17/2021



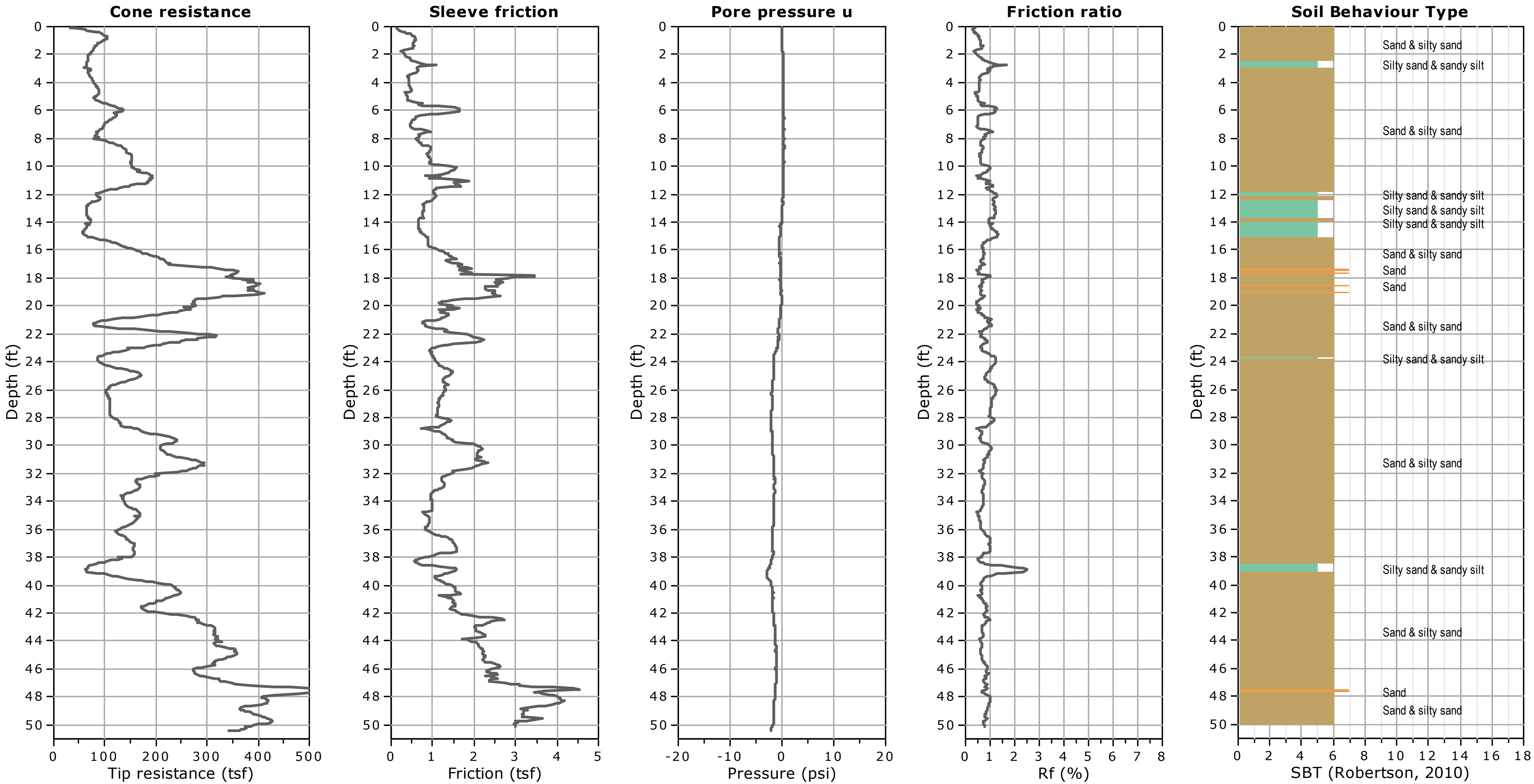
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Moore Twining Associates / The Neighborhoods at Lugonia Village

Location: Redlands, CA

CPT-3A

Total depth: 50.46 ft, Date: 11/17/2021



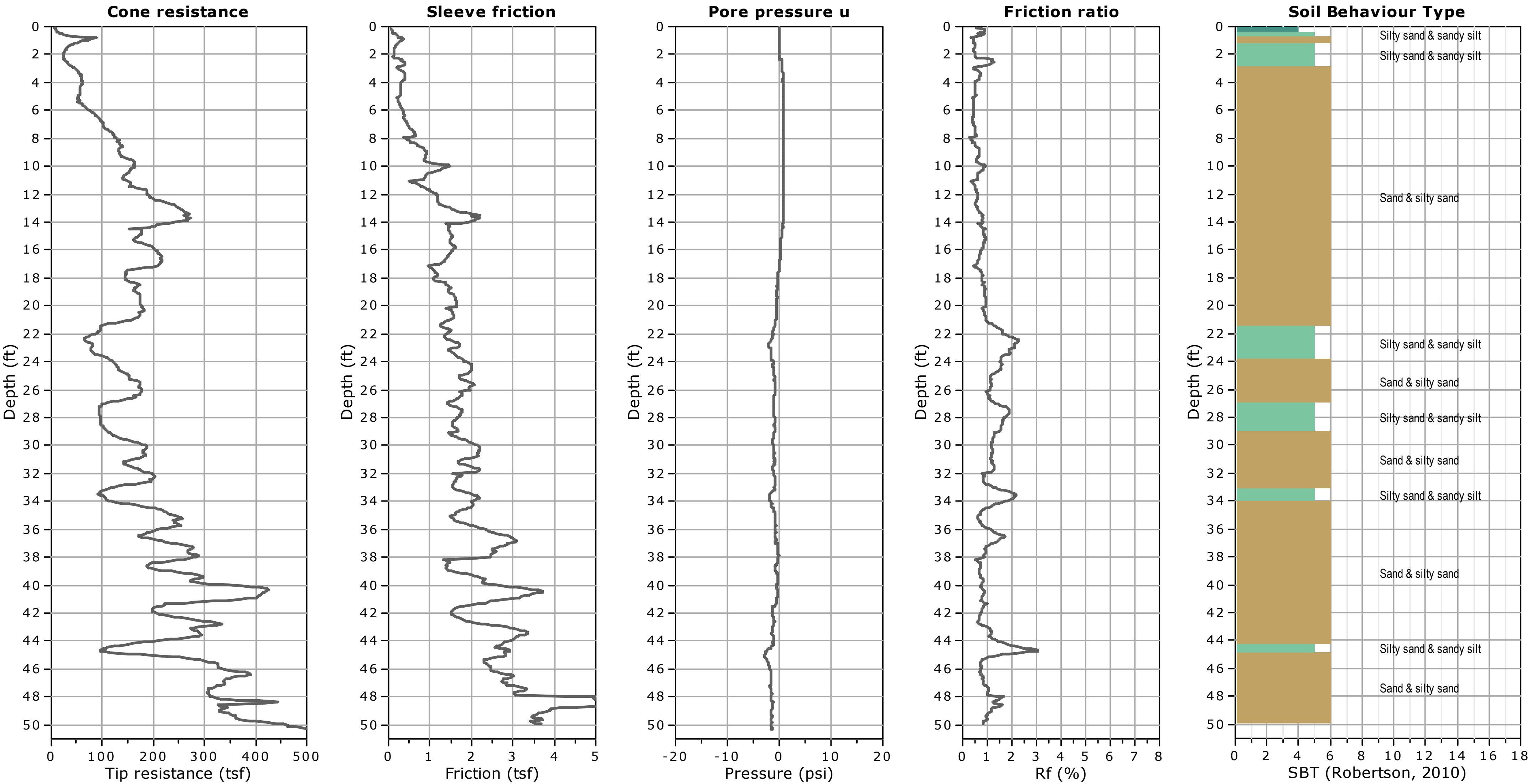
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Moore Twining Associates / The Neighborhoods at Lugonia Village

Location: Redlands, CA

CPT-4

Total depth: 50.35 ft, Date: 11/17/2021



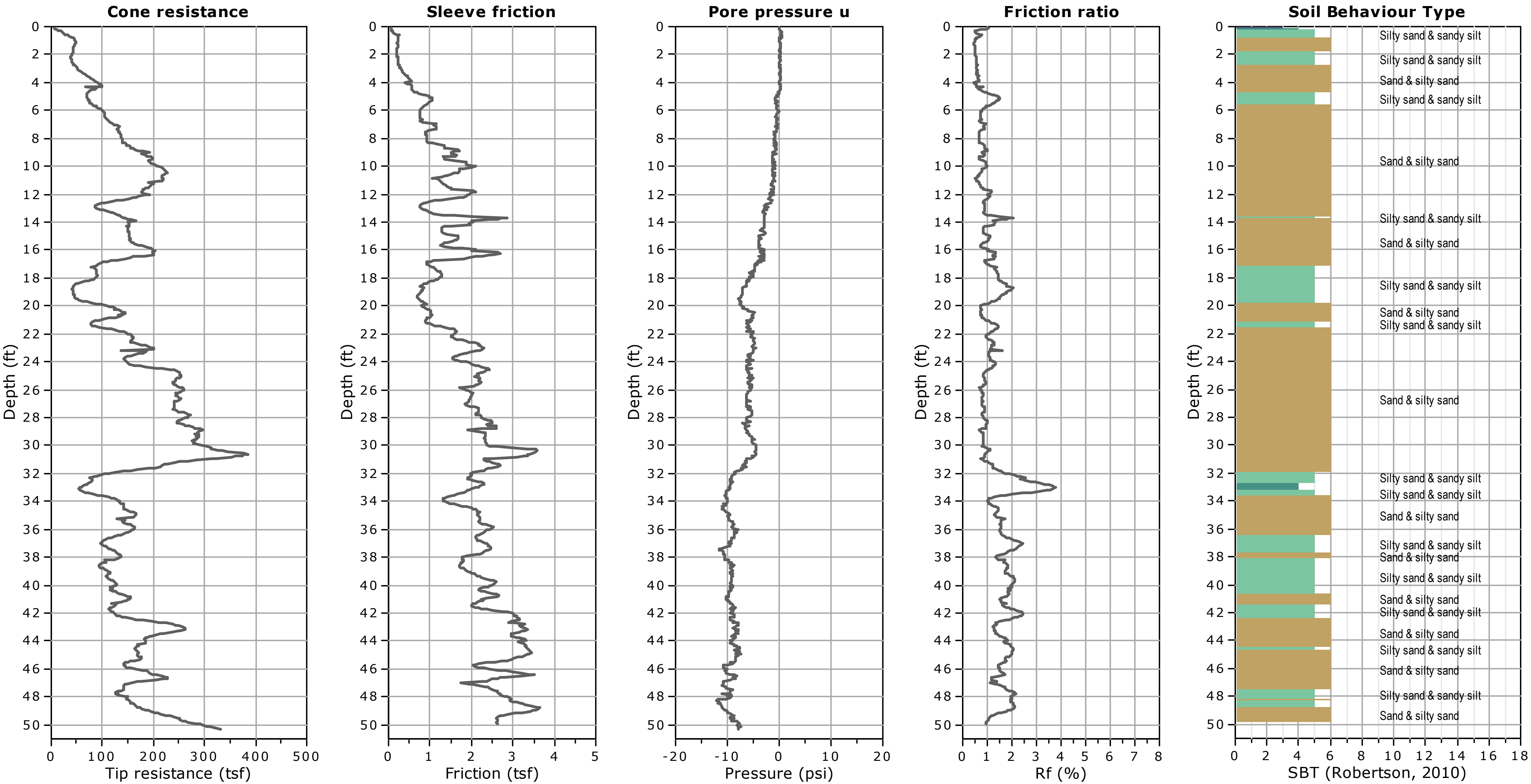
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Moore Twining Associates / The Neighborhoods at Lugonia Village

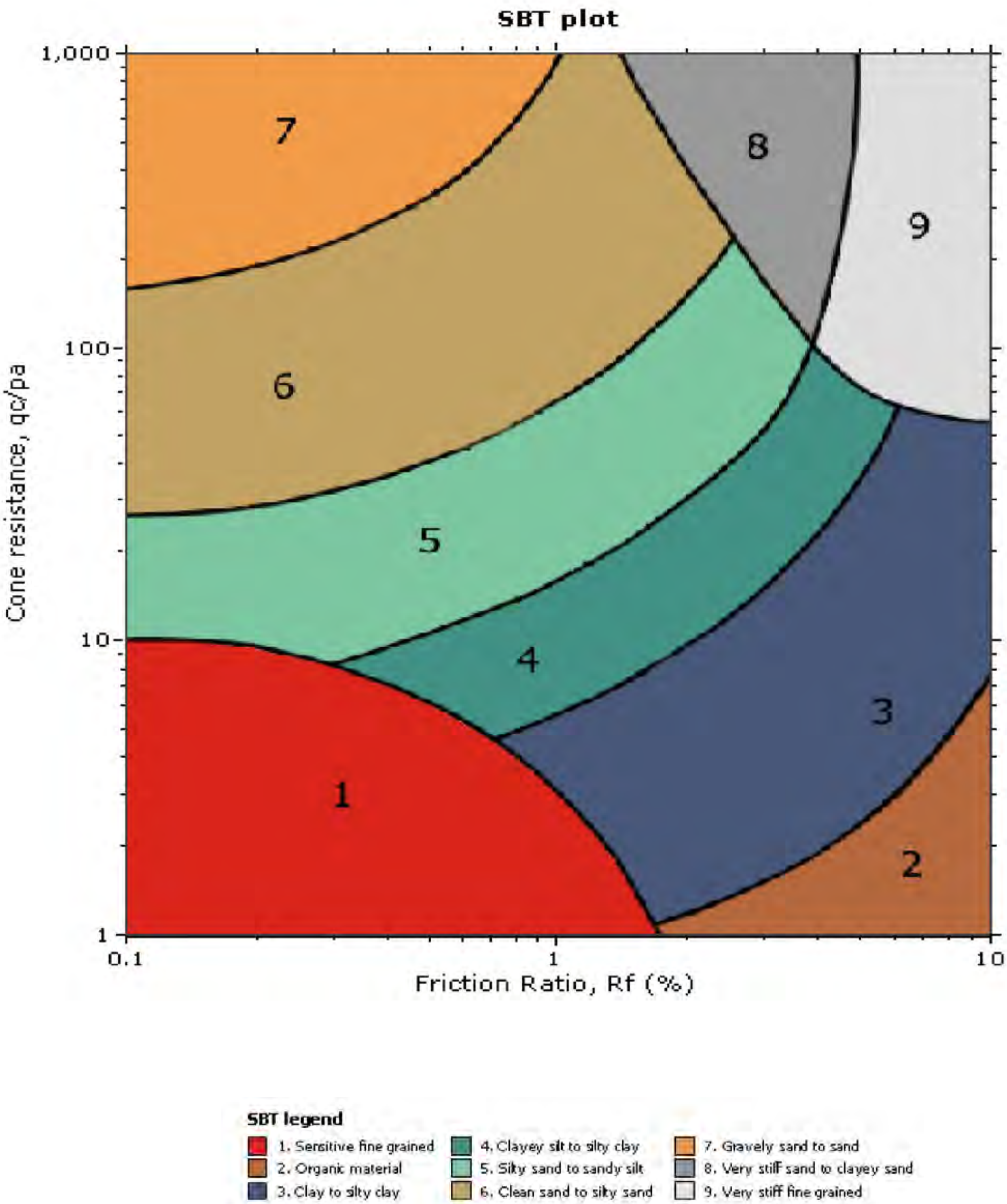
Location: Redlands, CA

CPT-5

Total depth: 50.27 ft, Date: 11/17/2021



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



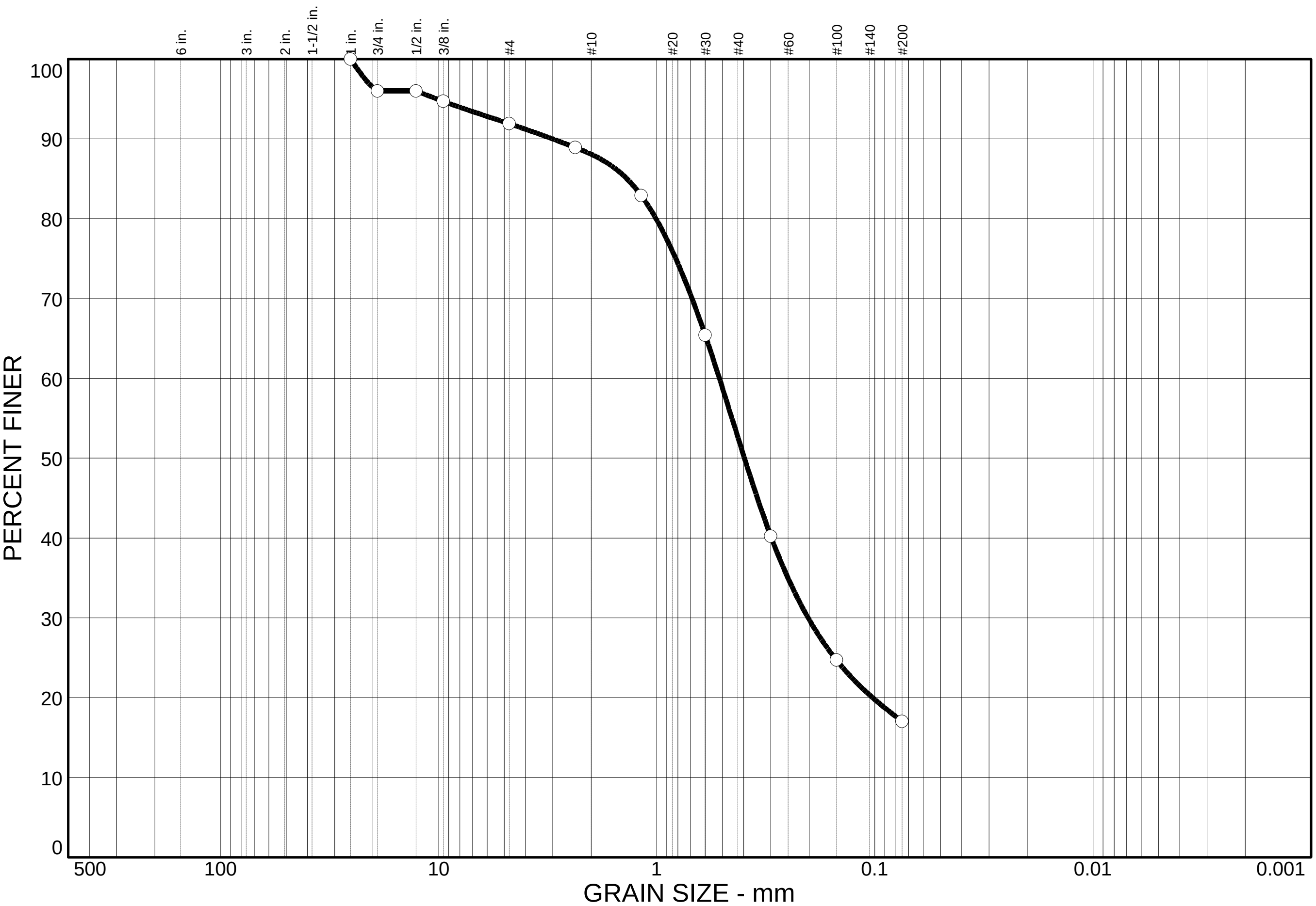
APPENDIX C

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).
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.
R-Value (ASTM D2844)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (ASTM D4327)	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 (ASTM D4327)	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 (ASTM D4972)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	8.1	74.9	17.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	96.0		
1/2 in.	96.0		
3/8 in.	94.7		
#4	91.9		
#8	88.9		
#16	82.9		
#30	65.4		
#50	40.2		
#100	24.7		
#200	17.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 1.37 D_{60}= 0.517 D_{50}= 0.397
D_{30}= 0.202 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-7 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

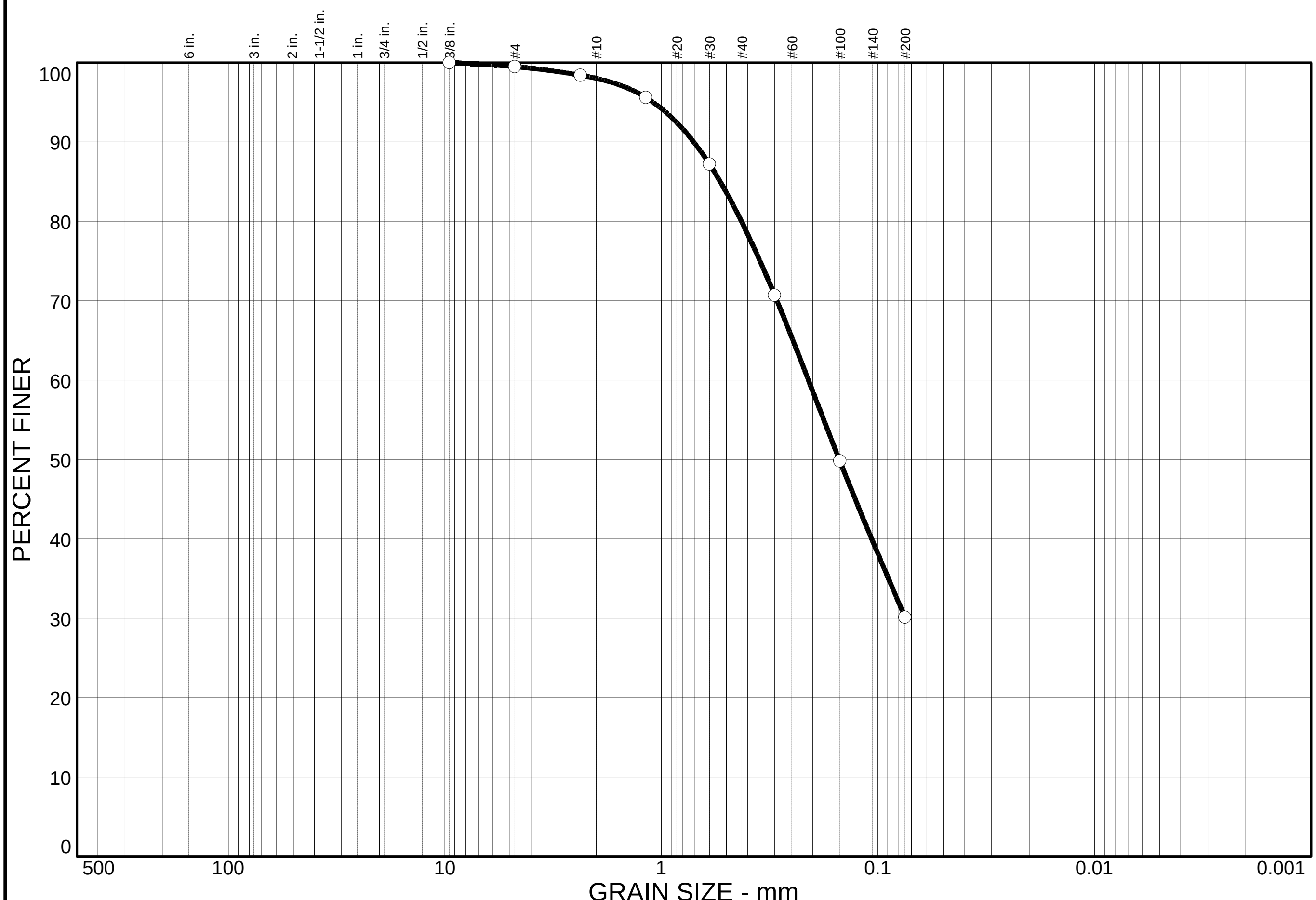
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.5	69.4	30.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.5		
#8	98.4		
#16	95.6		
#30	87.2		
#50	70.7		
#100	49.8		
#200	30.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.535 D_{60}= 0.210 D_{50}= 0.151
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-11 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 3.5-5'

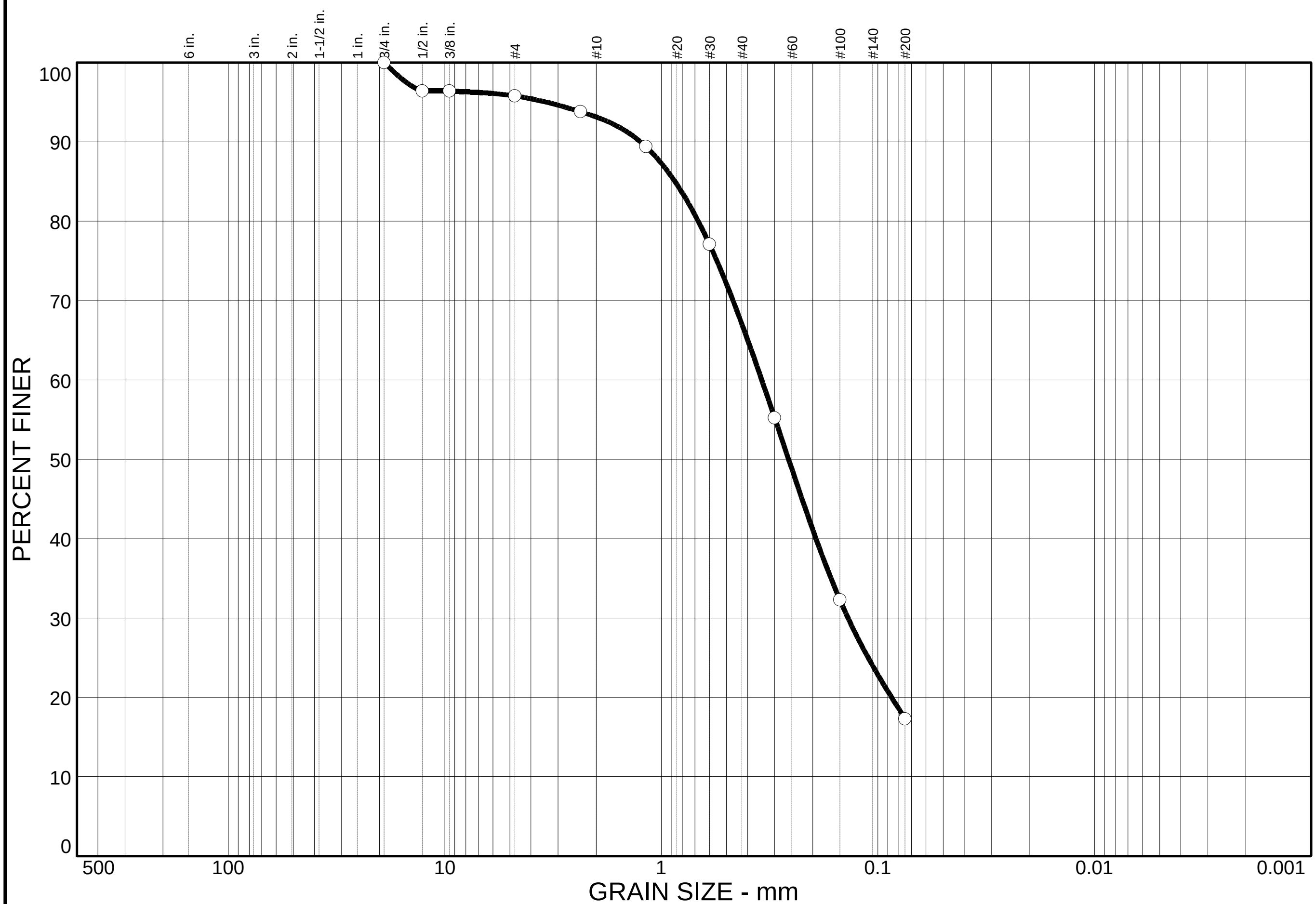
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	4.2	78.5	17.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	96.4		
3/8 in.	96.4		
#4	95.8		
#8	93.8		
#16	89.4		
#30	77.1		
#50	55.2		
#100	32.3		
#200	17.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.866 D_{60}= 0.344 D_{50}= 0.259
D_{30}= 0.138 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-17 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

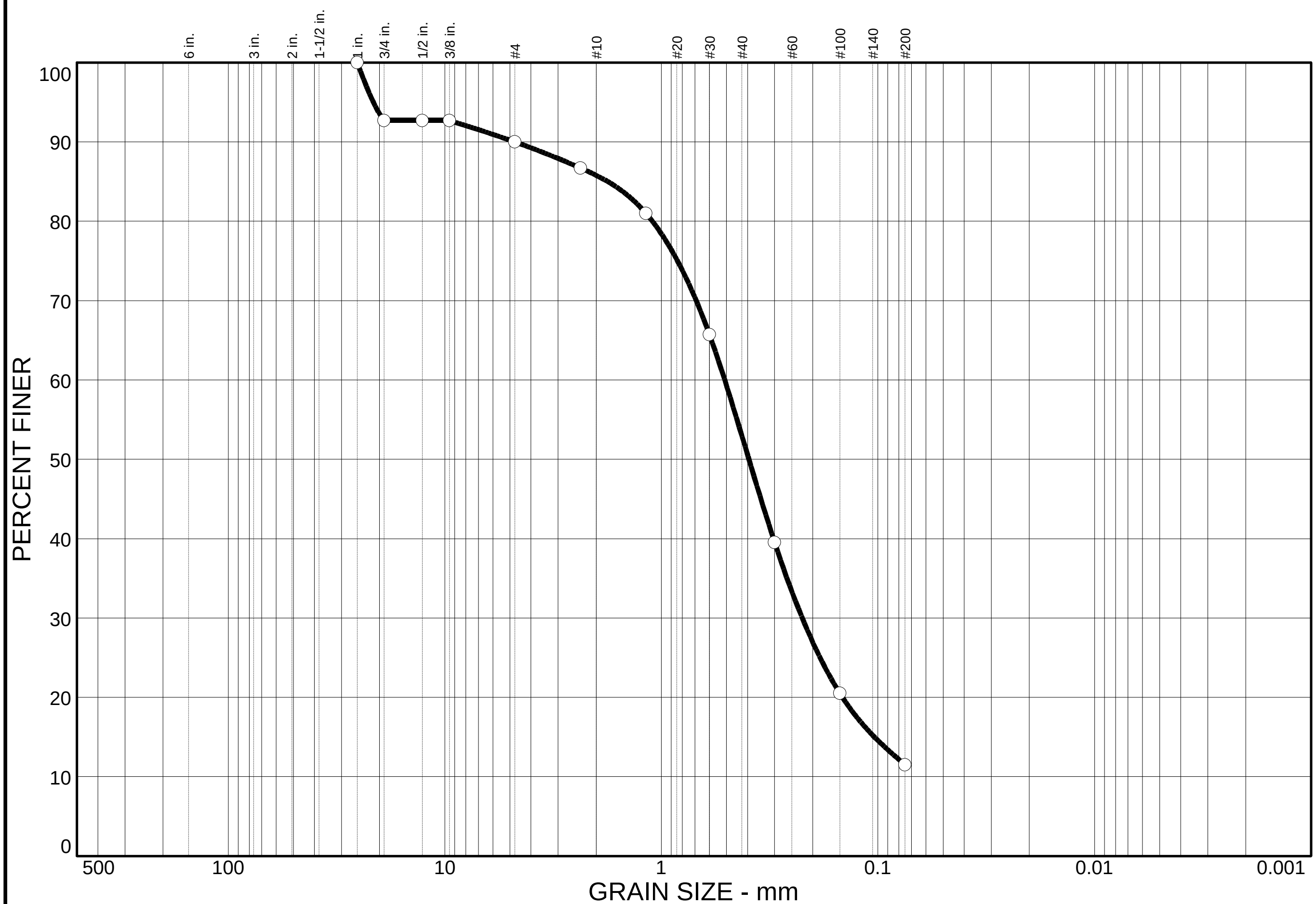
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	10.0	78.5	11.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	92.7		
1/2 in.	92.7		
3/8 in.	92.7		
#4	90.0		
#8	86.7		
#16	81.0		
#30	65.7		
#50	39.5		
#100	20.5		
#200	11.5		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 1.77 D_{60}= 0.510 D_{50}= 0.395
D_{30}= 0.224 D_{15}= 0.104 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 0-1.5'

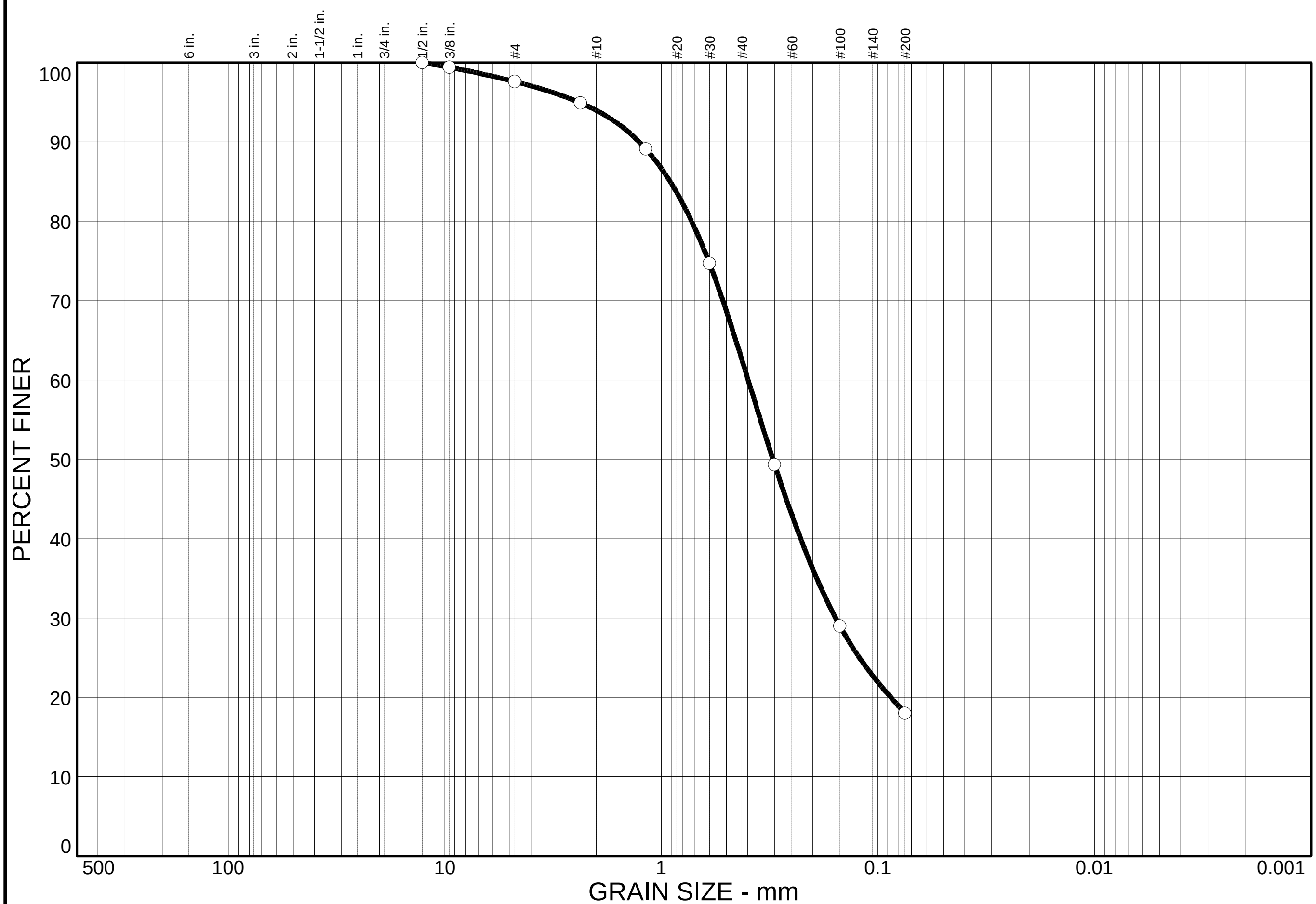
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.4	79.6	18.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.4		
#4	97.6		
#8	94.9		
#16	89.1		
#30	74.7		
#50	49.3		
#100	29.0		
#200	18.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.912 D_{60}= 0.398 D_{50}= 0.306
D_{30}= 0.157 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 3.5-5'

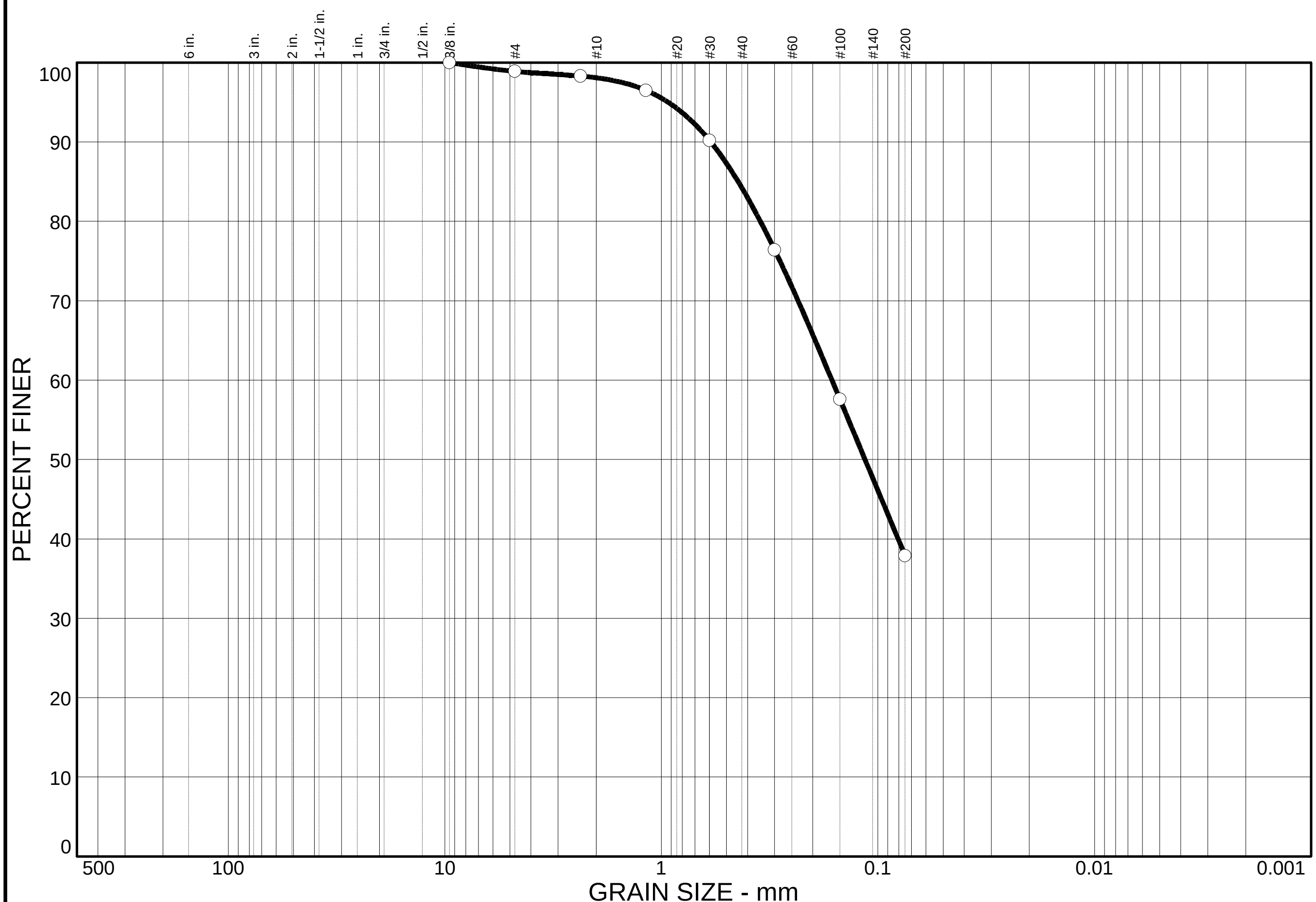
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.1	61.0	37.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.9		
#8	98.3		
#16	96.5		
#30	90.2		
#50	76.4		
#100	57.6		
#200	37.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.442 D_{60}= 0.163 D_{50}= 0.115
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

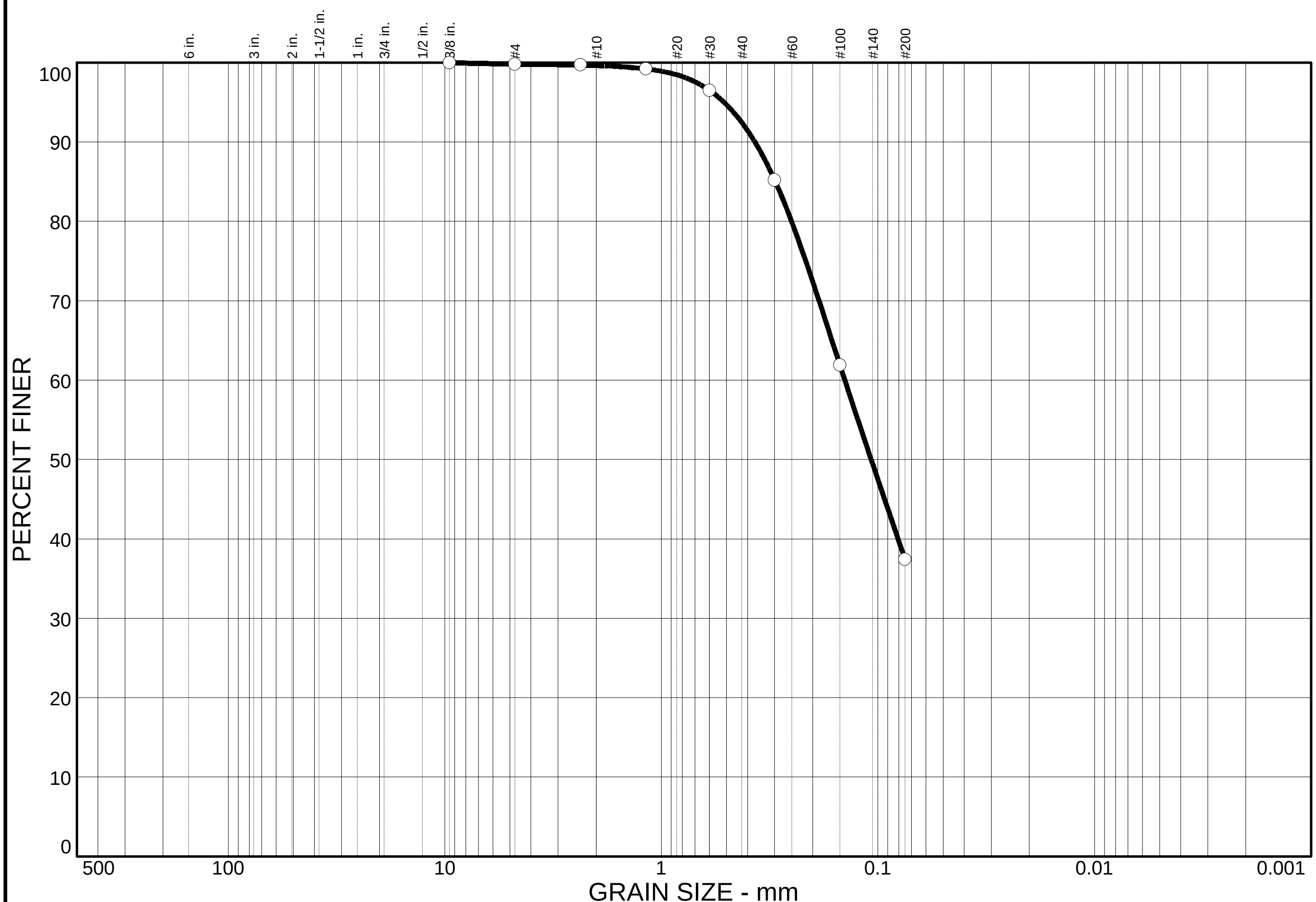
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	62.4	37.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.8		
#8	99.7		
#16	99.2		
#30	96.5		
#50	85.2		
#100	61.9		
#200	37.4		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.298 D_{60}= 0.142 D_{50}= 0.108
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 13.5-15'

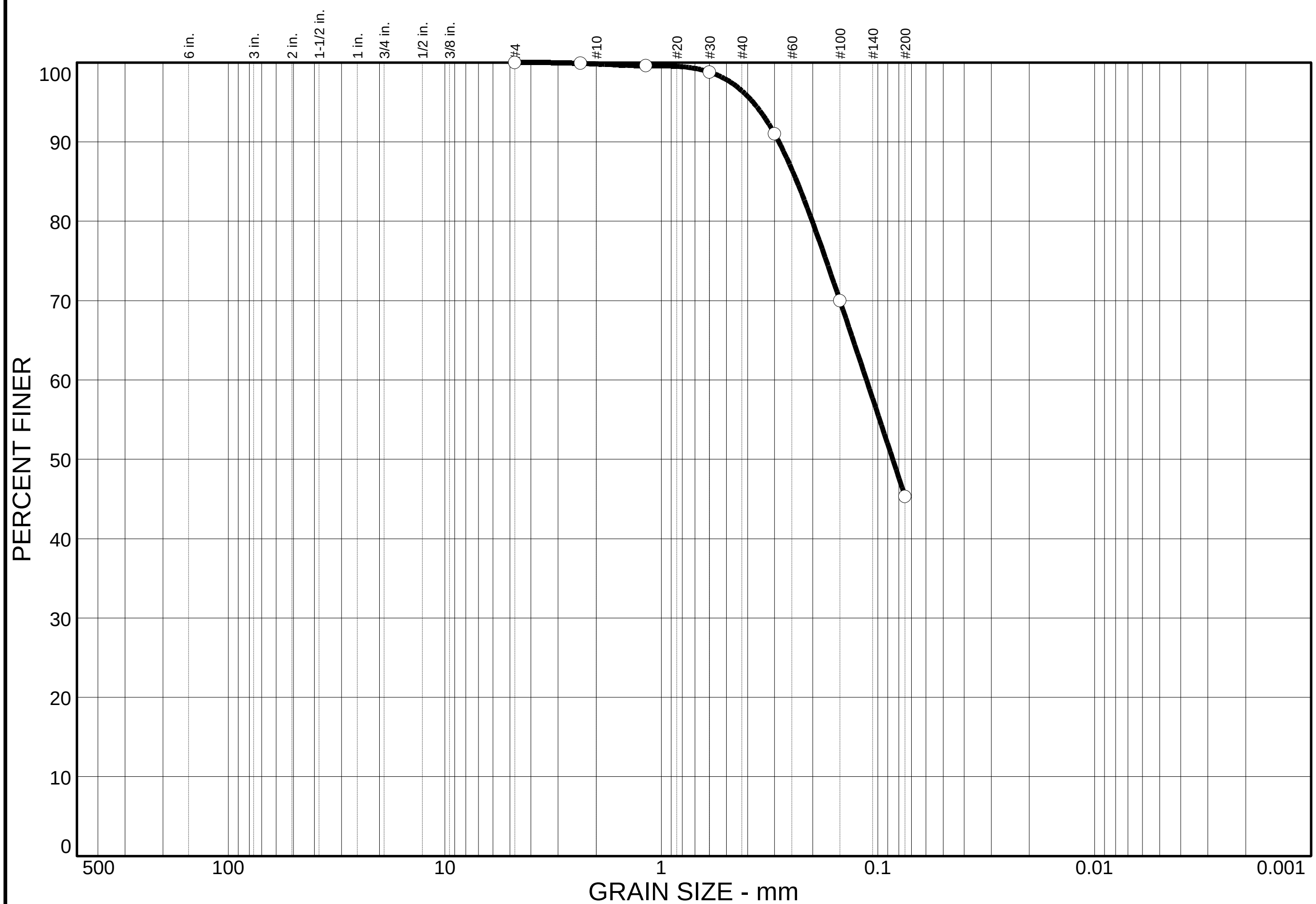
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	54.7	45.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.6		
#30	98.8		
#50	91.0		
#100	70.0		
#200	45.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.237 D_{60}= 0.113 D_{50}= 0.0855
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 23.5-25'

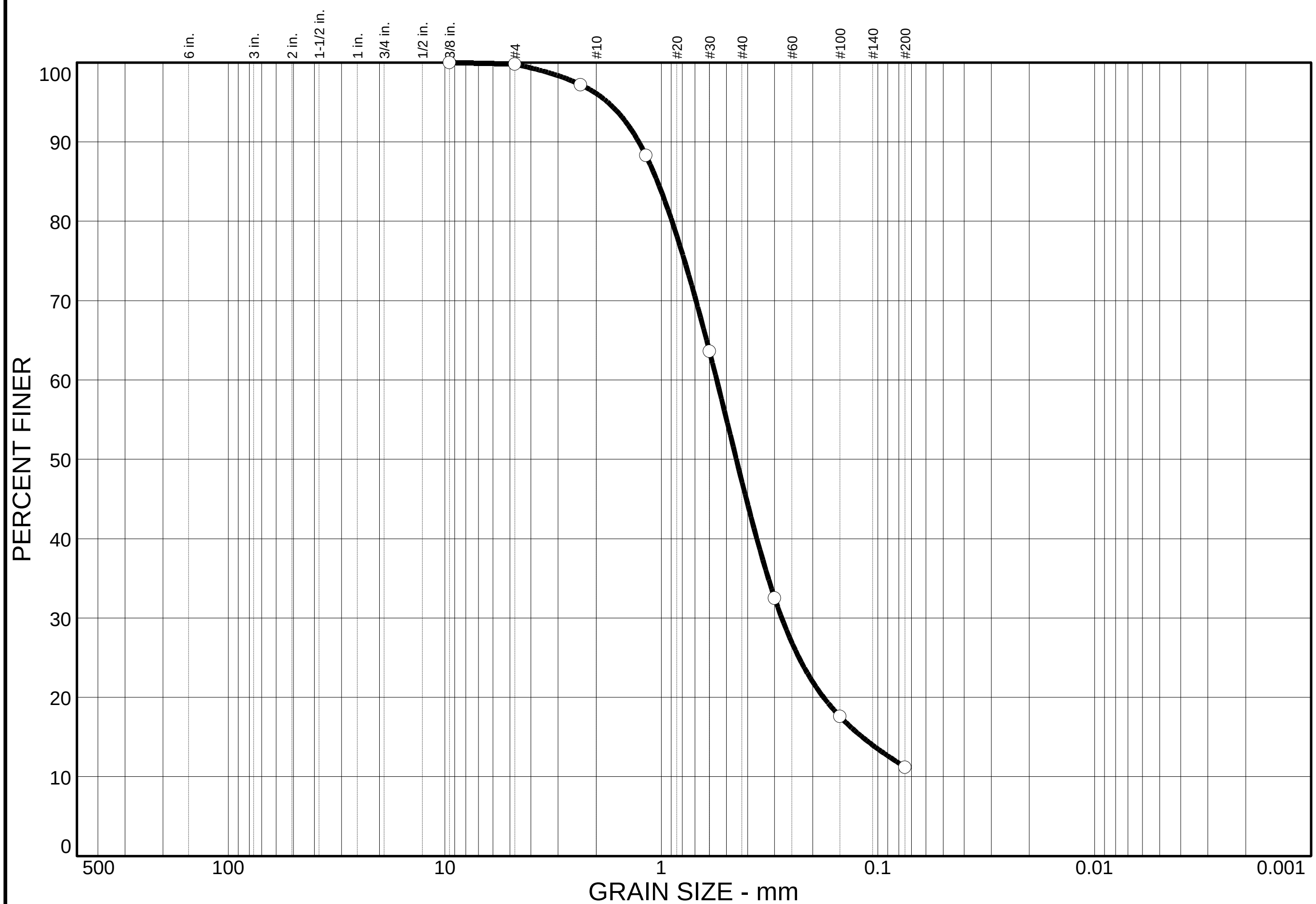
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	88.6	11.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.8		
#8	97.2		
#16	88.3		
#30	63.6		
#50	32.5		
#100	17.6		
#200	11.2		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.04 D_{60}= 0.556 D_{50}= 0.451
D_{30}= 0.278 D_{15}= 0.118 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 28.5-30'

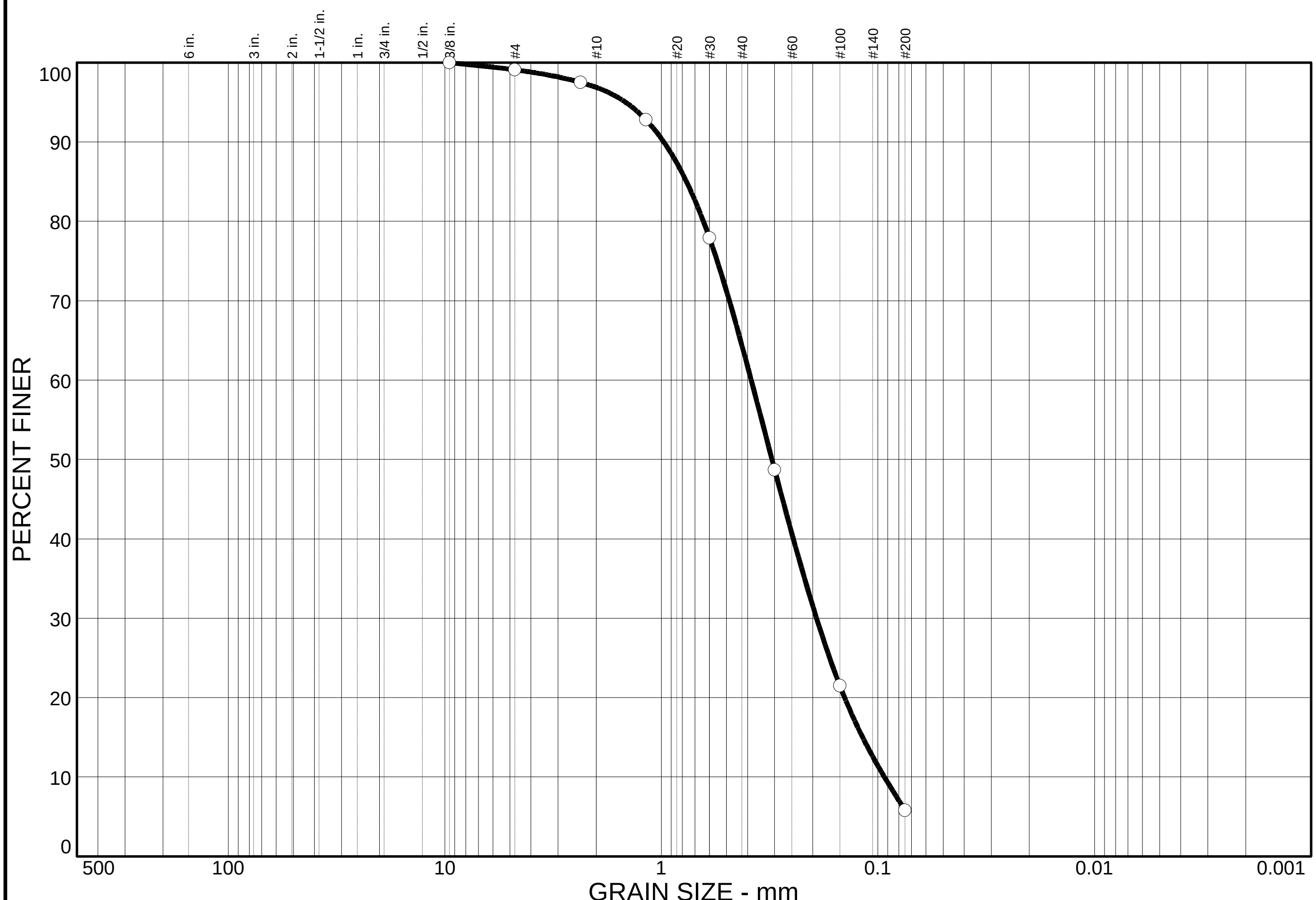
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.9	93.3	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.1		
#8	97.5		
#16	92.8		
#30	77.9		
#50	48.7		
#100	21.5		
#200	5.8		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.766 D_{60}= 0.385 D_{50}= 0.309
D_{30}= 0.192 D_{15}= 0.118 D_{10}= 0.0934
C_u= 4.12 C_c= 1.03

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 33.5-35'

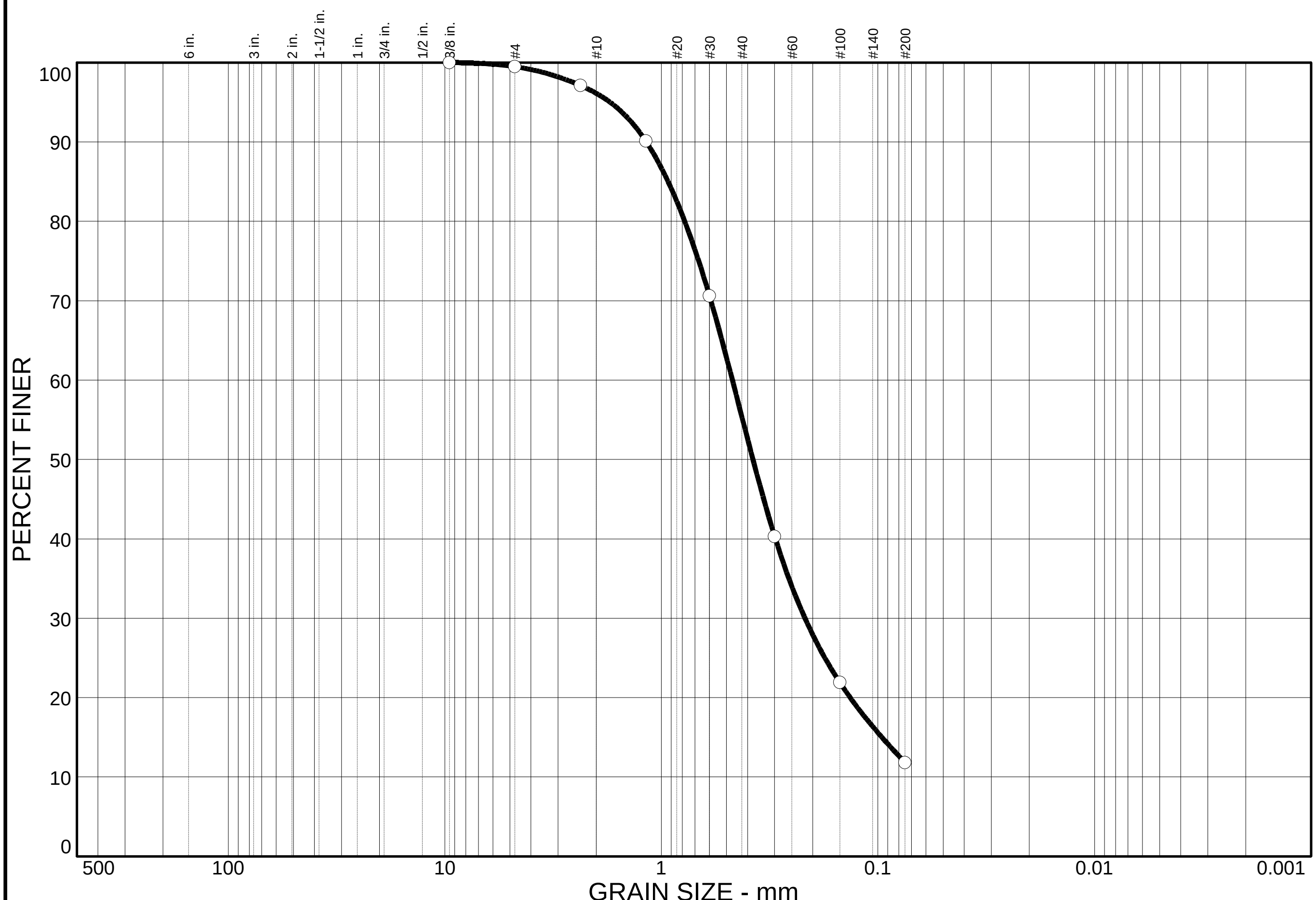
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.5	87.7	11.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.5		
#8	97.1		
#16	90.1		
#30	70.6		
#50	40.3		
#100	21.9		
#200	11.8		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.932 D_{60}= 0.470 D_{50}= 0.378
D_{30}= 0.217 D_{15}= 0.0959 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-22 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 43.5-45'

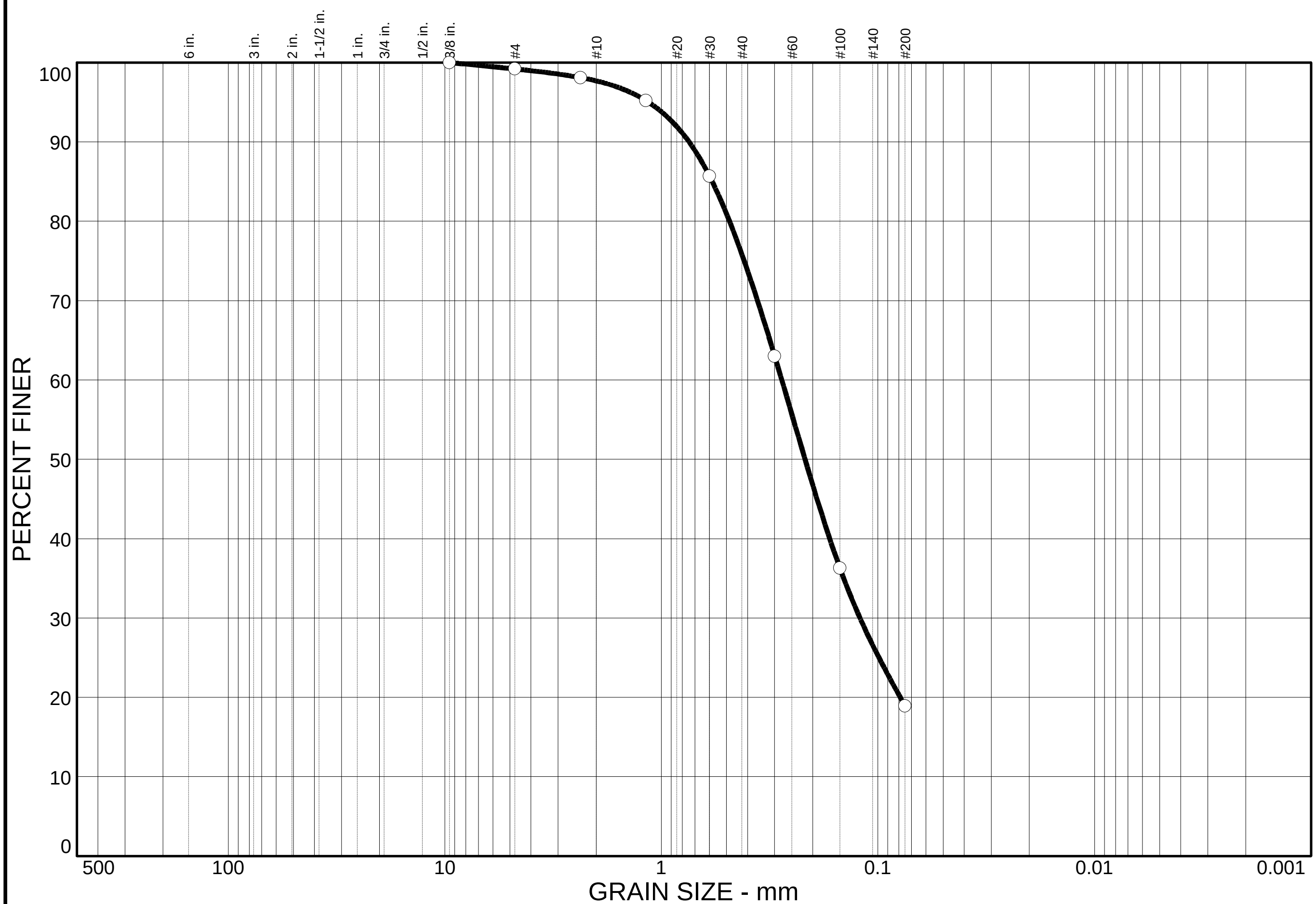
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.8	80.3	18.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.2		
#8	98.1		
#16	95.2		
#30	85.7		
#50	63.0		
#100	36.3		
#200	18.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.583 D_{60}= 0.278 D_{50}= 0.217
D_{30}= 0.121 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-25 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 0-1.5'

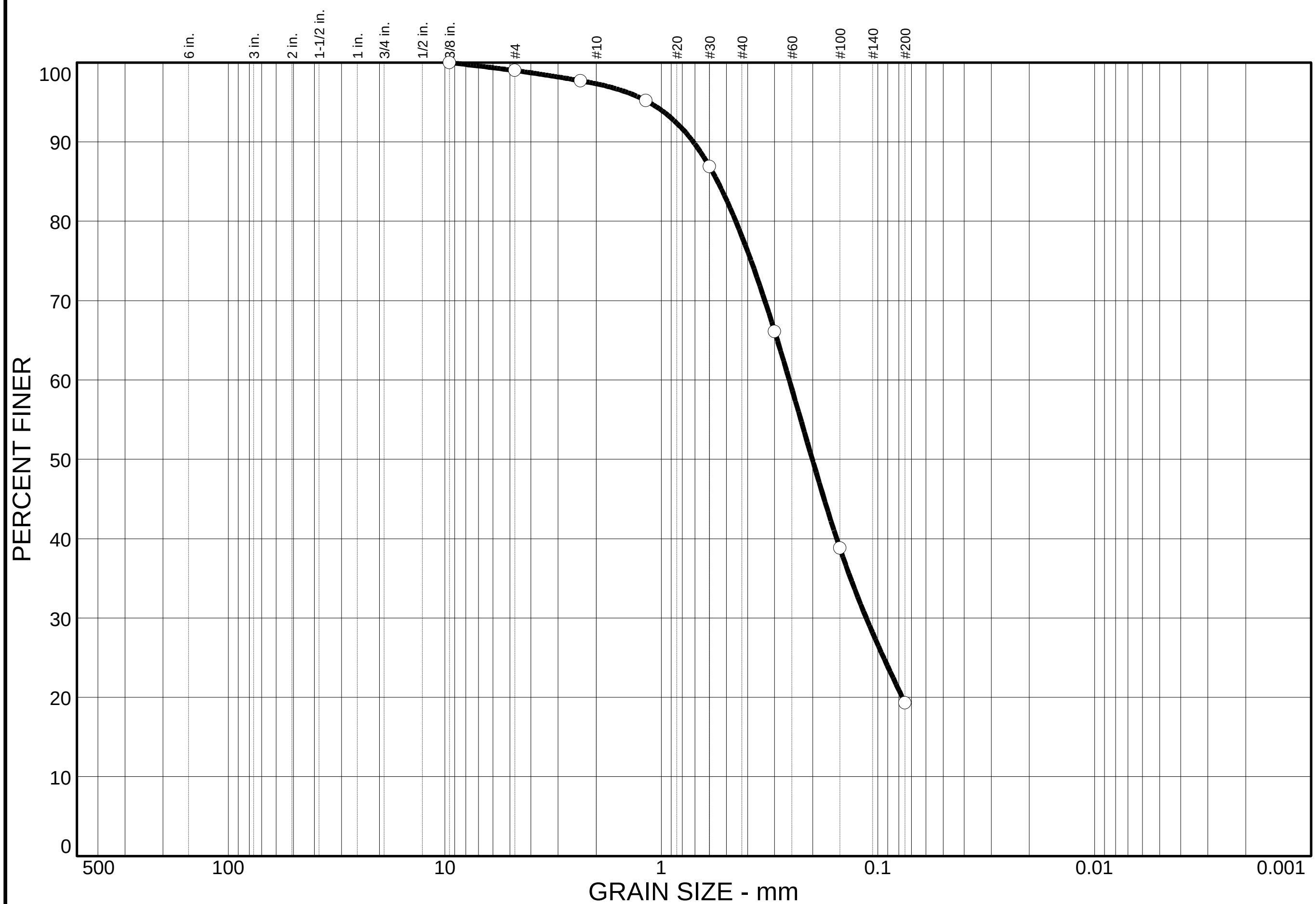
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.0	79.7	19.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.0		
#8	97.7		
#16	95.2		
#30	86.9		
#50	66.1		
#100	38.8		
#200	19.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.550 D_{60}= 0.257 D_{50}= 0.201
D_{30}= 0.113 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-25 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 1.5-3'

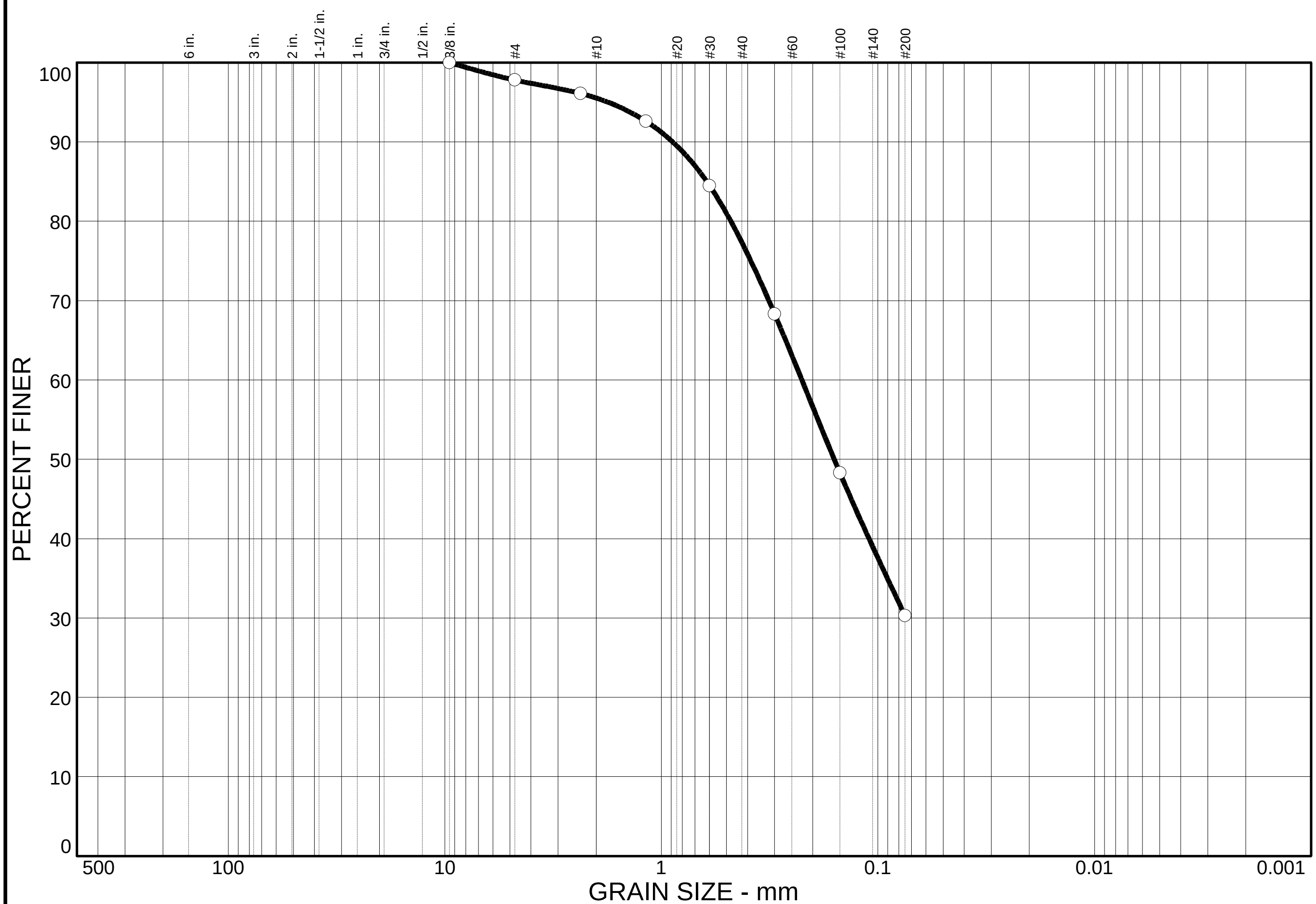
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.2	67.5	30.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	97.8		
#8	96.1		
#16	92.6		
#30	84.5		
#50	68.3		
#100	48.3		
#200	30.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.618 D_{60}= 0.225 D_{50}= 0.159
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-25 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 5-6.5'

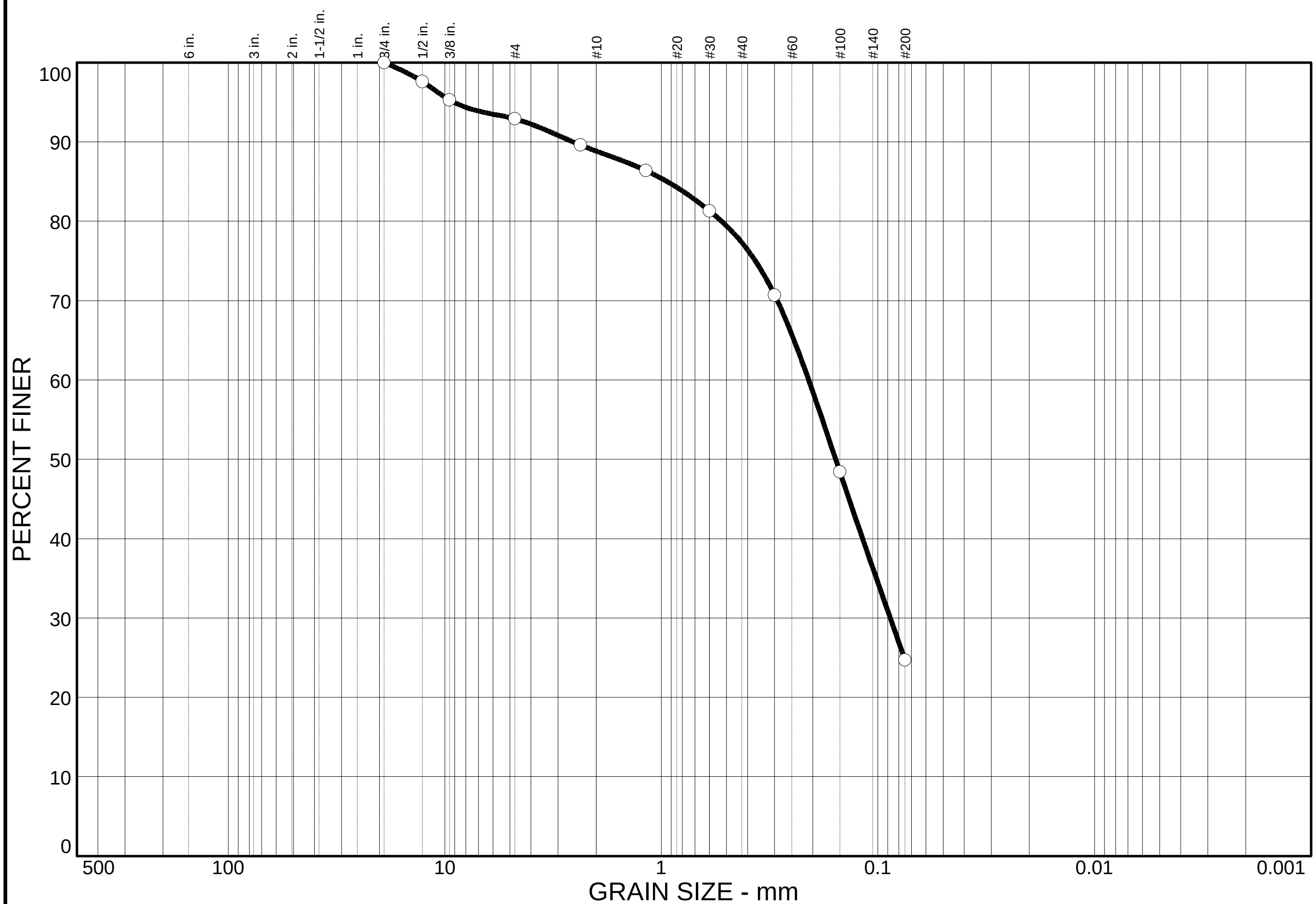
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.1	68.2	24.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	97.6		
3/8 in.	95.3		
#4	92.9		
#8	89.6		
#16	86.4		
#30	81.3		
#50	70.7		
#100	48.4		
#200	24.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.941 D_{60}= 0.209 D_{50}= 0.157
D_{30}= 0.0878 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-30 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

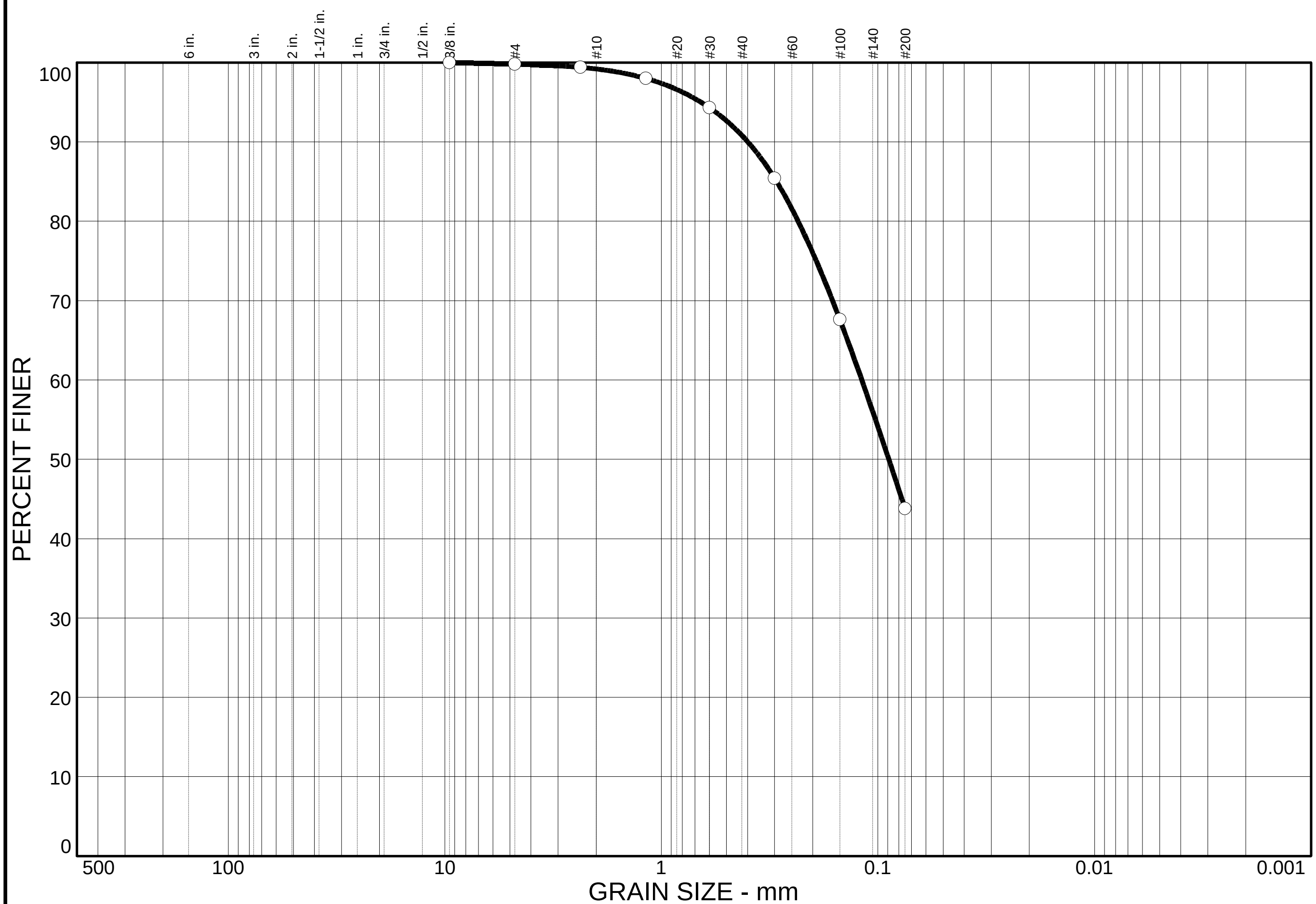
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	56.0	43.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.8		
#8	99.4		
#16	98.0		
#30	94.3		
#50	85.4		
#100	67.6		
#200	43.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.294 D_{60}= 0.119 D_{50}= 0.0892
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: P-1 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 13.5-15'

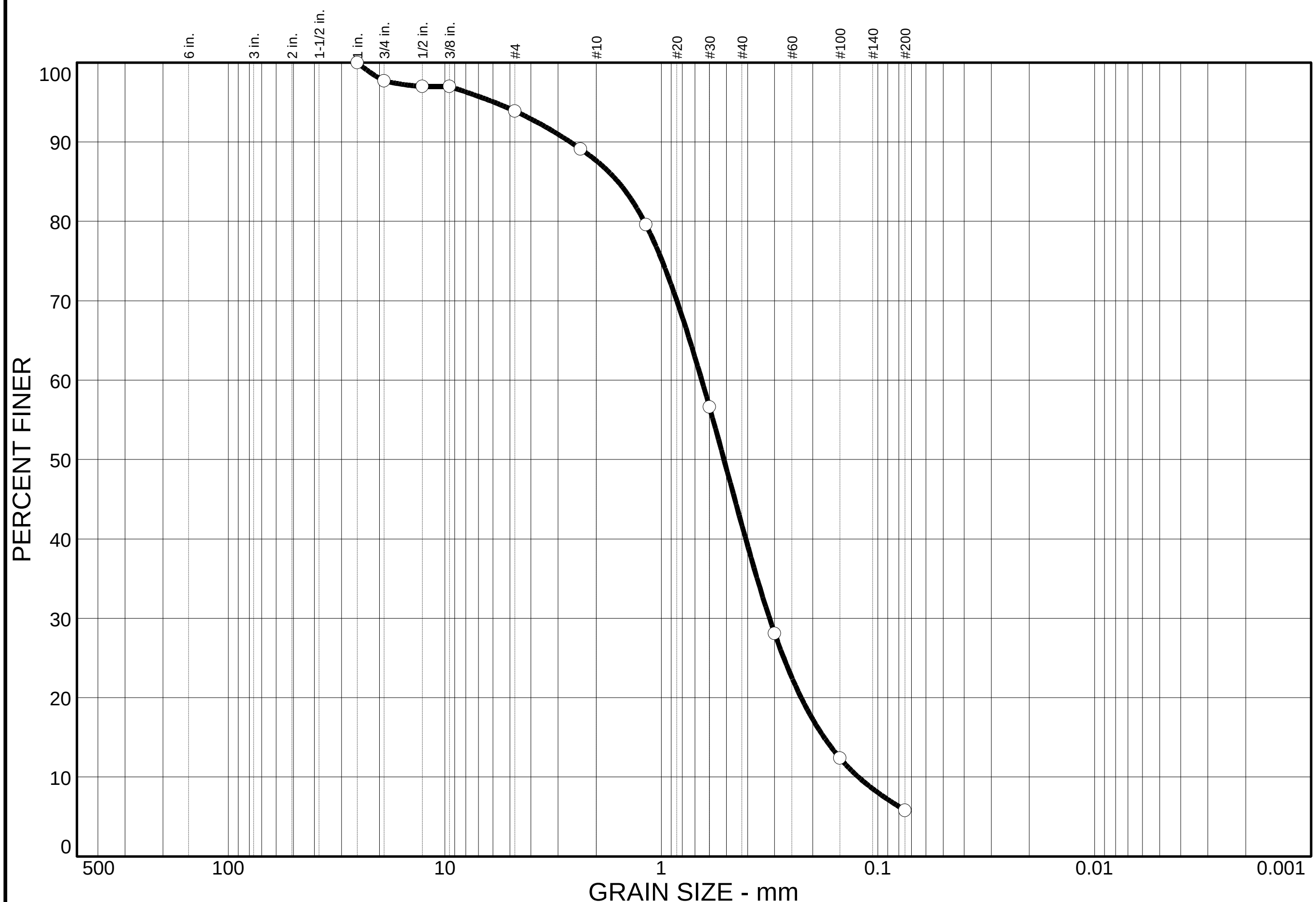
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	6.1	88.1	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	97.7		
1/2 in.	97.0		
3/8 in.	97.0		
#4	93.9		
#8	89.1		
#16	79.6		
#30	56.6		
#50	28.1		
#100	12.4		
#200	5.8		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.59 D_{60}= 0.651 D_{50}= 0.514
D_{30}= 0.317 D_{15}= 0.177 D_{10}= 0.123
C_u= 5.29 C_c= 1.25

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: P-2 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

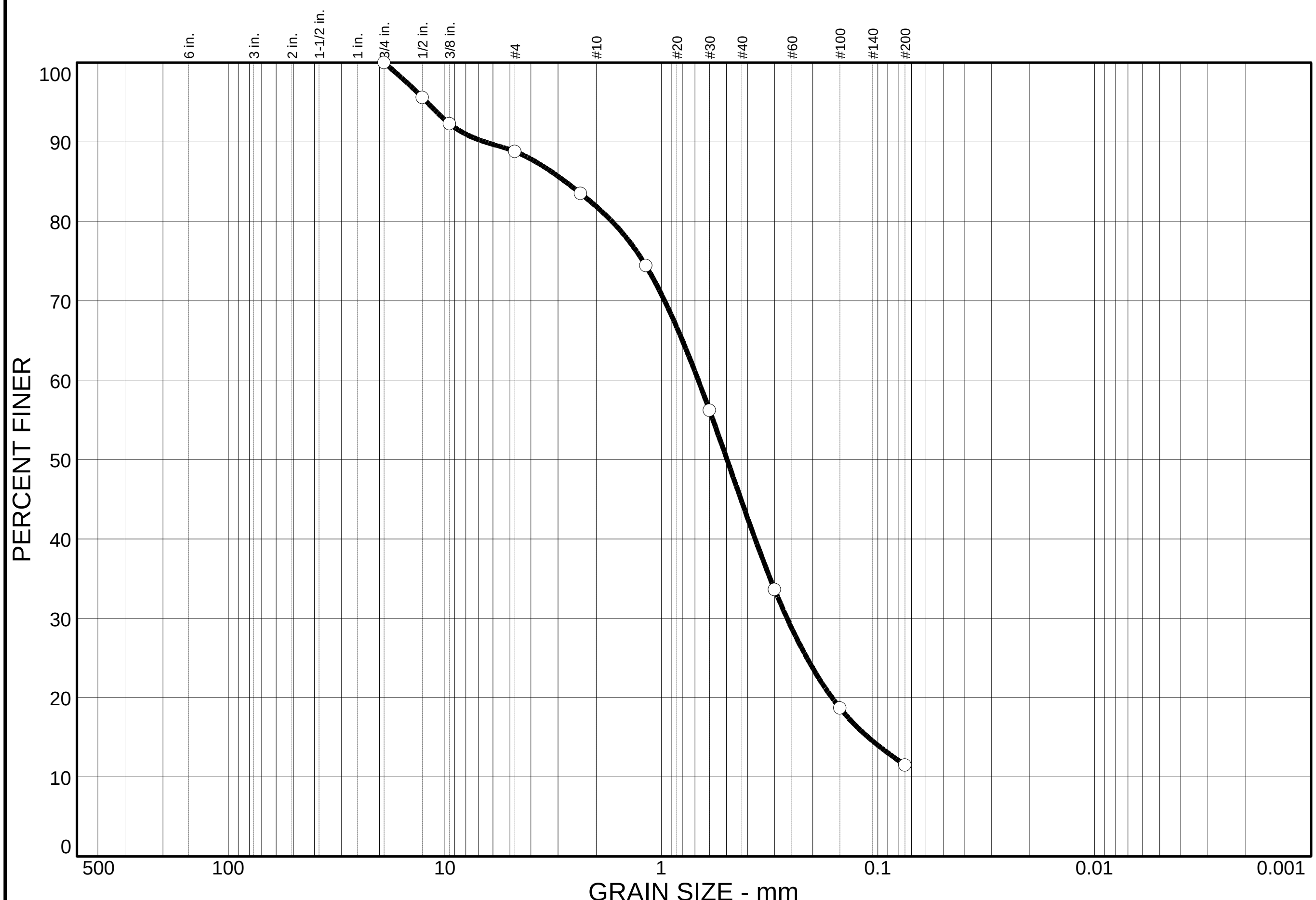
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	11.2	77.3	11.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	95.6		
3/8 in.	92.3		
#4	88.8		
#8	83.5		
#16	74.4		
#30	56.2		
#50	33.6		
#100	18.7		
#200	11.5		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.78 D_{60}= 0.676 D_{50}= 0.498
D_{30}= 0.263 D_{15}= 0.111 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: P-3 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 8.5-10'

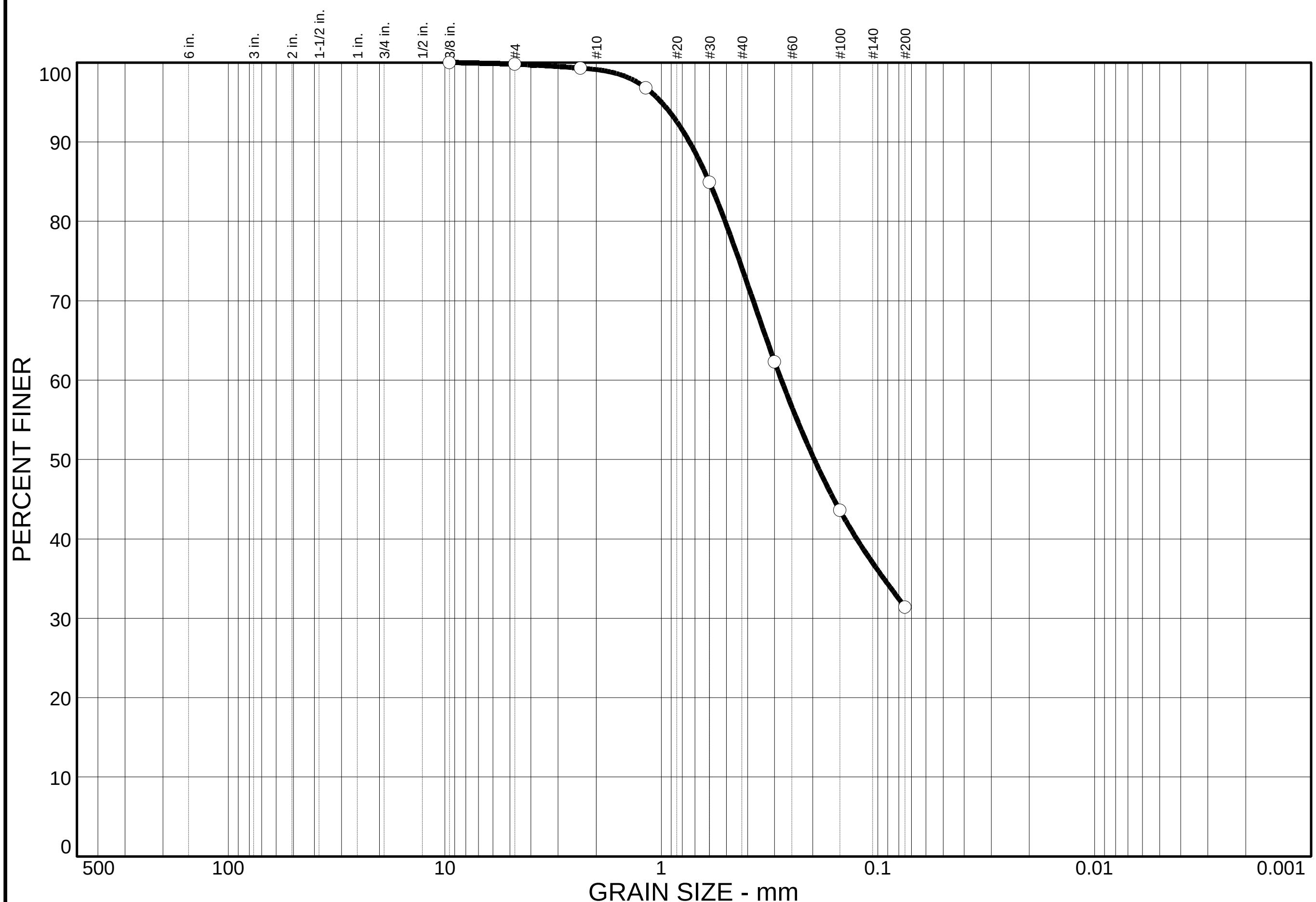
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.2	68.4	31.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.8		
#8	99.3		
#16	96.8		
#30	84.9		
#50	62.3		
#100	43.6		
#200	31.4		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.602 D_{60}= 0.279 D_{50}= 0.197
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: P-4 **Source of Sample:** **Date:** 11/15/21
Location: **Elev./Depth:** 13.5-15'

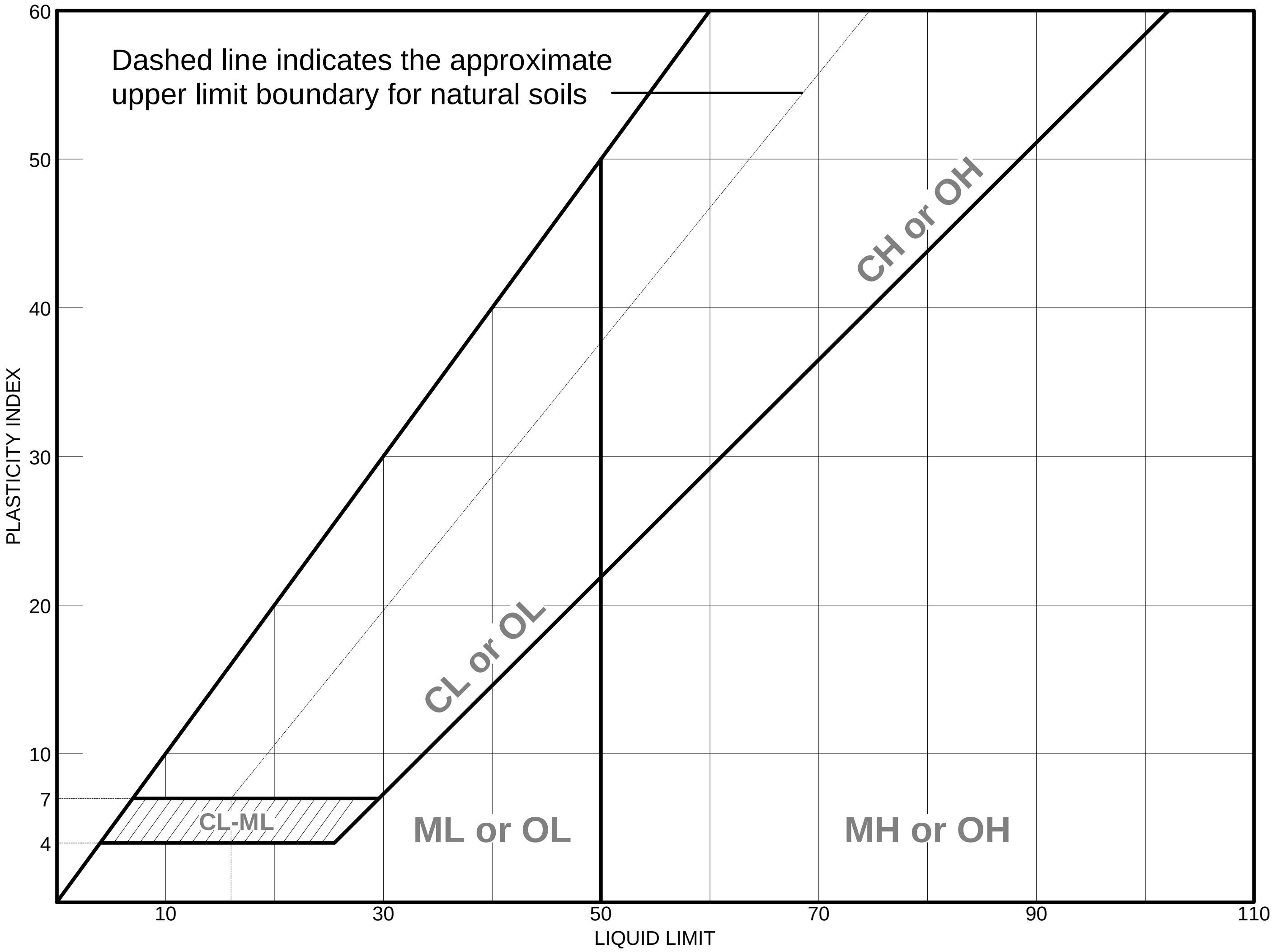
Moore Twining Associates, Inc.

Fresno, CA

Client: Redlands Summit, LLC
Project: The Neighborhoods at Lugonia Village, Redlands

Project No: H02901.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	52.6	17.0	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-7 **Elev./Depth:** 8.5-10'

Remarks:

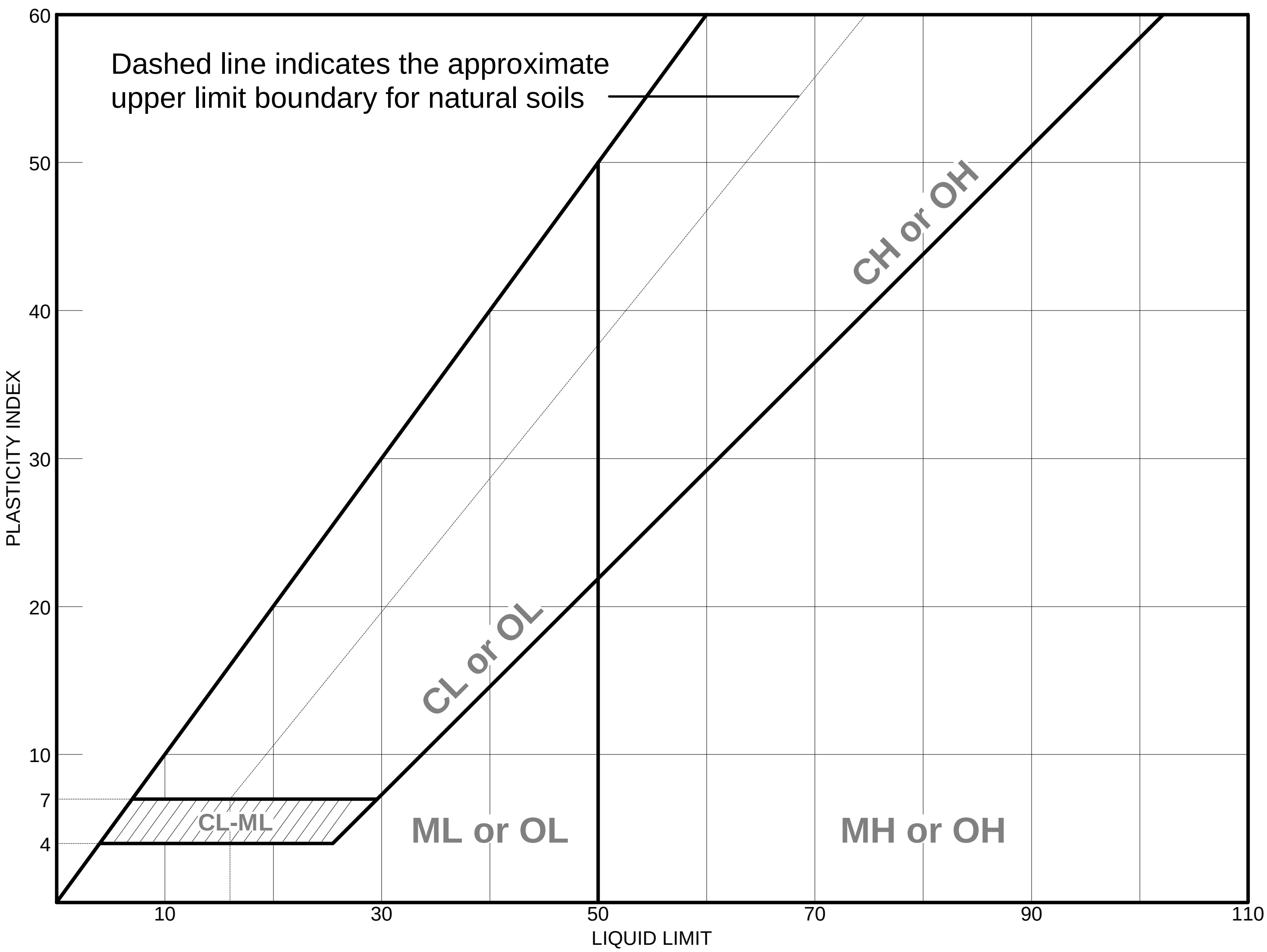
●

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	79.9	30.1	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-11 **Elev./Depth:** 3.5-5'

Remarks:

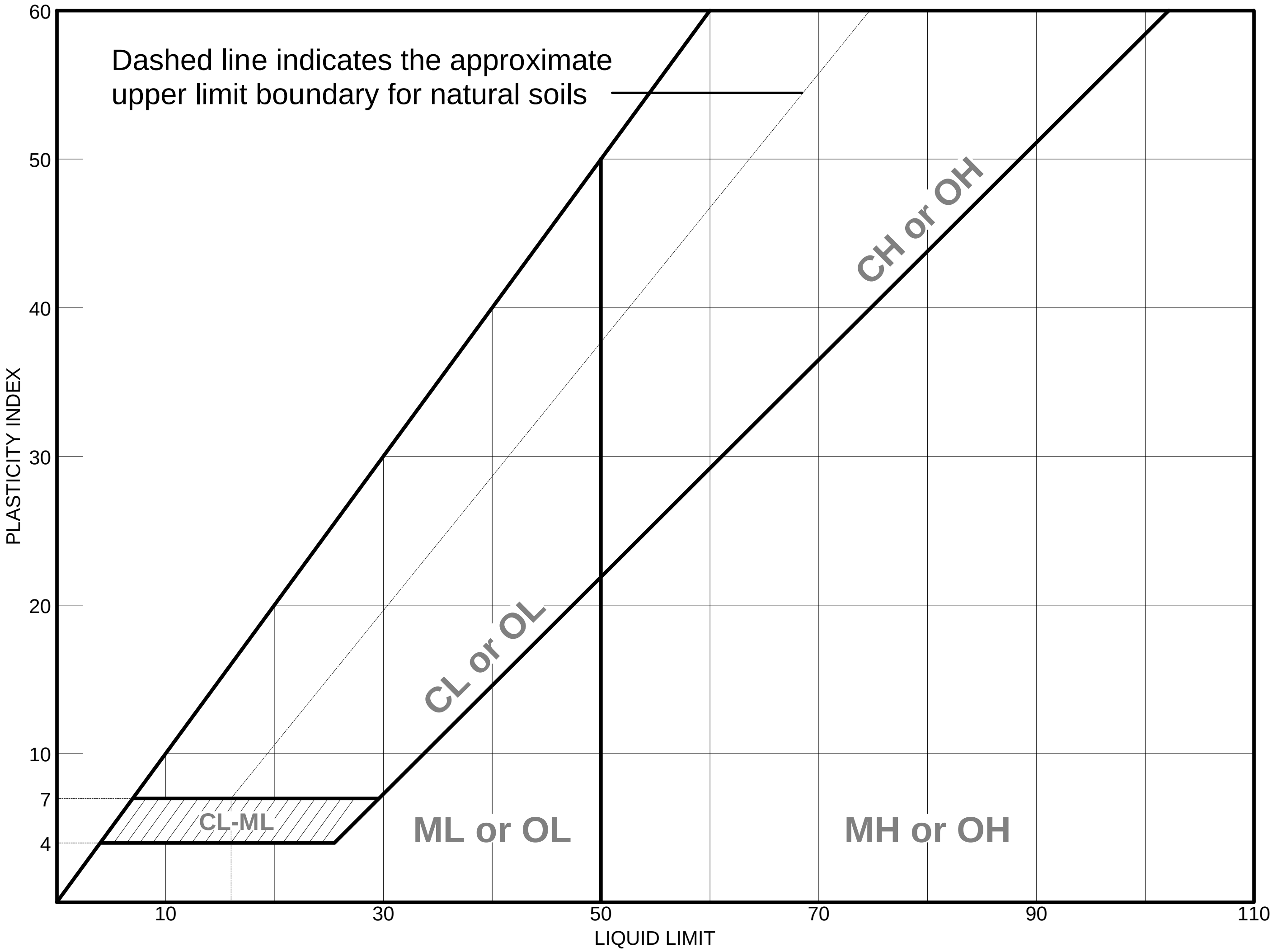
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly graded sand with silt	NV	NP	NP	52.9	11.5	SP-SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-22 **Elev./Depth:** 0-1.5'

Moore Twining Associates, Inc.

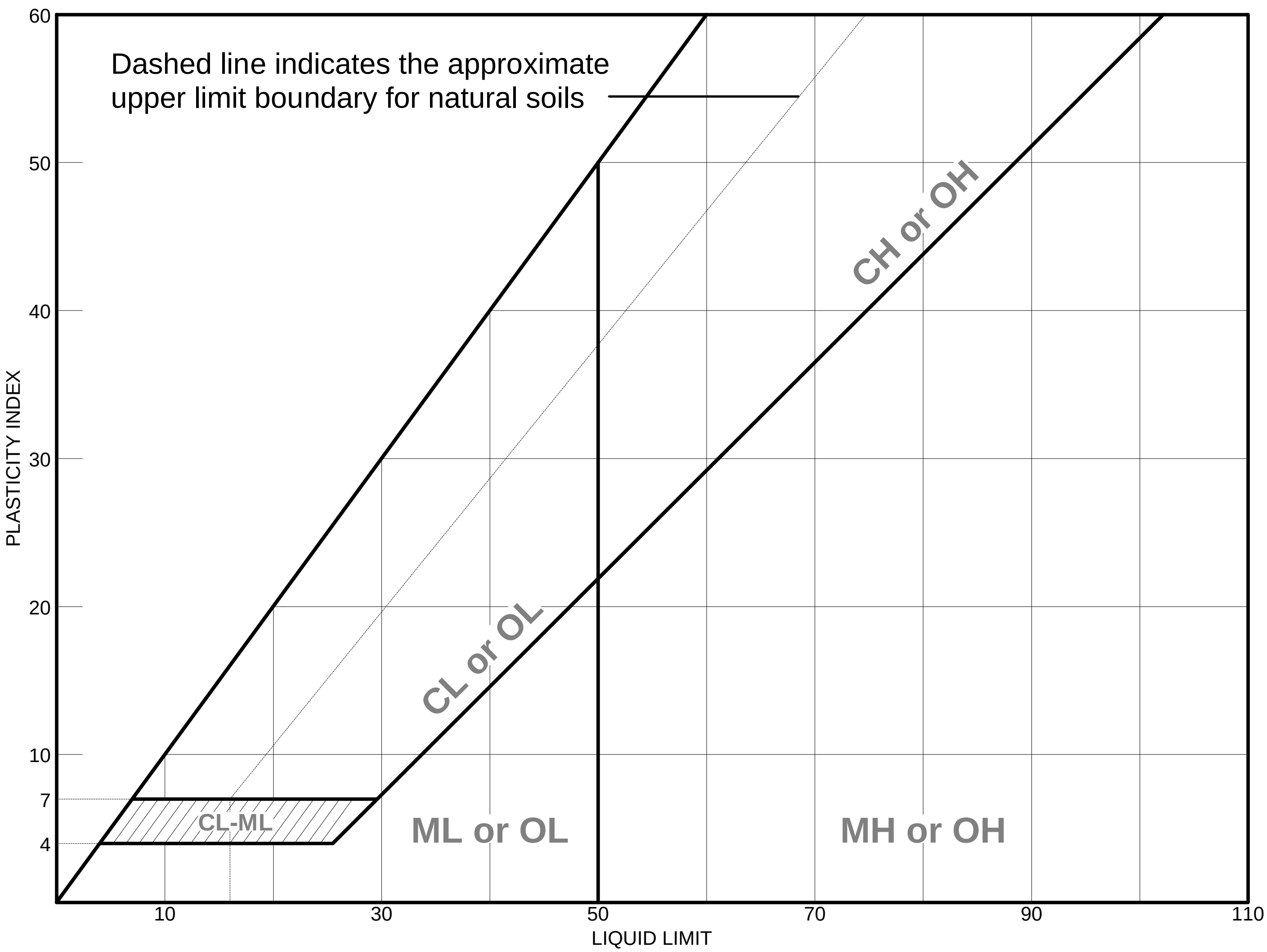
Fresno, CA

Remarks:

●

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	62.5	18.0	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-22 **Elev./Depth:** 3.5-5'

Remarks:

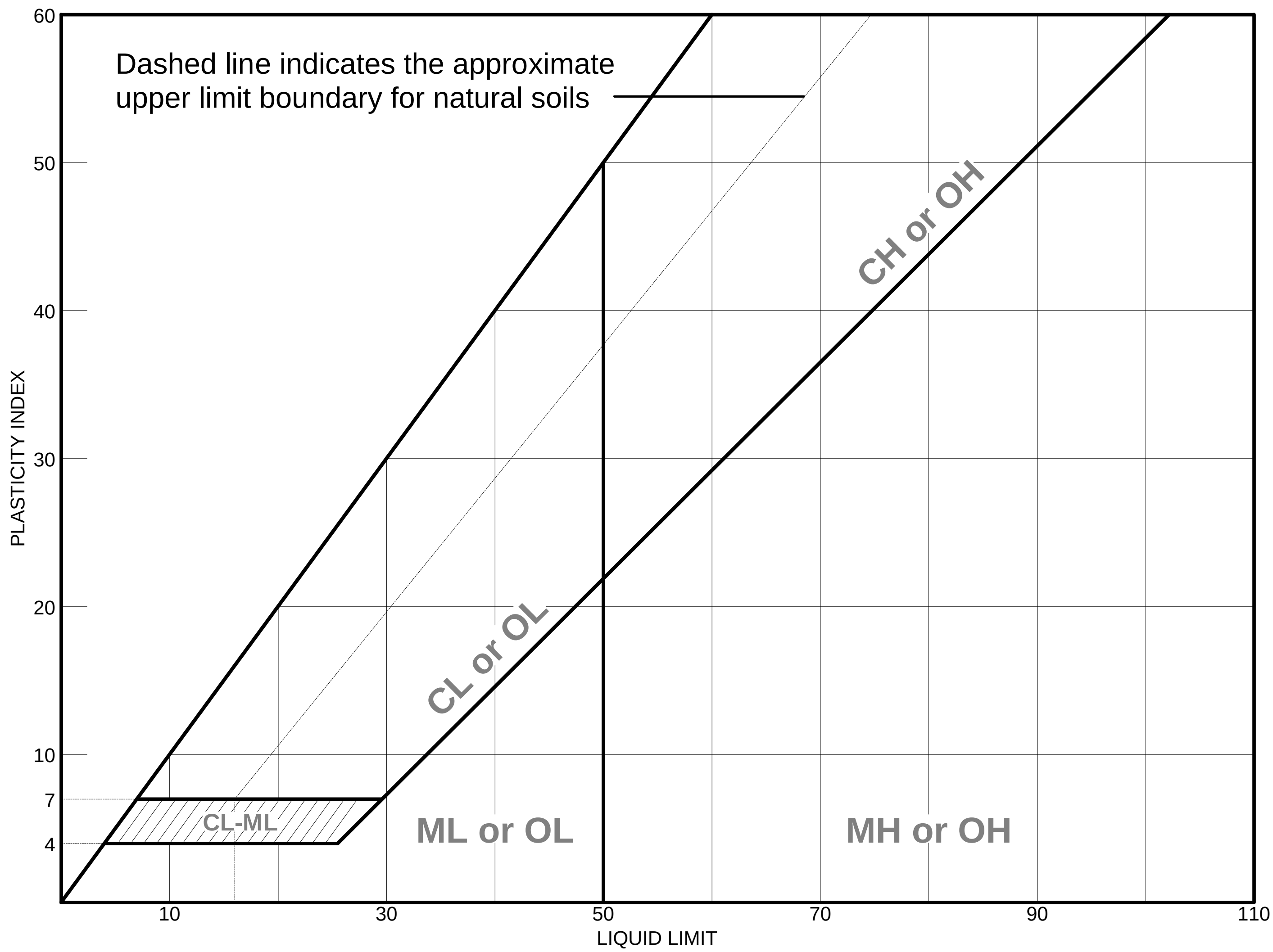
●

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	96.4	45.3	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-22 **Elev./Depth:** 23.5-25'

Remarks:

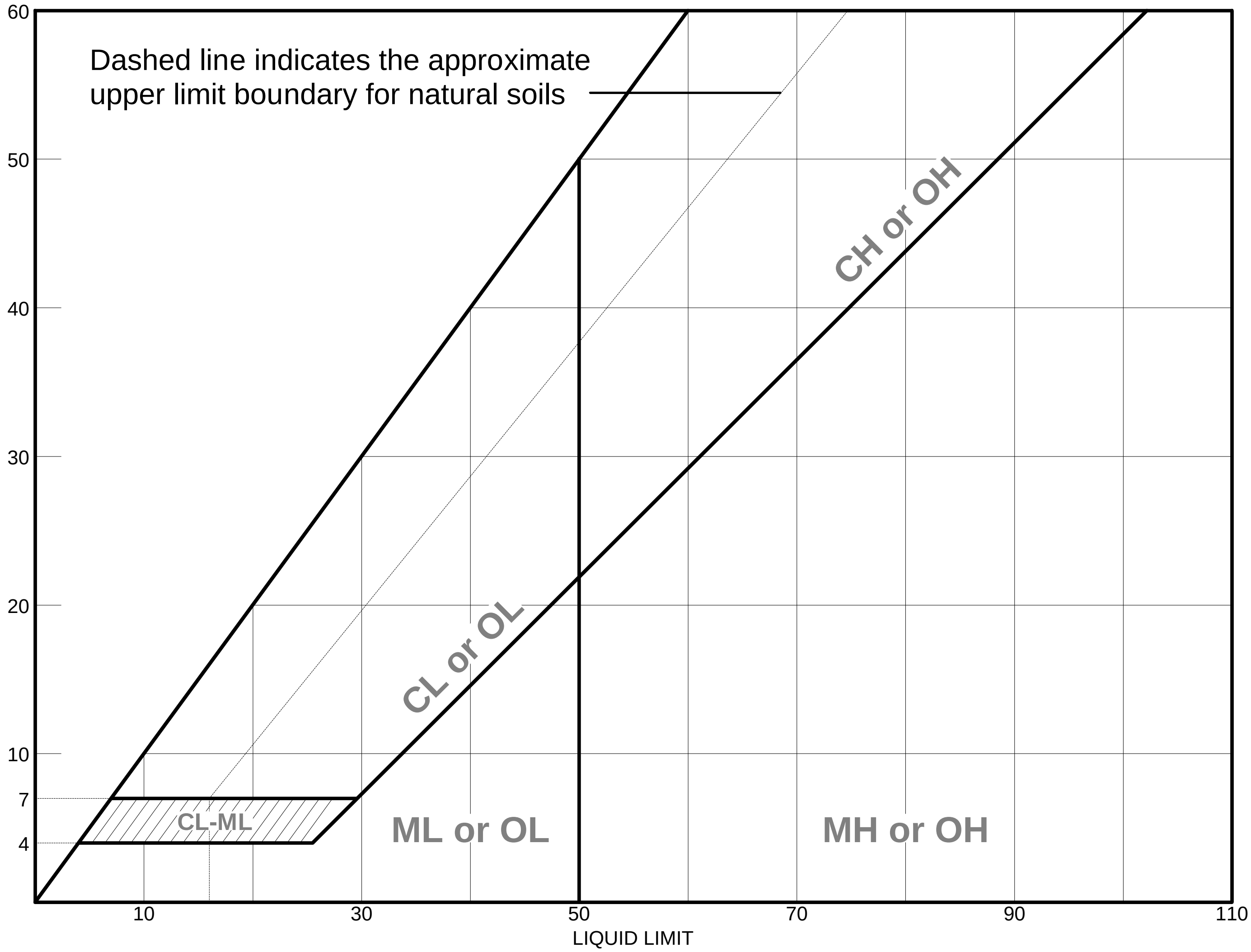
●

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	78.1	19.3	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-25 **Elev./Depth:** 1.5-3'

Remarks:

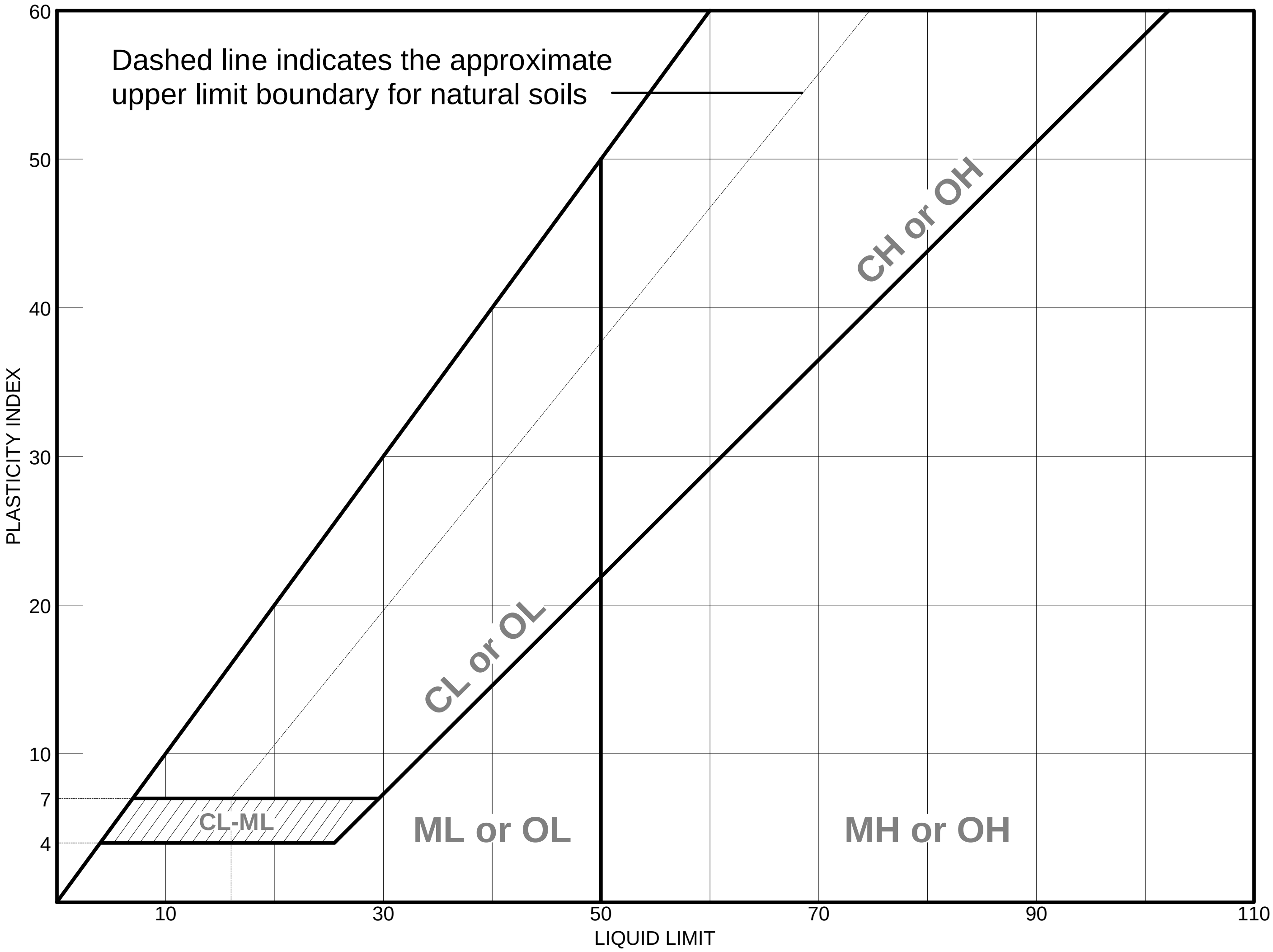
●

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	77.3	30.3	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-25 **Elev./Depth:** 5-6.5'

Remarks:

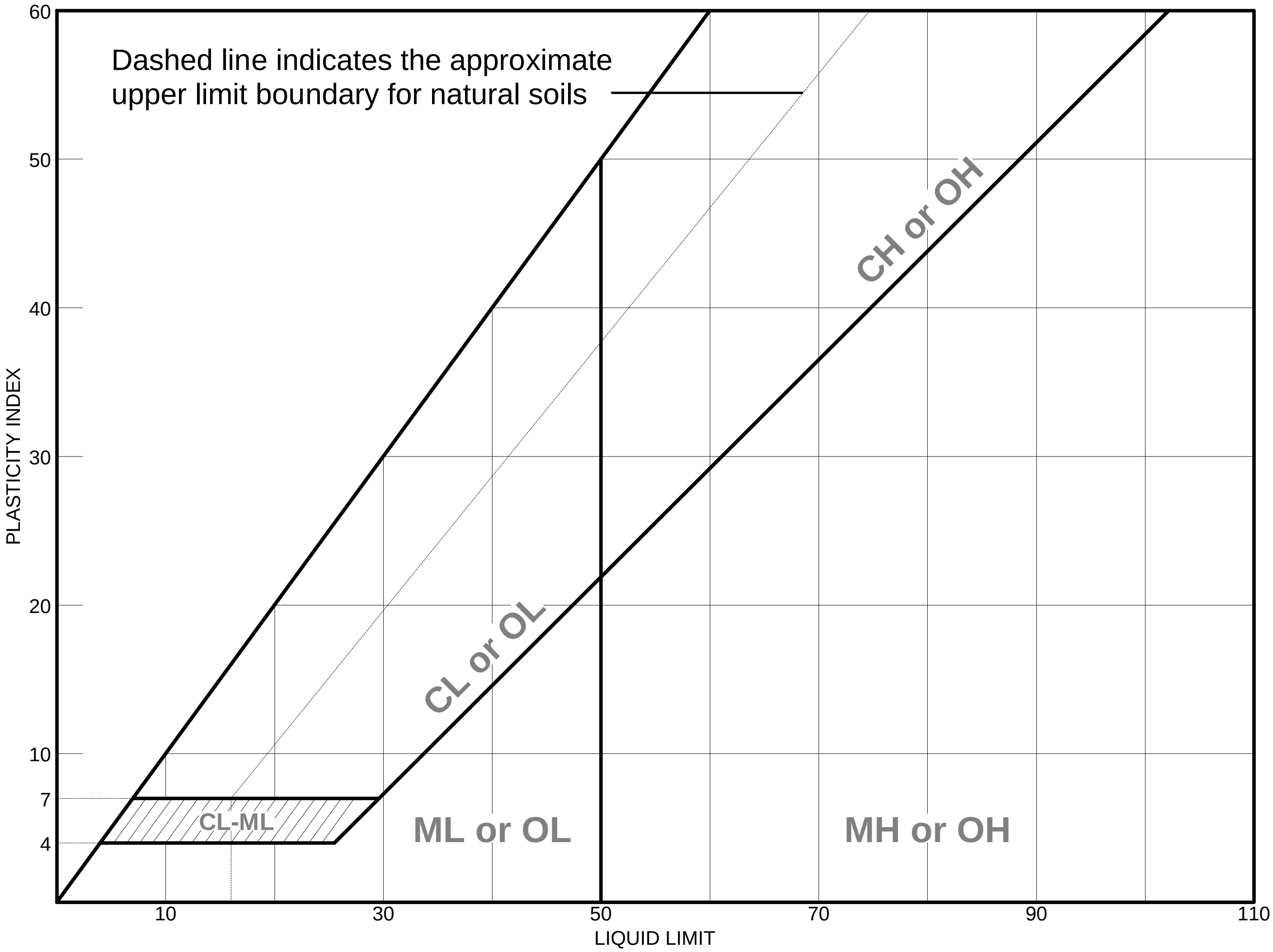
●

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	90.8	43.8	SM

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** P-1 **Elev./Depth:** 13.5-15'

Remarks:

●

Moore Twining Associates, Inc.

Fresno, CA

Figure

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	The Neighborhood at Lugonia, Redla	REPORT DATE:	12/20/2021
		TEST DATE:	12/15/2021
MTA PROJECT NO.:	H02901.01		
SAMPLE I.D.:	B-3 @ 0-3.5'		
SAMPLED BY:	YA		
SAMPLE DATE:	11/15/2021	TESTED BY:	AL
MATERIALS DESCRIPTION:	Silty sand		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9550
Pan + Dry Soil Wt., gm	227.2	Dry Soil Wt., lbs	0.7958
Pan Wt., gm	0.0		
Initial % Moisture Content	10.0	Final % Moisture Content	20.0

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8757	Ring + Sample Wt., lbs	0.9550
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8757	Remolded Wt., lbs	0.9550
Remolded Wet Density, pcf	120.4	Remolded Wet Density, pcf	131.3
Remolded Dry Density, pcf	109.4	Remolded Dry Density, pcf	109.4

Initial Volume	Final Volume
0.00727222	0.007272

Expansion Data:	
Initial Gage Reading, in:	0.0752
Final Gage Reading, in:	0.0752
Expansion, in:	0.0000
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

www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.

Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	The Neighborhood at Lugonia, Redla	REPORT DATE:	12/20/2021
		TEST DATE:	12/15/2021
MTA PROJECT NO.:	H02901.01		
SAMPLE I.D.:	B-7 @ 0-3.5'		
SAMPLED BY:	YA		
SAMPLE DATE:	11/15/2021	TESTED BY:	AL
MATERIALS DESCRIPTION:	Silty sand		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	260.0	Wet Soil Wt., lbs	0.9455
Pan + Dry Soil Wt., gm	236.4	Dry Soil Wt., lbs	0.7881
Pan Wt., gm	0.0		
Initial % Moisture Content	10.0	Final % Moisture Content	20.0

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8668	Ring + Sample Wt., lbs	0.9455
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8668	Remolded Wt., lbs	0.9455
Remolded Wet Density, pcf	119.2	Remolded Wet Density, pcf	130.0
Remolded Dry Density, pcf	108.4	Remolded Dry Density, pcf	108.4

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007272
Initial Gage Reading, in:	0.0693		
Final Gage Reading, in:	0.0693		
Expansion, in:	0.0000		
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

www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.
Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	The Neighborhood at Lugonia, Redla	REPORT DATE:	12/20/2021
		TEST DATE:	12/15/2021
MTA PROJECT NO.:	H02901.01		
SAMPLE I.D.:	B-25 @ 0-3.5'		
SAMPLED BY:	YA		
SAMPLE DATE:	11/15/2021	TESTED BY:	AL

MATERIALS DESCRIPTION: Silty sand

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	255.5	Wet Soil Wt., lbs	0.9450
Pan + Dry Soil Wt., gm	232.2	Dry Soil Wt., lbs	0.7872
Pan Wt., gm	0.0		
Initial % Moisture Content	10.0	Final % Moisture Content	20.0

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8662	Ring + Sample Wt., lbs	0.9450
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8662	Remolded Wt., lbs	0.9450
Remolded Wet Density, pcf	119.1	Remolded Wet Density, pcf	130.3
Remolded Dry Density, pcf	108.2	Remolded Dry Density, pcf	108.5

Expansion Data:

Initial Volume	Final Volume
0.00727222	0.007253

Initial Gage Reading, in:	0.0648
Final Gage Reading, in:	0.0621
Expansion, in:	-0.0027
Expansion Index	0

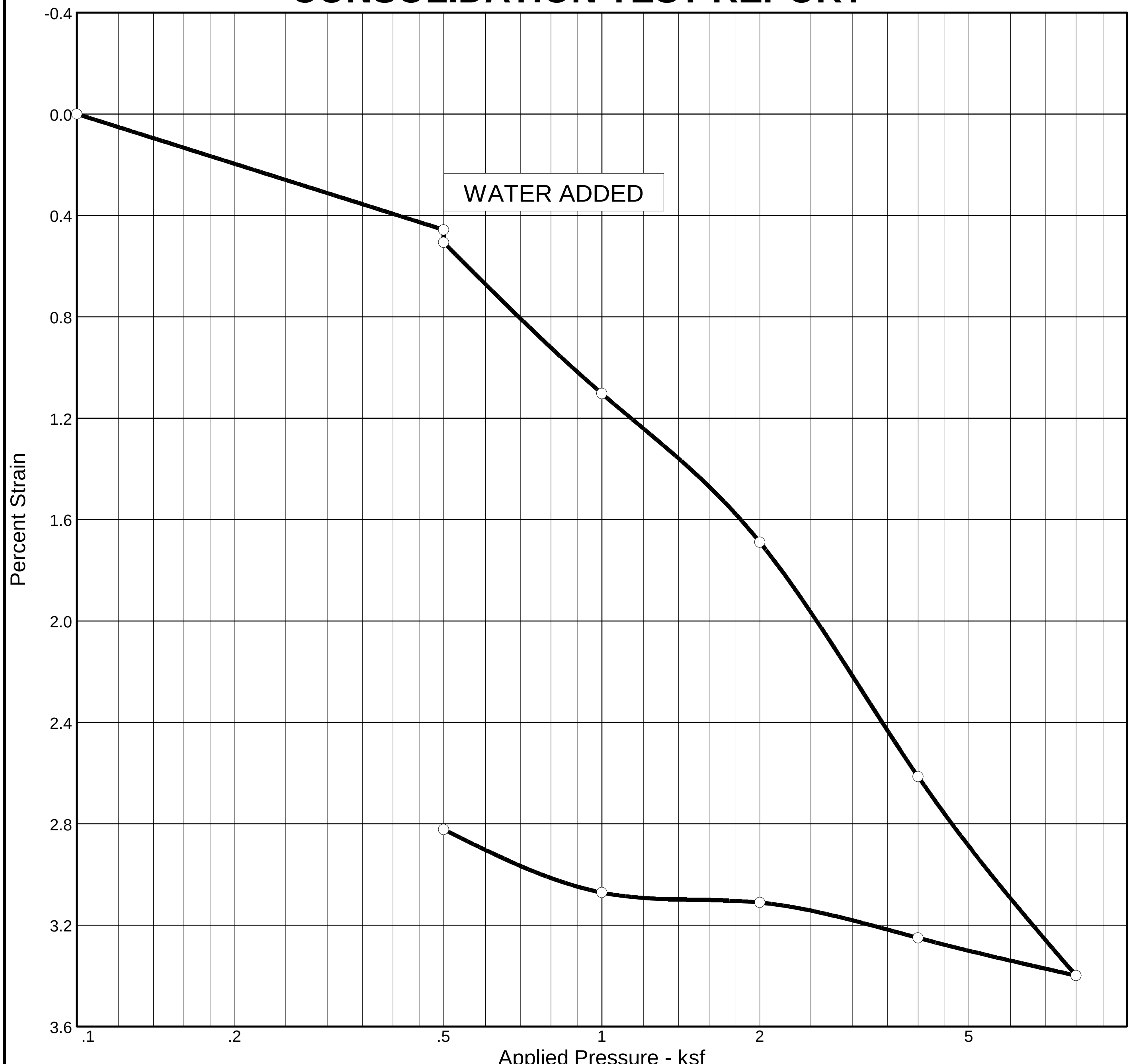
Comments: **Very Low Expansion Potential**

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)	Swell %	e_o
Sat.	Moist.											
19.6 %	3.9 %	108.5			2.65		1.43	0.04	0.01			0.525

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-1 **Elev./Depth:** 3.5-5'

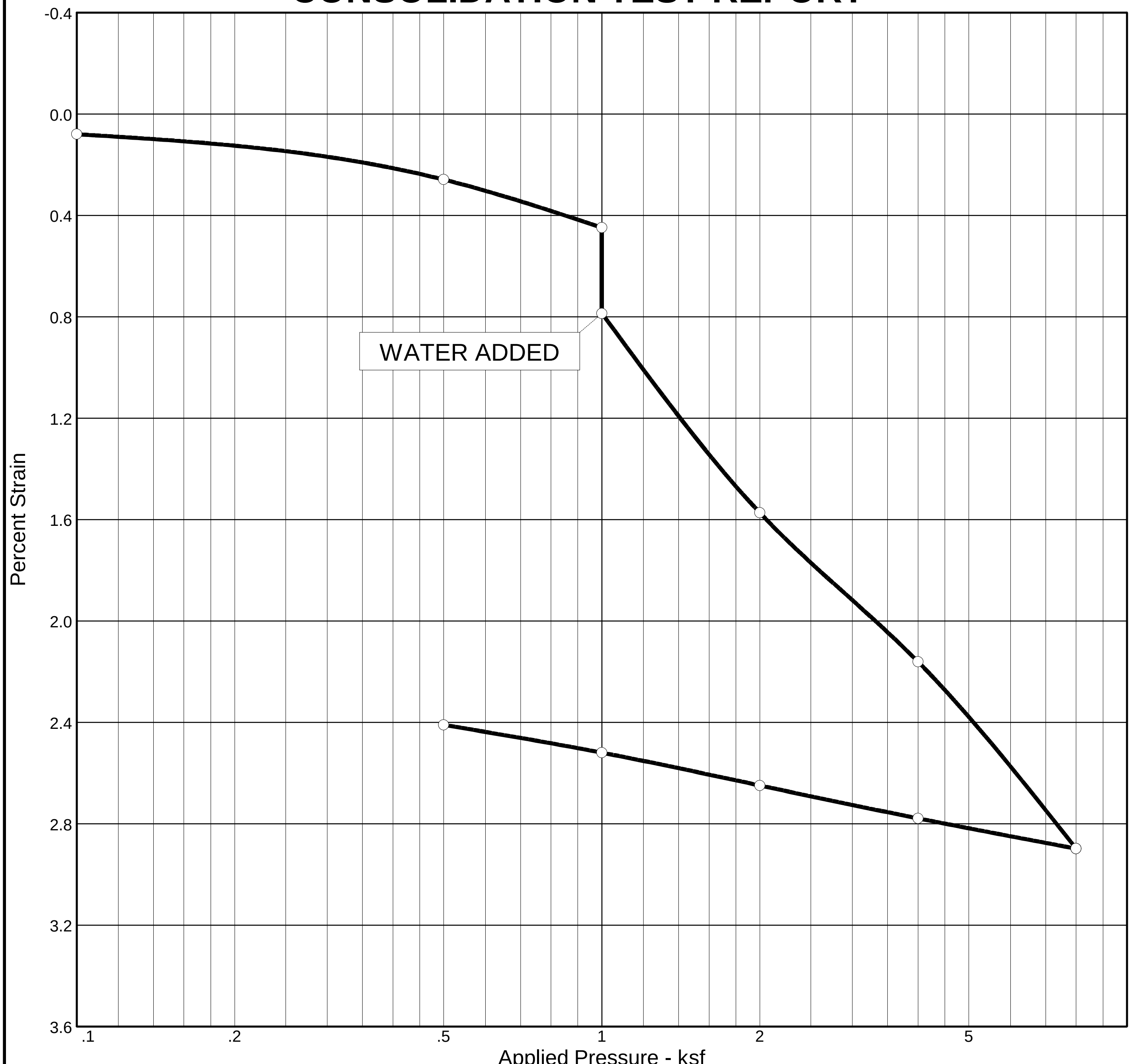
Moore Twining Associates, Inc.

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.											
18.9 %	4.1 %	104.9			2.65		2.73	0.04	0.01		0.4	0.576

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-5 **Elev./Depth:** 5-6.5'

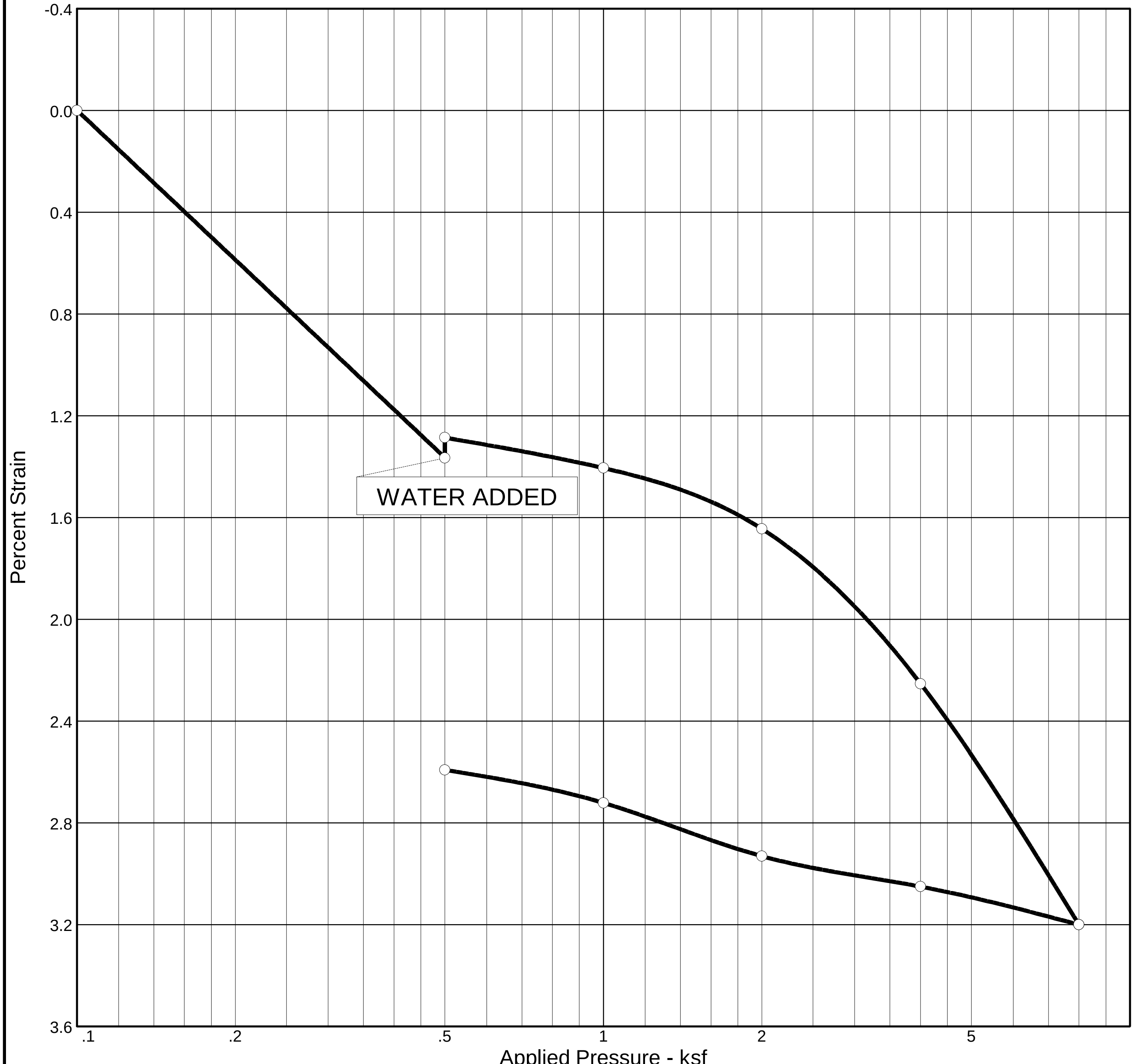
Moore Twining Associates, Inc.

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.											
36.8 %	6.2 %	114.2			2.65		2.84	0.05	0.01	0.81	0.1	0.449

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-7 **Elev./Depth:** 3.5-5'

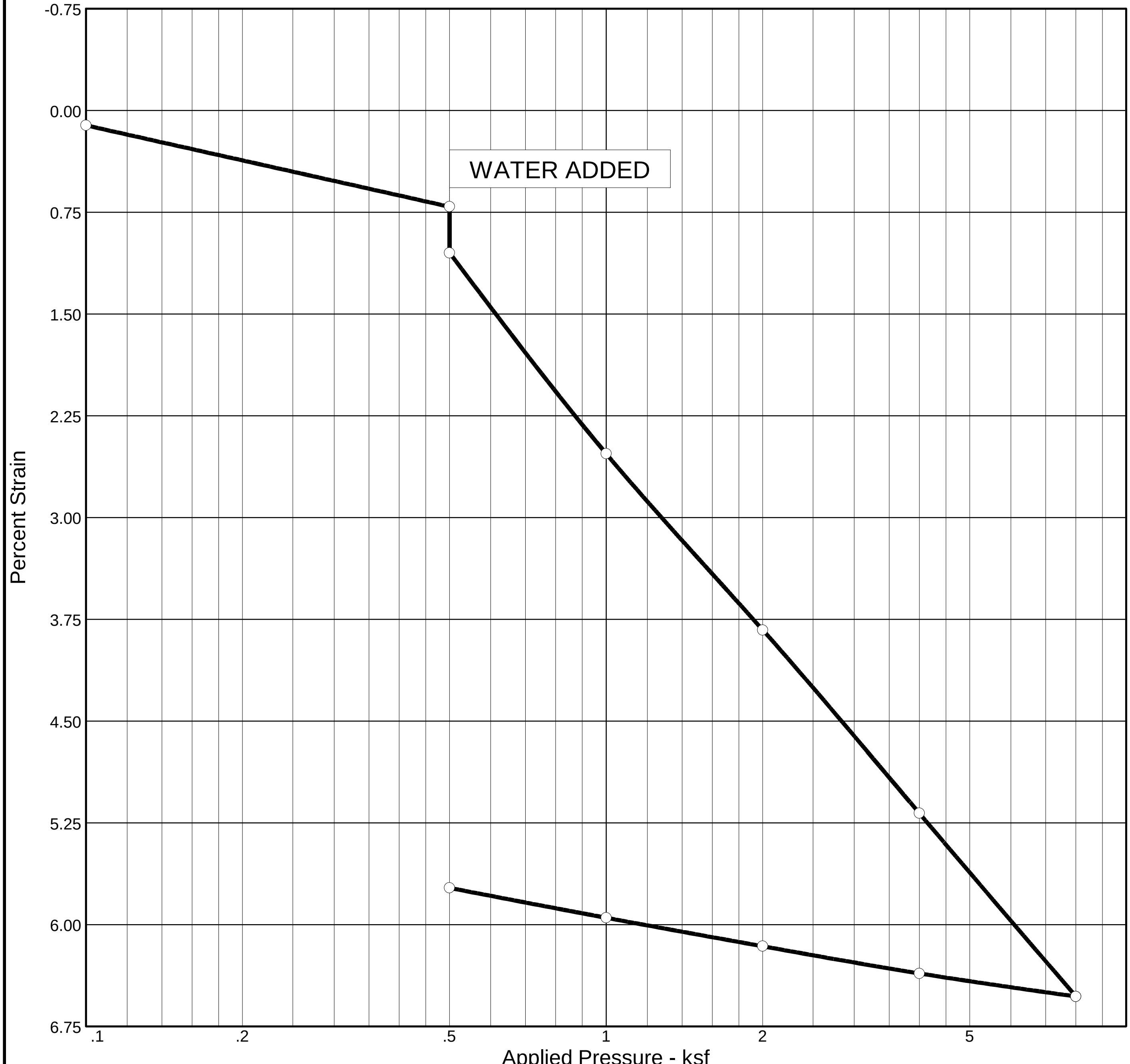
Moore Twining Associates, Inc.

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.											
13.5 %	2.9 %	105.0			2.65		1.05	0.07	0.01		0.4	0.576

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-14 **Elev./Depth:** 2-3.5'

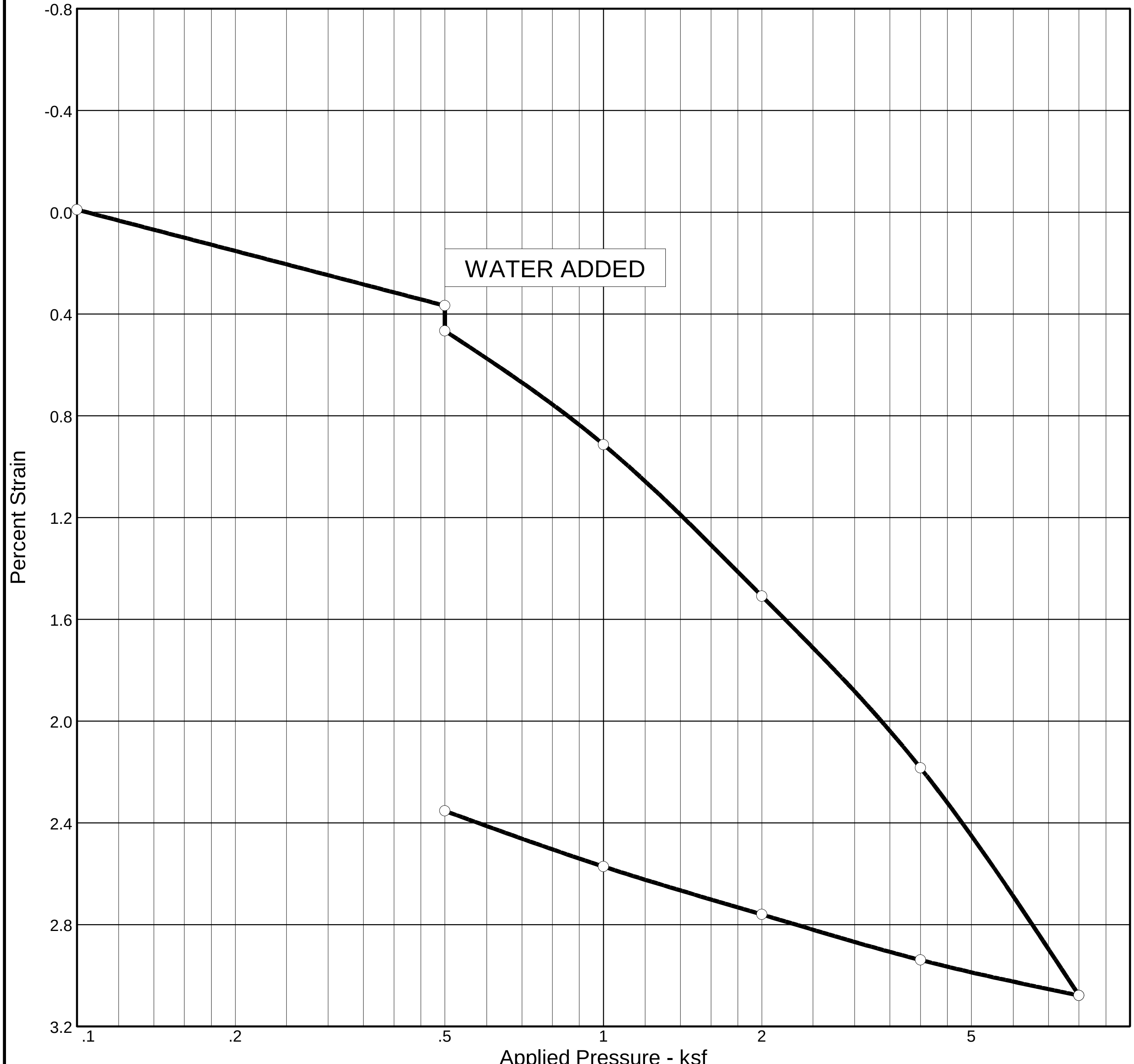
Moore Twining Associates, Inc.

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.											
25.8 %	5.3 %	107.0			2.65		1.85	0.05	0.01		0.1	0.546

MATERIAL DESCRIPTION	USCS	AASHTO
Silty Sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-16 **Elev./Depth:** 3.5-5'

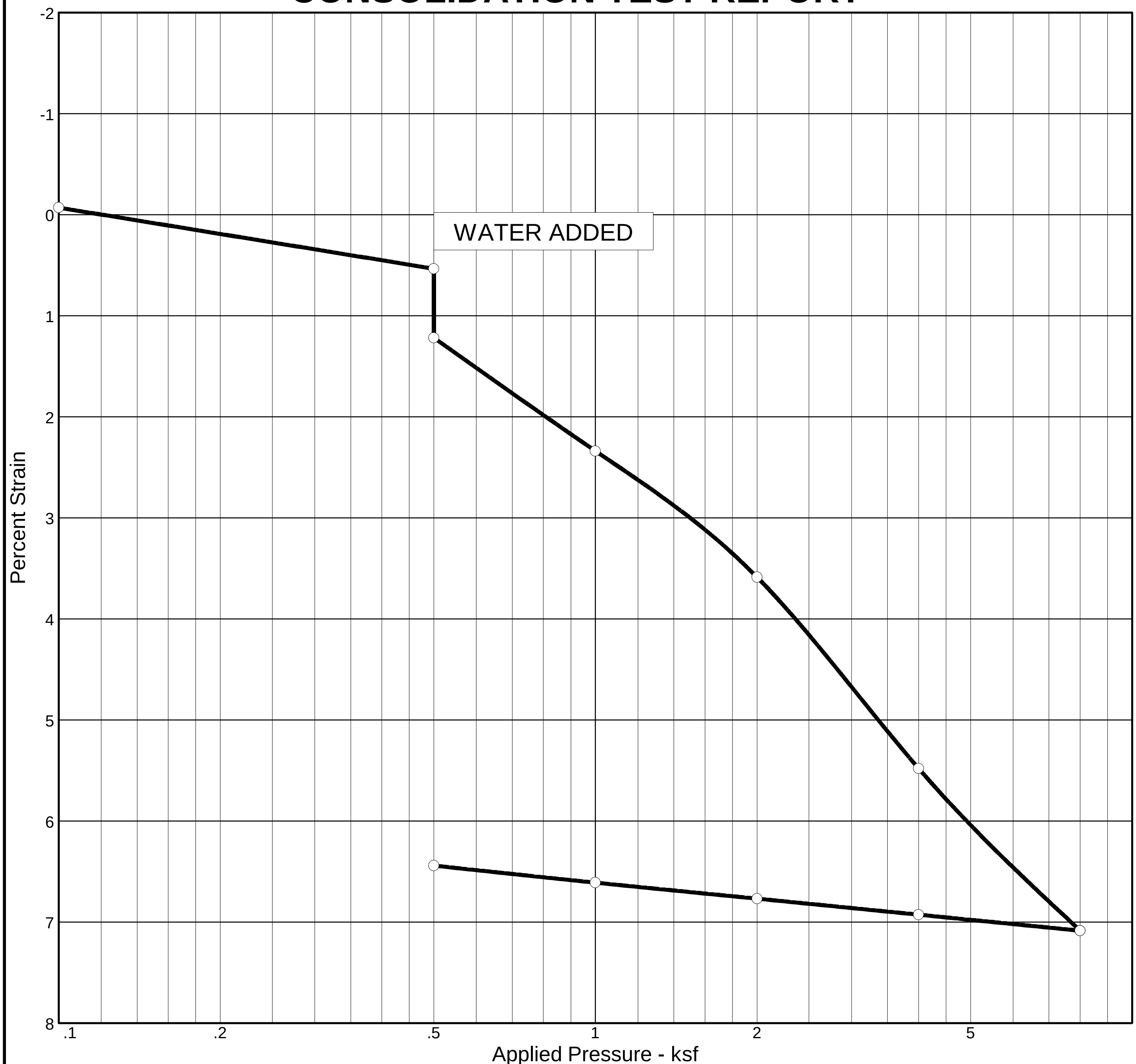
Moore Twining Associates, Inc.

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.											
10.4 %	2.3 %	104.8			2.65		1.42	0.08	0.01		0.7	0.578

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-21 **Elev./Depth:** 2-3.5'

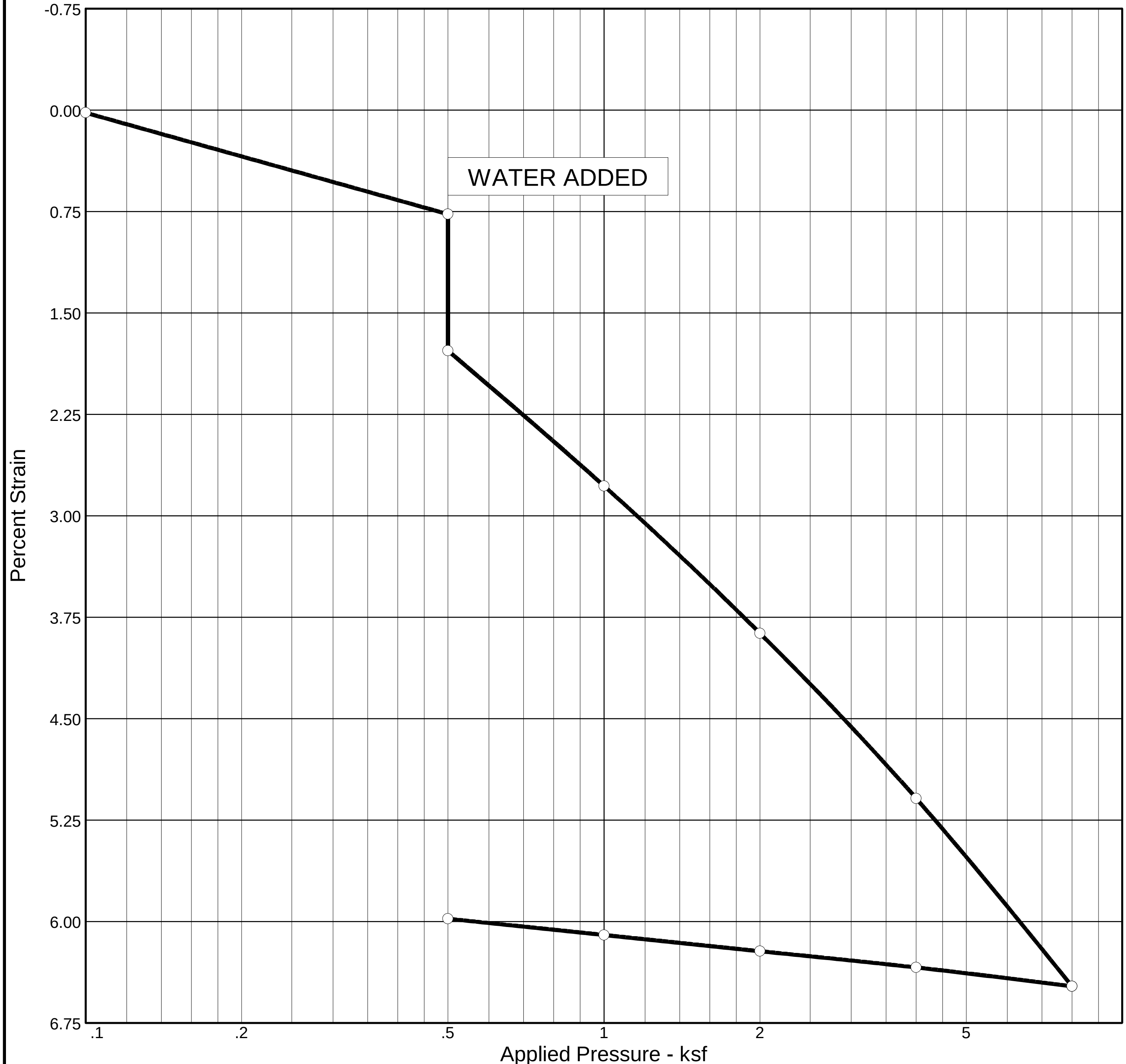
Moore Twining Associates, Inc.

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.											
14.5 %	2.8 %	109.4	NV	NP	2.65		4.07	0.07	0.01		1.0	0.513

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-22 **Elev./Depth:** 3.5-5'

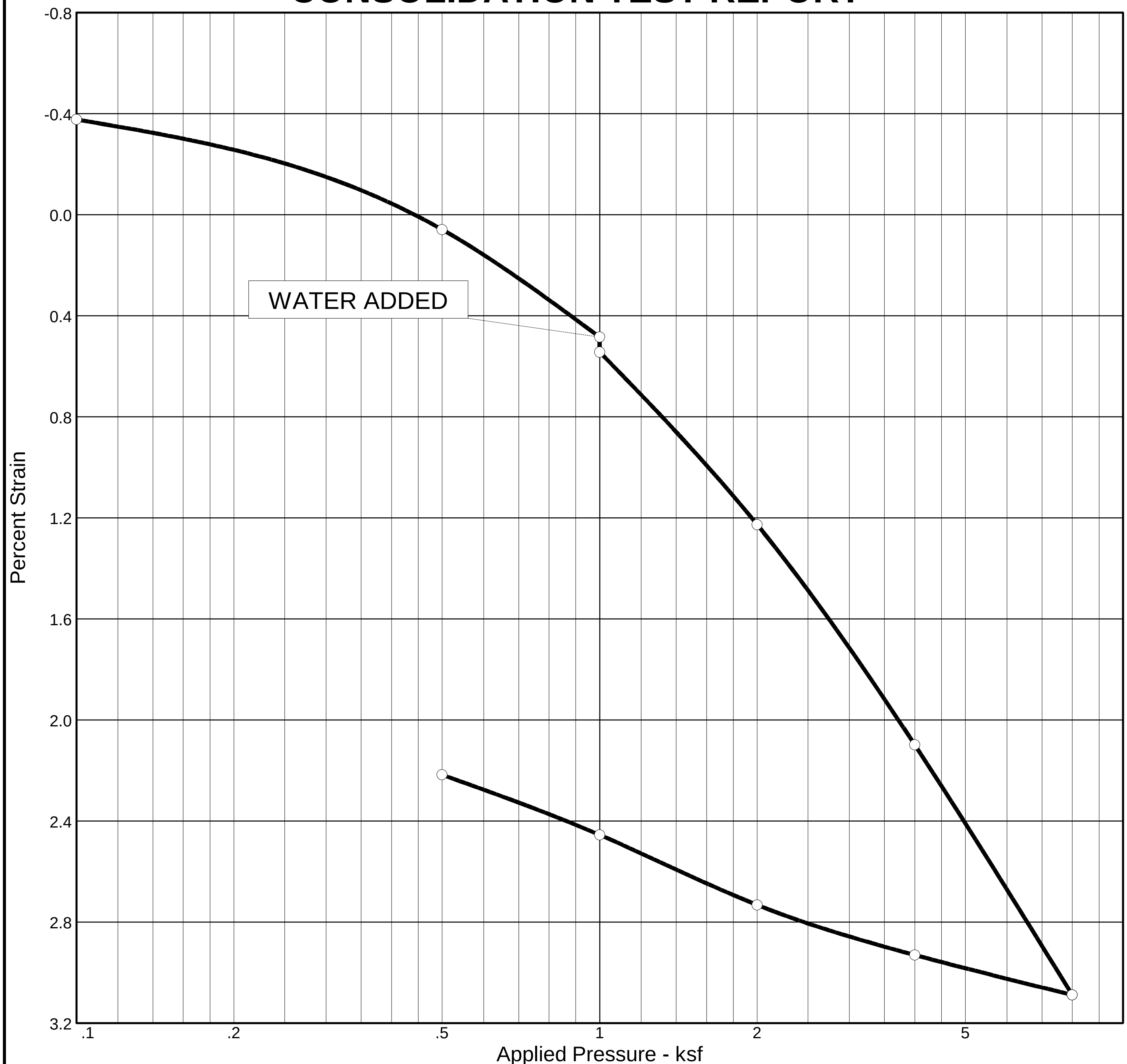
Moore Twining Associates, Inc.

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.											
20.2 %	3.9 %	109.4	NV	NP	2.65		1.12	0.05	0.01			0.512

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Source: **Sample No.:** B-25 **Elev./Depth:** 5-6.5'

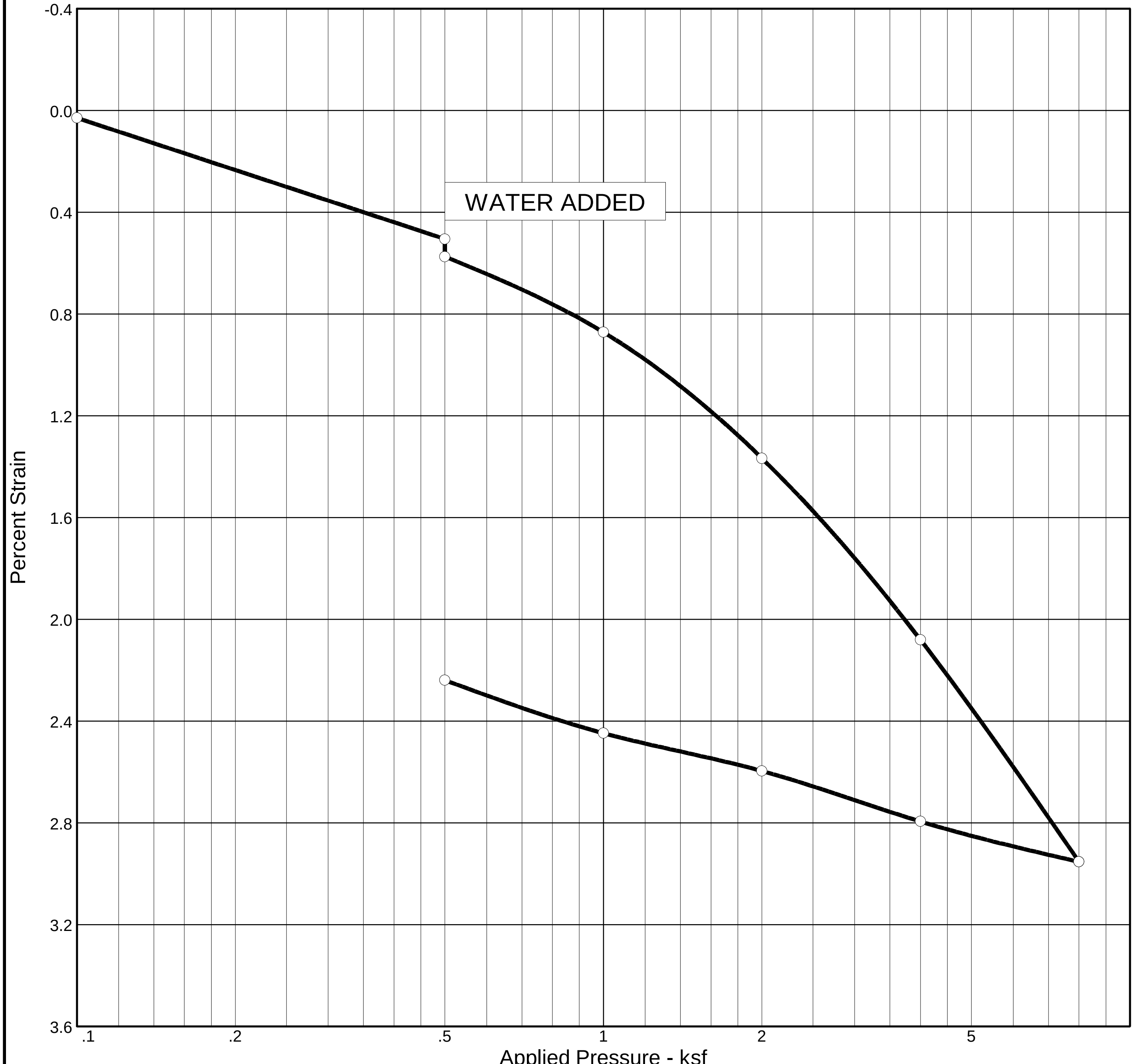
Moore Twining Associates, Inc.

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.											
42.3 %	7.9 %	110.7			2.65		1.76	0.04	0.01		0.1	0.495

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

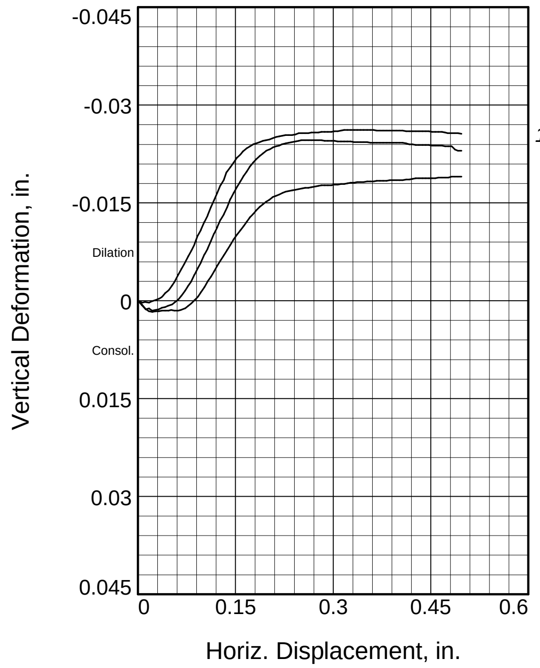
Source: **Sample No.:** B-30 **Elev./Depth:** 3.5-5'

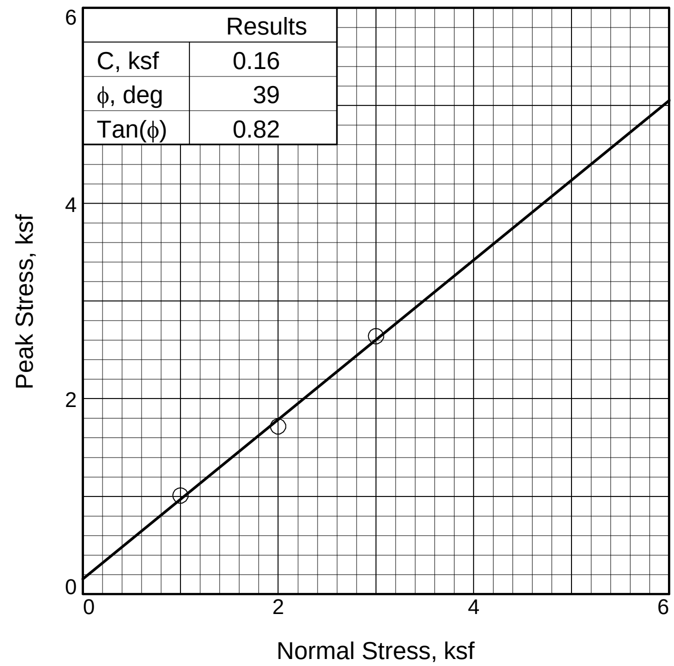
Remarks:

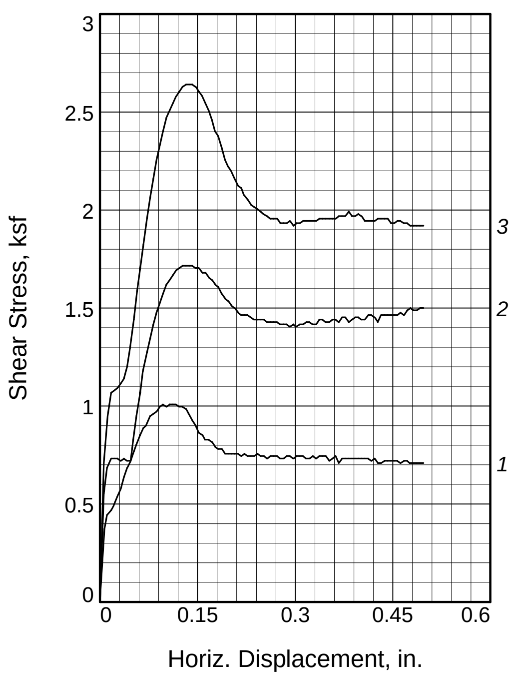
Moore Twining Associates, Inc.

Fresno, CA

Figure







Sample No.		1	2	3
Initial	Water Content, %	4.1	4.0	4.8
	Dry Density, pcf	105.0	107.4	109.6
	Saturation, %	18.9	19.7	24.7
	Void Ratio	0.5763	0.5409	0.5101
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	20.3	18.6	17.7
	Dry Density, pcf	106.3	110.0	112.5
	Saturation, %	96.5	97.6	99.9
	Void Ratio	0.5566	0.5045	0.4704
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.98	0.97
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		1.01	1.72	2.64
Displacement, in.		0.10	0.13	0.13
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		4.0	5.3	5.5

Sample Type:

Description: Silty sand

Specific Gravity= 2.65

Remarks:

Figure ________

Client: Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

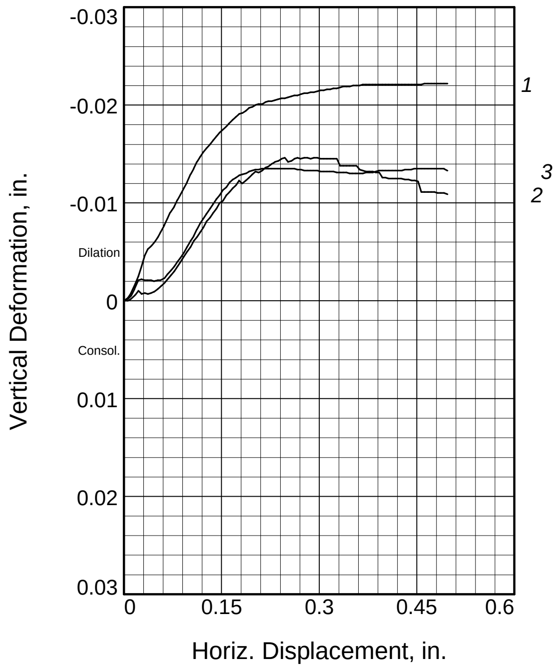
Sample Number: B-5 **Depth:** 5-6.5'

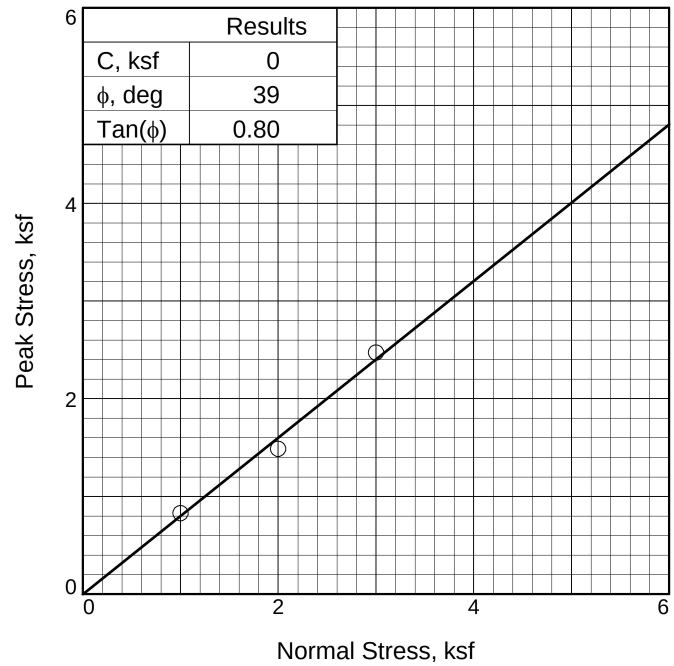
Proj. No.: H02901.01 **Date Sampled:** 11/15/21

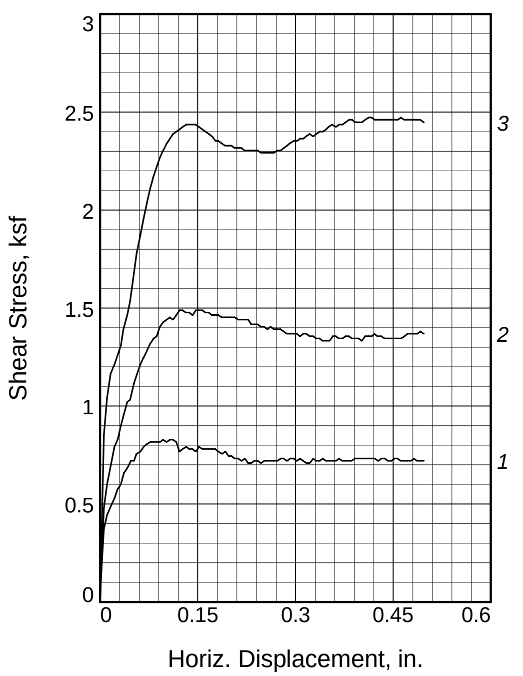
DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Fresno, CA







Sample No.		1	2	3
Initial	Water Content, %	4.2	4.2	3.8
	Dry Density, pcf	105.6	106.0	110.1
	Saturation, %	19.5	19.6	20.3
	Void Ratio	0.5664	0.5602	0.5025
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	19.7	18.3	16.5
	Dry Density, pcf	107.4	110.1	113.7
	Saturation, %	96.4	96.3	95.8
	Void Ratio	0.5407	0.5025	0.4550
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.96	0.97
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		0.83	1.49	2.47
Displacement, in.		0.10	0.12	0.41
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		4.0	5.0	17.1

Sample Type:
Description: Silty sand

Specific Gravity= 2.65
Remarks:

Figure ________

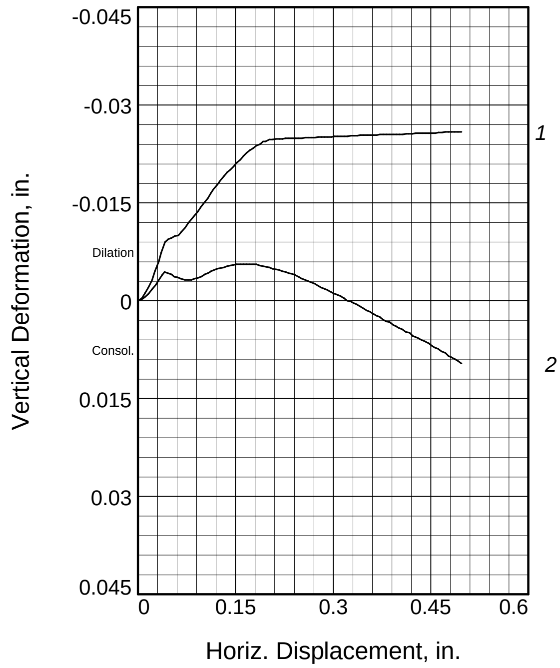
Client: Redlands Summit, LLC

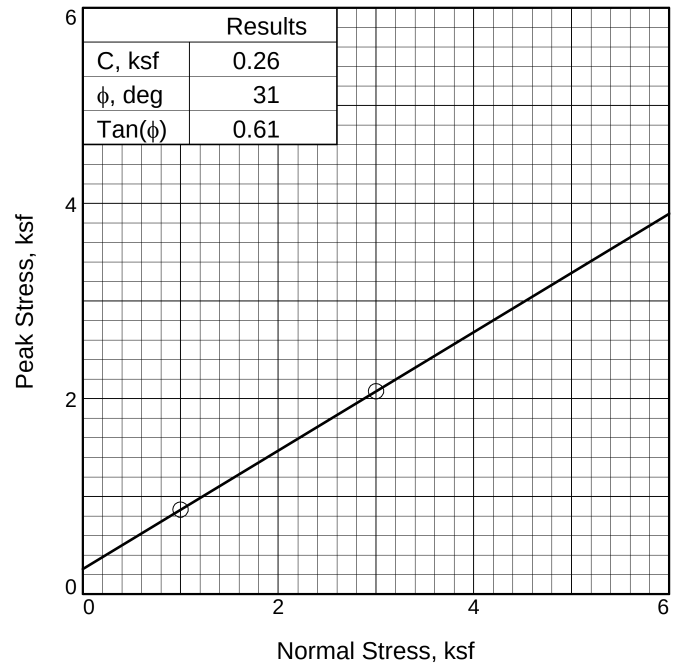
Project: The Neighborhoods at Lugonia Village, Redlands

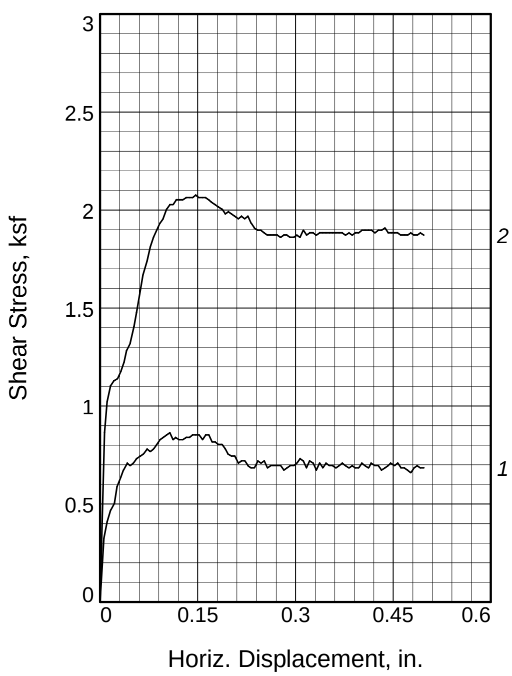
Sample Number: B-14 **Depth:** 2-3.5'

Proj. No.: H02901.01 **Date Sampled:** 11/15/21

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







	Sample No.	1	2
Initial	Water Content, %	4.5	3.4
	Dry Density, pcf	109.7	108.8
	Saturation, %	23.3	17.5
	Void Ratio	0.5082	0.5209
	Diameter, in.	2.42	2.42
	Height, in.	1.00	1.00
At Test	Water Content, %	18.3	17.3
	Dry Density, pcf	110.9	112.3
	Saturation, %	98.8	96.7
	Void Ratio	0.4919	0.4730
	Diameter, in.	2.42	2.42
	Height, in.	0.99	0.97
	Normal Stress, ksf	1.00	3.00
	Peak Stress, ksf	0.86	2.08
	Displacement, in.	0.11	0.15
	Ultimate Stress, ksf		
	Displacement, in.		
	Strain at peak, %	4.4	6.1

Sample Type:
Description: Silty sand

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

Figure ________

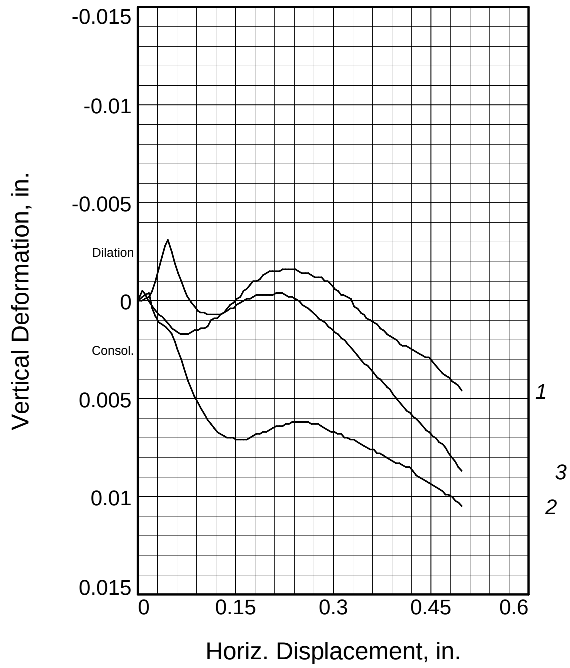
Client: Redlands Summit, LLC

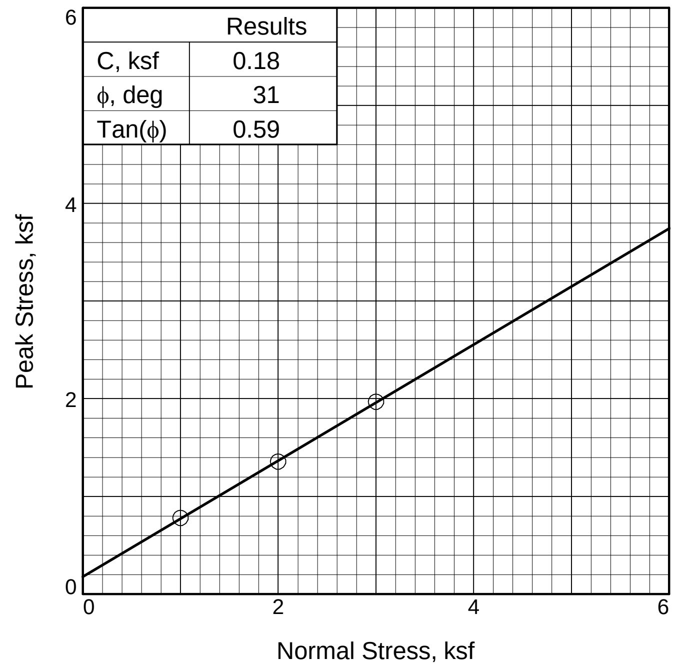
Project: The Neighborhoods at Lugonia Village, Redlands

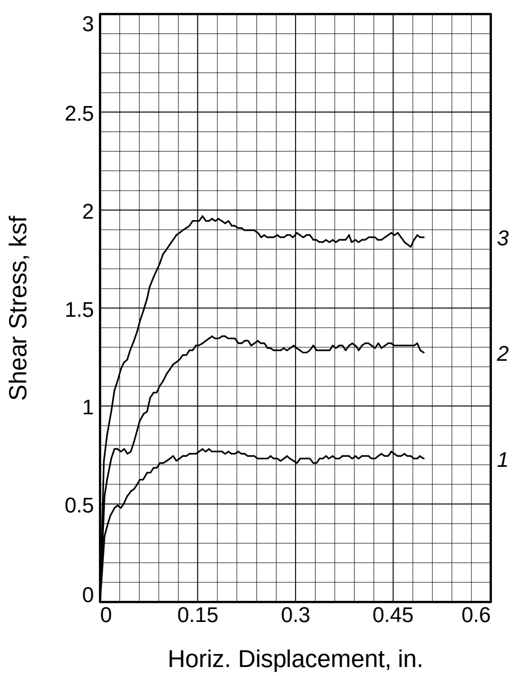
Sample Number: B-22 **Depth:** 3.5-5'

Proj. No.: H02901.01 **Date Sampled:** 11/15/21

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







	Sample No.	1	2	3
Initial	Water Content, %	3.2	2.3	3.0
	Dry Density, pcf	109.2	110.1	106.8
	Saturation, %	16.4	12.3	14.3
	Void Ratio	0.5151	0.5025	0.5493
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.1	17.9	18.8
	Dry Density, pcf	111.2	112.2	109.7
	Saturation, %	98.3	99.9	97.7
	Void Ratio	0.4883	0.4743	0.5085
	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	0.78	1.36	1.97
	Displacement, in.	0.16	0.17	0.16
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	6.5	7.1	6.5

Sample Type:

Description: Silty sand

Specific Gravity= 2.65

Remarks:

Figure ________

Client: Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

Sample Number: B-24 **Depth:** 2-3.5'

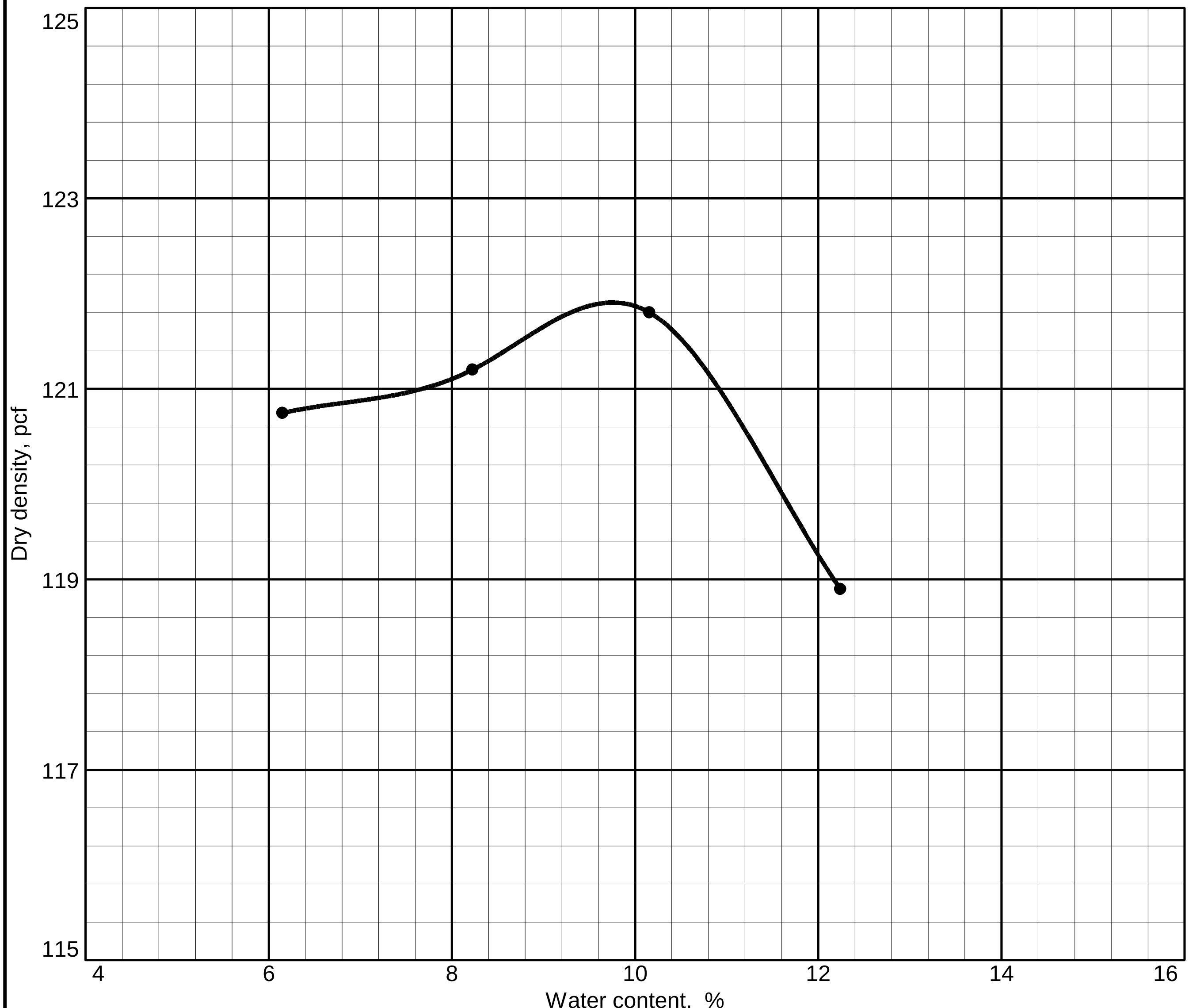
Proj. No.: H02901.01 **Date Sampled:** 11/15/21

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						
0-3.5'	SM							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 121.9 pcf Optimum moisture = 9.8 %	Silty sand

Project No. H02901.01 **Client:** Redlands Summit, LLC

Project: The Neighborhoods at Lugonia Village, Redlands

● **Source:** **Sample No.:** B-10 **Elev./Depth:** 0-3.5'

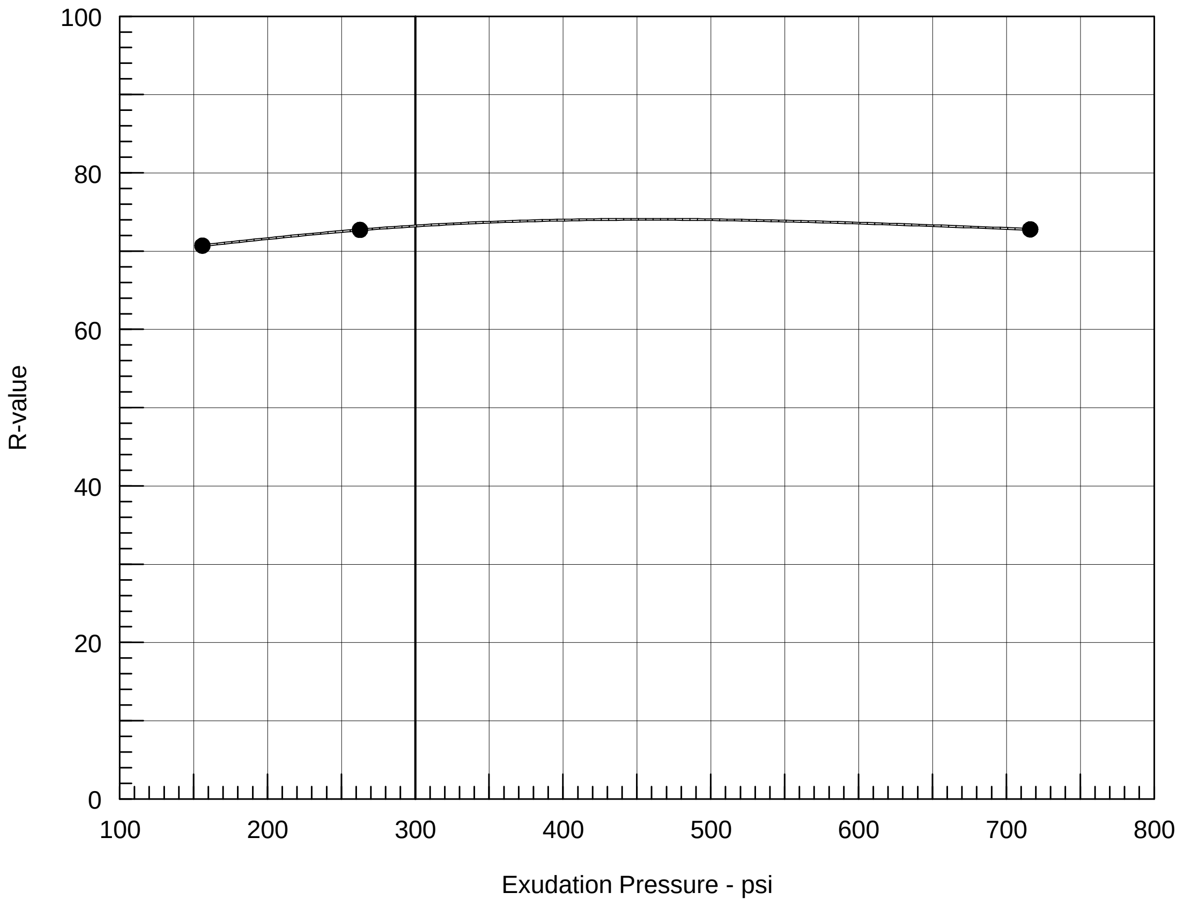
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

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	350	121.0	9.5	0.00	21	2.44	263	74	73
2	350	122.0	9.0	0.00	20	2.42	716	74	73
3	350	121.4	10.0	0.00	22	2.43	156	72	71

Test Results	Material Description
R-value at 300 psi exudation pressure = 73	Silty sand

Project No.: H02901.01

Project: The Neighborhoods at Lugonia Village, Redlands

Sample Number: B-1 **Depth:** 0-3.5'

Date: 12/20/2021

Tested by: MP

Checked by: MS

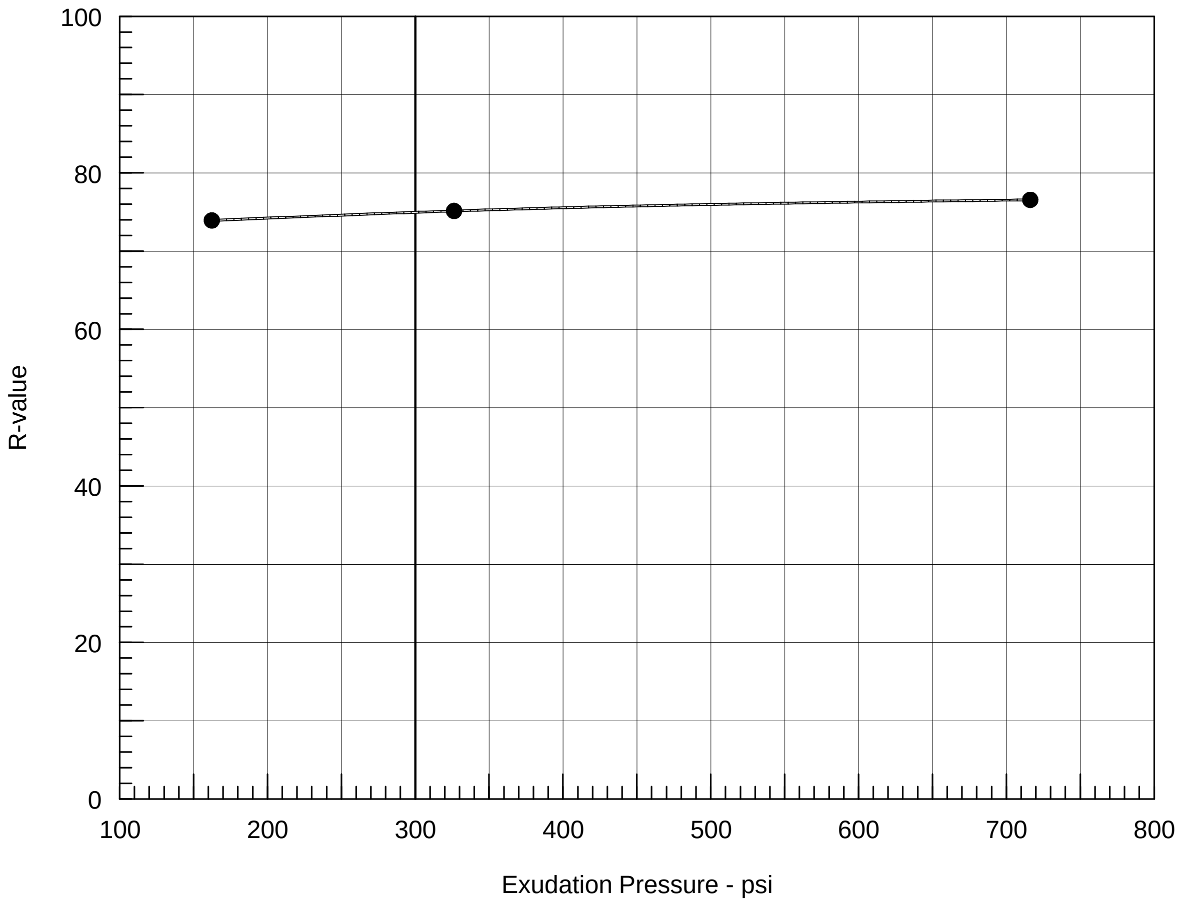
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

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	350	124.0	9.1	0.00	19	2.32	326	78	75
2	350	122.5	9.7	0.00	20	2.34	162	76	74
3	350	123.2	8.6	0.00	18	2.33	716	79	77

Test Results	Material Description
R-value at 300 psi exudation pressure = 75	Silty sand

Project No.: H02901.01

Project: The Neighborhoods at Lugonia Village, Redlands

Sample Number: B-14 **Depth:** 0-3.5'

Date: 12/20/2021

Tested by: MP

Checked by: MS

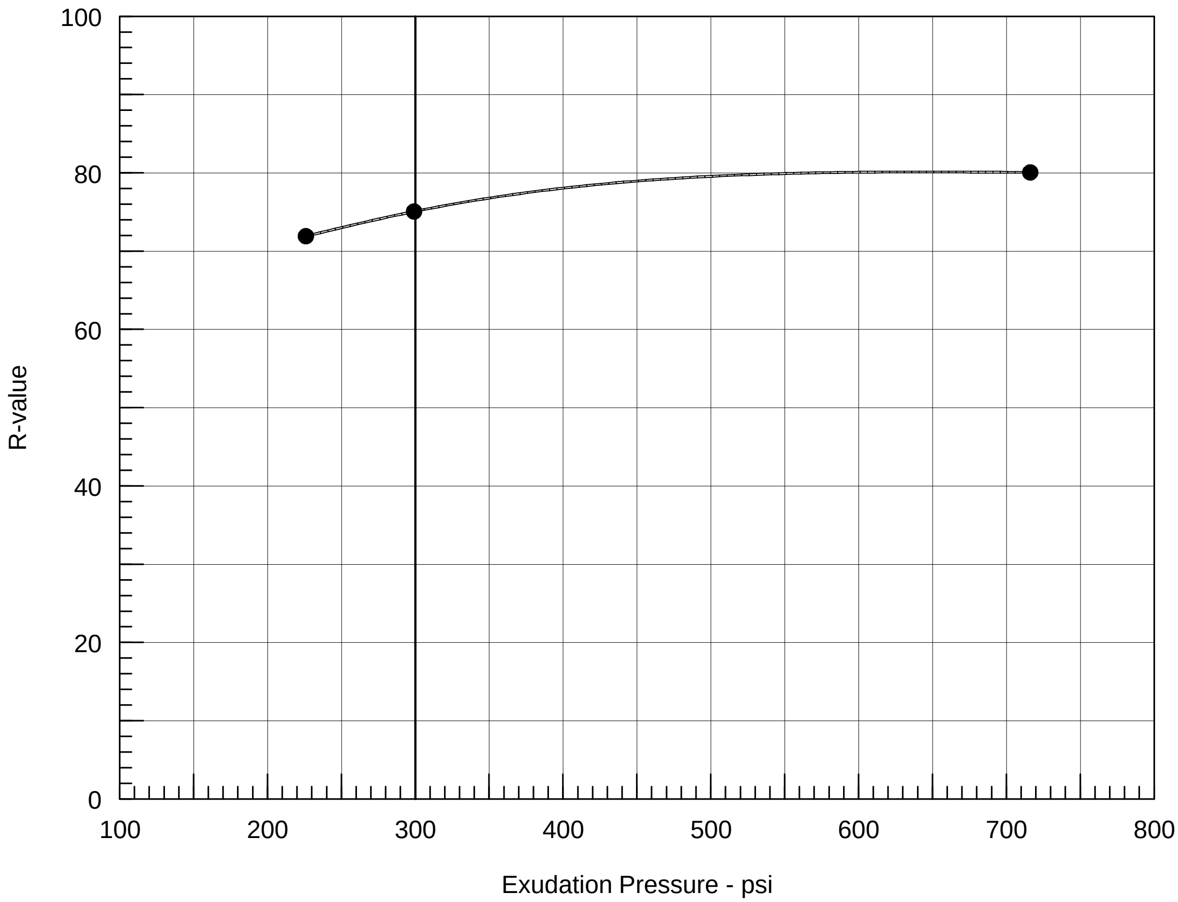
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

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	350	125.1	8.7	0.00	22	2.37	299	77	75
2	350	123.8	9.2	0.00	22	2.38	226	74	72
3	350	123.9	8.2	0.00	18	2.39	716	81	80

Test Results	Material Description
R-value at 300 psi exudation pressure = 75	Silty sand

Project No.: H02901.01

Project: The Neighborhoods at Lugonia Village, Redlands

Sample Number: B-22 **Depth:** 3.5-5'

Date: 12/20/2021

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

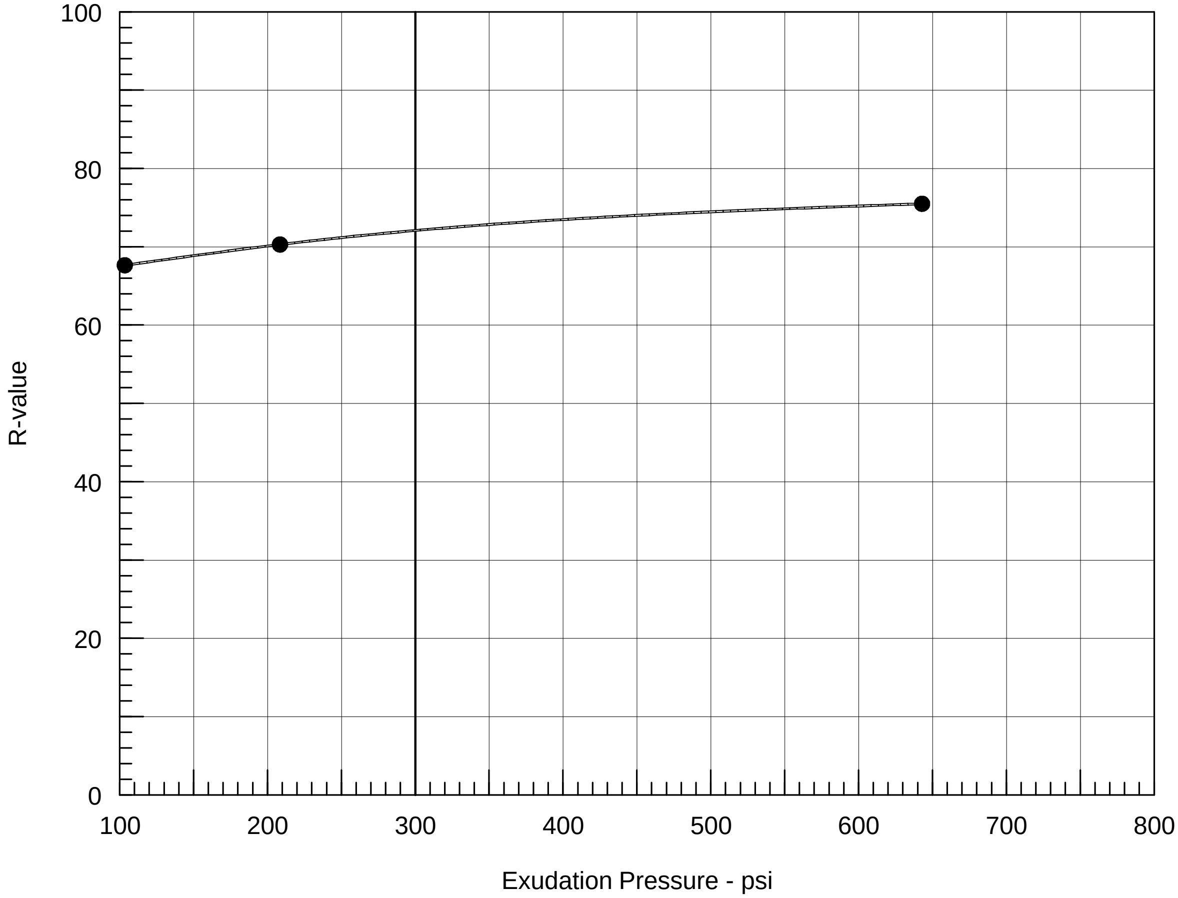
Tested by: MP

Checked by: MS

Remarks:

Figure N/A

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	350	120.9	9.5	0.00	20	2.42	643	77	75
2	350	119.9	10.6	0.00	27	2.43	103	69	68
3	350	121.2	10.1	0.00	25	2.41	208	72	70

Test Results	Material Description
R-value at 300 psi exudation pressure = 72	Silty sand

Project No.: H02901.01

Project: The Neighborhoods at Lugonia Village, Redlands

Sample Number: B-28 **Depth:** 0-3.5'

Date: 12/20/2021

Tested by: MP

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure NA

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

December 03, 2021

Work Order #: **HK24011**

Allen Harker
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: The Neighborhoods At Lugonia Village, Redlands

Enclosed are the analytical results for samples received by our laboratory on **11/24/21** . For your reference, these analyses have been assigned laboratory work order number **HK24011**.

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.

Susan Federico
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 2527 Fresno Street Fresno CA, 93721	**Project:** The Neighborhoods At Lugonia Village, Redlands **Project Number:** H02901.01 **Project Manager:** Allen Harker	**Reported:** 12/03/2021

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B5@ 0-3.5'		HK24011-01	Soil	11/18/21 00:00	11/24/21 11:20
B16@ 0-3.5'		HK24011-02	Soil	11/16/21 00:00	11/24/21 11:20
B31@ 0-3.5'		HK24011-03	Soil	11/17/21 00:00	11/24/21 11:20

Amendment: Corrected Project name per client request via email. SMF 12/3/21

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 2527 Fresno Street Fresno CA, 93721	**Project:** **Project Number:** **Project Manager:**	The Neighborhoods At Lugonia Village, Redlands H02901.01 Allen Harker	**Reported:** 12/03/2021

B5@ 0-3.5'

HK24011-01 (Soil) Sampled: 11/18/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 422
Chloride		ND	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
Sulfate as SO4		ND	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
pH		**7.3**	0.10	pH Units	1	B1L0112	12/01/21	12/02/21	Cal Test 643
Sulfate as SO4		ND	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 417

B16@ 0-3.5'

HK24011-02 (Soil) Sampled: 11/16/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**7.4**	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 422
Chloride		**0.00074**	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
Sulfate as SO4		ND	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
pH		**7.2**	0.10	pH Units	1	B1L0112	12/01/21	12/02/21	Cal Test 643
Sulfate as SO4		ND	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 417

B31@ 0-3.5'

HK24011-03 (Soil) Sampled: 11/17/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		ND	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 422
Chloride		ND	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
Sulfate as SO4		ND	0.00060	% by Weight	3	[CALC]	12/02/21	12/02/21	[CALC]
pH		**7.8**	0.10	pH Units	1	B1L0112	12/01/21	12/02/21	Cal Test 643
Sulfate as SO4		ND	6.0	mg/kg	3	B1L0112	12/01/21	12/02/21	Cal Test 417

Notes and Definitions

RPD2	A high RPD was observed due to the low concentration of the target analyte.
µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
	Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field. If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

MOORE TWINING ASSOCIATES, INC.

Since 1898

Project Name:	The Neighborhood at Lugonia, Redlands	Report Date:	12/20/2021
		Sample Date:	11/15/2021
Project Number:	H02901.01		
		Sampled By:	YA
Subject:	Minimum Resistivity, ASTM G187	Tested By:	AL
Material Description:	Silty sand	Test Date:	12/15/2021
Location:	B-5 @ 0-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
75 mls	45,000
100 mls	32,000
125 mls	25,000
150 mls	20,000
175 mls	19,000
200 mls	20,000

Remarks: Min. Resistivity is 19,000 Ohm-cm

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

MOORE TWINING
ASSOCIATES, INC.

Project Name:	The Neighborhood at Lugonia, Redlands	Report Date:	12/20/2021
		Sample Date:	11/15/2021
Project Number:	H02901.01		
		Sampled By:	YA
Subject:	Minimum Resistivity, ASTM G187	Tested By:	AL
Material Description:	Silty sand	Test Date:	12/15/2021
Location:	B-16 @ 0-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
75 mls	51,000
100 mls	36,000
125 mls	27,000
150 mls	25,000
175 mls	25,000
200 mls	26,000

Remarks: Min. Resistivity is 25,000 Ohm-cm

www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

MOORE TWINING ASSOCIATES, INC.

Since 1898

Project Name:	The Neighborhood at Lugonia, Redlands	Report Date:	12/20/2021
		Sample Date:	11/15/2021
Project Number:	H02901.01		
		Sampled By:	YA
Subject:	Minimum Resistivity, ASTM G187	Tested By:	AL
Material Description:	Silty sand	Test Date:	12/15/2021
Location:	B-31 @ 0-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	75,000
75 mls	51,000
100 mls	33,000
125 mls	28,000
150 mls	29,000
175 mls	30,000

Remarks: Min. Resistivity is 28,000 Ohm-cm

www.mooretwining.com

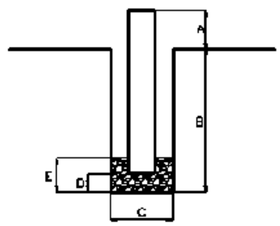
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST
No. P-1

Project:	The Neighborhoods at Lugonia Village	**Project No.**	H02901.01
Location:	NWC of Lugonia Avenue and Karon Street, Redlands, CA	**Test Date:**	11/18/2021
Coordinates:			



A. Top of Pipe Above Ground	18	Inches
B. Depth of Hole	180	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	30	Inches
F. Pipe Length	196	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: 5 gallons of water on 11/18/21 at 2:30 p.m.
Checked Dry at 3 p.m. on 11/18/21

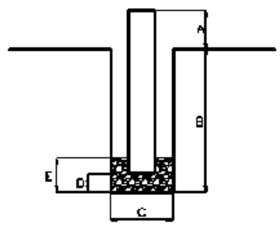
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	11/18/2021	15:01:58	16				
	11/18/2021	15:06:00	16.48	4.03	5.76	1.8	12.9
2	11/18/2021	15:08:00	16.1				
	11/18/2021	15:13:00	16.38	5.00	3.36	3.8	6.1
3	11/18/2021	15:15:00	16.1				
	11/18/2021	15:20:00	16.47	5.00	4.44	2.9	9.0
4	11/18/2021	15:22:00	16.1				
	11/18/2021	15:27:00	16.48	5.00	4.56	2.8	9.3
5	11/18/2021	15:30:00	16.1				
	11/18/2021	15:35:00	16.61	5.00	6.12	2.1	15.1
6	11/18/2021	15:37:00	16.1				
	11/18/2021	15:42:00	16.63	5.00	6.36	2.0	16.2
7	11/18/2021	15:45:00	16.1				
	11/18/2021	15:50:00	16.62	5.00	6.24	2.1	15.7
8	11/18/2021	15:52:00	16.1				
	11/18/2021	15:57:00	16.62	5.00	6.24	2.1	15.9
9	11/18/2021	16:00:00	16.1				
	11/18/2021	16:05:00	16.62	5.00	6.24	2.1	15.9
10	11/18/2021	16:07:00	16.1				
	11/18/2021	16:12:00	16.62	5.00	6.24	2.1	15.9
11	11/18/2021	16:14:00	16.1				
	11/18/2021	16:19:00	16.62	5.00	6.24	2.1	15.9
12	11/18/2021	16:20:00	16.1				
	11/18/2021	16:25:00	16.62	5.00	6.24	2.1	15.9

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project: The Neighborhoods at Lugonia Village **Project No.** H02901.01
Location: NWC of Lugonia Avenue and Karon Street, Redlands, CA **Test Date:** 11/18/2021
Coordinates:



A. Top of Pipe Above Ground	10 Inches
B. Depth of Hole	126 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	32 Inches
F. Pipe Length	134 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 5 gallons of water on 11/19/21 at 6:30 a.m.
Checked Dry at 7 a.m. on 11/19/21

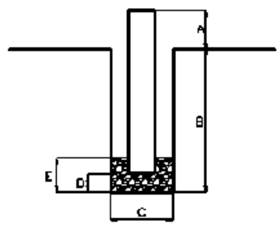
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	11/19/2021	7:05:00	11.1				
	11/19/2021	7:10:00	11.51	5.00	4.92	2.6	19.4
2	11/19/2021	7:12:00	11.1				
	11/19/2021	7:17:00	11.4	5.00	3.6	3.6	11.1
3	11/19/2021	7:20:00	11.1				
	11/19/2021	7:25:00	11.48	5.00	4.56	2.8	16.7
4	11/19/2021	7:26:00	11.1				
	11/19/2021	7:31:00	11.38	5.00	3.36	3.8	9.9
5	11/19/2021	7:32:00	11.1				
	11/19/2021	7:37:00	11.31	5.00	2.52	5.1	6.6
6	11/19/2021	7:38:00	11.1				
	11/19/2021	7:43:00	11.32	5.00	2.64	4.8	7.0
7	11/19/2021	7:45:00	11.1				
	11/19/2021	7:50:00	11.32	5.00	2.64	4.8	7.0
8	11/19/2021	7:51:00	11.1				
	11/19/2021	7:56:00	11.32	5.00	2.64	4.8	7.1
9	11/19/2021	7:57:00	11.1				
	11/19/2021	8:02:00	11.32	5.00	2.64	4.8	7.1
10	11/19/2021	8:04:00	11.1				
	11/19/2021	8:09:00	11.32	5.00	2.64	4.8	7.1
11	11/19/2021	8:10:00	11.1				
	11/19/2021	8:15:00	11.32	5.00	2.64	4.8	7.1
12	11/19/2021	8:16:00	11.1				
	11/19/2021	8:21:00	11.32	5.00	2.64	4.8	7.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-3

Project:	The Neighborhoods at Lugonia Village	**Project No.**	H02901.01
Location:	NWC of Lugonia Avenue and Karon Street, Redlands, CA	**Test Date:**	11/18/2021
Coordinates:			



A. Top of Pipe Above Ground	43 Inches
B. Depth of Hole	115 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	28 Inches
F. Pipe Length	156 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 5 gallons of water on 11/19/21 at 8:30 a.m.
Checked Dry at 8:58 a.m. on 11/19/21

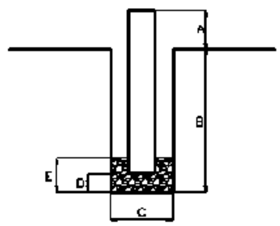
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	11/19/2021	8:55:00	10.5				
	11/19/2021	9:00:00	10.81	5.00	3.72	3.4	1.1
2	11/19/2021	9:01:00	10.5				
	11/19/2021	9:06:00	10.91	5.00	4.92	2.6	1.4
3	11/19/2021	9:07:00	10.5				
	11/19/2021	9:12:00	10.98	5.00	5.76	2.2	1.7
4	11/19/2021	9:14:00	10.5				
	11/19/2021	9:19:00	10.76	5.00	3.12	4.1	0.9
5	11/19/2021	9:20:00	10.5				
	11/19/2021	9:25:00	10.76	5.00	3.12	4.1	0.9
6	11/19/2021	9:27:00	10.5				
	11/19/2021	9:32:00	10.76	5.00	3.12	4.1	0.9
7	11/19/2021	9:37:00	10.5				
	11/19/2021	9:42:00	10.76	5.00	3.12	4.1	0.9
8	11/19/2021	9:45:00	10.5				
	11/19/2021	9:50:00	10.76	5.00	3.12	4.1	0.9
9	11/19/2021	9:51:00	10.5				
	11/19/2021	9:56:00	10.77	5.00	3.24	3.9	0.9
10	11/19/2021	9:58:00	10.5				
	11/19/2021	10:03:00	10.76	5.00	3.12	4.1	0.9
11	11/19/2021	10:05:00	10.5				
	11/19/2021	10:10:00	10.76	5.00	3.12	4.1	0.9
12	11/19/2021	10:12:00	10.5				
	11/19/2021	10:17:00	10.76	5.00	3.12	4.1	0.9

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-4

Project: The Neighborhoods at Lugonia Village **Project No.** H02901.01
Location: NWC of Lugonia Avenue and Karon Street, Redlands, CA **Test Date:** 11/18/2021
Coordinates:



A. Top of Pipe Above Ground	13.5 Inches
B. Depth of Hole	182 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	45 Inches
F. Pipe Length	193.5 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 5 gallons of water on 11/18/21 at 13:45 p.m.
Checked Dry at 14:15 p.m. on 11/18/21

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	11/18/2021	14:22:08	15				
	11/18/2021	14:27:16	15.51	5.13	6.12	2.1	3.8
2	11/18/2021	14:29:19	15.1				
	11/18/2021	14:34:07	15.42	4.80	3.84	3.2	2.6
3	11/18/2021	14:39:02	15.1				
	11/18/2021	14:44:04	15.49	5.03	4.68	2.8	3.1
4	11/18/2021	14:45:06	15.1				
	11/18/2021	14:50:07	15.48	5.02	4.56	2.8	3.0
5	11/18/2021	14:51:16	15.1				
	11/18/2021	14:56:26	15.39	5.17	3.48	3.8	2.1
6	11/18/2021	14:57:28	15.1				
	11/18/2021	15:02:31	15.42	5.05	3.84	3.4	2.4
7	11/18/2021	15:03:04	15.1				
	11/18/2021	15:08:01	15.43	4.95	3.96	3.2	2.6
8	11/18/2021	15:09:00	15.1				
	11/18/2021	15:14:02	15.43	5.03	3.96	3.3	2.6
9	11/18/2021	15:15:18	15.1				
	11/18/2021	15:20:36	15.43	5.30	3.96	3.4	2.4
10	11/18/2021	15:21:15	15.1				
	11/18/2021	15:26:15	15.43	5.00	3.96	3.2	2.6
11	11/18/2021	15:27:10	15.1				
	11/18/2021	15:32:11	15.43	5.02	3.96	3.2	2.6
12	11/18/2021	15:33:10	15.1				
	11/18/2021	15:38:01	15.42	4.85	3.84	3.2	2.6

* Depth to water measured from top of pipe

APPENDIX E

PHOTOGRAPHS

Photograph No. 1: Viewing east along south side of site adjacent to West Lugonia Avenue

Photograph No. 2: Viewing east at U-shaped concrete structure (electrical easement) that trends in a north-south orientation across the south-central portion of the site

Photograph No. 3: Viewing north along east side of site; small rodent burrows in foreground

Photograph No. 4: Viewing south along east side of site; west-facing slope adjacent to Karon Street shown in background on left side of picture

Photograph No. 5: Rectangular-shaped concrete structure filled with debris in northeast corner of site near the intersection of Karon Street and Pennsylvania Avenue

Photograph No. 6: Rectangular-shaped structure in northeast corner of site and U-shaped concrete drainage structure in background trending east-west along center line of future extension of Pennsylvania Avenue

Photograph No. 7: U-shaped concrete structure trending east-west along center line of future extension of Pennsylvania Avenue is about 4.5 feet wide and 12 inches in height

Photograph No. 8: Viewing east from northwest corner of site showing scattered concrete debris in foreground

Photograph No. 9: Viewing south along west side of site showing concrete drainage structure (U-shaped structure in foreground and circular concrete pipe in background)

Photograph No. 10: Close-up of concrete drainage structure on west side of site

Photograph No. 11: Seven-inch deep rodent burrow on west side of site

Photograph No.12: Thirteen-inch deep rodent burrow on west side of site

Photograph No. 13: Open cylindrical excavation (shaft) made of brick, concrete and filled with debris in the southwest portion of the site

Photograph No. 14: Open cylindrical excavation (shaft) in southwest portion of site is about 5 feet in diameter