MOORE TWINING ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT

THE RESIDENCE AT CASA LOMA

SOUTHEAST CORNER OF EAST LUGONIA AVENUE

AND OCCIDENTAL DRIVE

REDLANDS, CALIFORNIA

Project Number: G57101.01-01

For:

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

February 12, 2018
(Revised December 5, 2019)

February 12, 2018 (Revised December 5, 2019) E88904.01-01

Mr. Alex Kang
Planning Analyst
The Planning Associates Group
9880 Irvine Center Drive, Suite 100
Irvine, CA 92618

Subject: **Geotechnical Engineering Investigation**
Proposed Multi-Family Residential Development
The Residence at Casa Loma
Southeast Corner of East Lugonia Avenue and Occidental Drive
Redlands, California

Dear Mr. Kang:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed multi-family residential development (The Residence at Casa Loma) to be located at the southeast corner of East Lugonia Avenue and Occidental Drive 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.

We appreciate the opportunity to be of service to The Planning Associates Group. 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.

Allen H. Harker

Allen H. Harker, PG
Project Geologist
Geotechnical Engineering Division

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for the proposed multi-family residential development (The Residence at Casa Loma) to be located at the southeast corner of East Lugonia Avenue and Occidental Drive in Redlands, California.

It is our understanding the project will include a total of 147 multi-family residential units in three (3) separate buildings. The proposed buildings are indicated to include 2 and 3 story wood-framed construction with concrete slabs-on-grade. In addition, the proposed improvements will include an in-ground pool, in-ground spa, carports, multi-purpose room, leasing office, courtyard areas and a car wash area. Appurtenant construction is anticipated to include asphalt concrete and concrete pavements, concrete flatwork, underground utilities, and landscaped areas.

At the time of our investigation, the eastern portion of the subject site was partially developed with residential structures and various associated improvements including asphalt concrete paved driveways, fences and concrete flatwork.

The site also includes existing street and sidewalk improvements (Crystal Court). The majority of the site appears to have been previously graded for a residential subdivision. At the time of our field exploration, the vacant portion of the site was generally covered by silty sand soils and scattered gravel, cobbles, and boulders, and construction debris.

On January 17 and 18, 2018, seven (7) test borings were drilled in the proposed building areas to depths ranging from about 20 to 51½ feet below site grades (BSG). In addition, four (4) percolation test borings were drilled in the anticipated proposed BMP infiltration areas to depths ranging from about 3 to 5 feet BSG.

The soils encountered in the borings conducted for this investigation consisted of silty sands with varying amounts of gravel and potential cobbles extending to depths of about 1½ to 13½ feet BSG. The silty sands are underlain by interbedded layers of poorly graded sands with silt, silty sands and silty sands with gravel, well graded sands with silt and gravel, silty gravel with potential cobbles, poorly graded sands, and poorly graded gravel with potential cobbles extending to the maximum depth explored, about 51½ feet BSG. The soil layers described above are also anticipated to contain localized boulders; however, the presence of boulders could not be confirmed due to the small size of the boreholes.

Groundwater was not encountered in the test borings drilled at the time of our January 17 and 18, 2018 field exploration to the maximum depth explored, about 51½ feet BSG.

Based on our field and laboratory investigation, the near surface soils tested possess a very low expansion potential, low compressibility characteristics, and excellent pavement support characteristics when compacted as engineered fill.

In order to limit the static settlement of new foundations to 1 inch total and ½ inch differential, over-excavation of the near surface soils and placement of engineered fill is recommended below foundations. In addition, over-excavation will be required to remove soils disturbed from removal of surface and subsurface improvements and all fill soils that are encountered.

EXECUTIVE SUMMARY (continued)

Since groundwater is anticipated to be at a depth of 100 feet or greater, the potential for liquefaction to occur is low. However, the analyses indicated that some of the granular soil layers encountered in boring B-1 would be subject to dry seismic settlement. The majority of the dry seismic settlement occurs from the granular soil layers encountered at depths of greater than 30 feet BSG. Seismic settlements were estimated to be about ¾ inches total and ½ inch differential in 40 feet.

Chemical testing of the near surface soil samples indicated the soils exhibit a “corrosive”corrosion potential. Chemical analyses also indicated a “negligible” potential for sulfate attack on concrete placed in contact with the near surface soils.

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

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AND OCCIDENTAL DRIVE

REDLANDS, CALIFORNIA

Project Number: G57101.01-01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed multi-family residential development (The Residence at Casa Loma) to be located at the subject property in Redlands, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by The Planning Associates Group 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 BMPs;

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 geologic/seismic hazards evaluation, flood plain investigation, compaction tests, environmental investigation, nor an environmental audit.

2.2 **<u>Scope</u>:** Our proposal, dated December 21, 2017, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 The Residence at Casa Loma, Design Review Submittal, dated April 5, 2019, including a Conceptual Site Plan, provided by The Planning Associates Group, was reviewed to gain an understanding of the proposed structures.

2.2.2 A Topographic Survey, dated February 14, 2017, prepared by RdM Surveying, Inc., was reviewed.

2.2.3 A Conceptual Grading Plan, prepared by DRC Engineering, Inc., dated December 13, 2018, was reviewed.

2.2.4 Tentative Tract Map No. 20162, prepared by DRC Engineering, Inc., dated April 18, 2018, was reviewed.

2.2.5 A visual site reconnaissance and subsurface exploration were conducted.

2.2.6 Satellite images of the site between the years 1995 and 2016 from online sources, were reviewed. In addition, historical aerial photographs, dated various years between 1930 and 2012, prepared by Environmental Data Resources, Inc., were reviewed.

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

2.2.8 Mr. Alex Kang (The Planning Associates Group), Mr. Jesus Navidad (The Planning Associates Group), and Mr. Howard Hardin (The Planning Associates Group), were consulted during the investigation.

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

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

3.1 Site Description: The proposed development is to be located on an L-shaped property located at the southeast corner of East Lugonia Avenue and Occidental Drive in Redlands, California. The general site location is noted on Drawing No. 1 in Appendix A of this report. The overall site appears to comprise seventeen (17) separate parcels with a total area of about 5.7 acres as depicted on Tentative Tract Map No. 20162, prepared by DRC Engineering, Inc., dated April 18, 2018.

The easternmost portion of the site includes three (3) existing single-family residences (see photograph Nos. 7 through 10 in Appendix E of this report). The improvements around the residences included asphalt concrete paved driveways, concrete flatwork, underground utilities, fences, and landscaped areas, including several juvenile and mature trees. The three residential properties are bordered to the south by vacant land (not part of the subject site) and East Brockton Avenue beyond, to the east by University Street, and to the north by an existing apartment complex with East Lugonia Avenue beyond.

The remainder of the site is bordered to the west by Occidental Drive, to the south by residential properties and to the north by an apartment complex ans East Lugonia Avenue.

The subject site includes an asphalt concrete paved street named Crystal Court with a cul-de-sac and surrounding concrete sidewalk (see photograph No. 4 in Appendix A). Underground utilities for water and gas were marked along the sidewalk bordering Crystal Court. Underground utility services and underground storm drain improvements have been installed as part of the existing Crystal Court street improvements.

A man-made earthen drainage ditch is located in the southwest portion of the site, on the south side of Crystal Court (see photograph No. 3 in Appendix A). It appears that it was made to collect and infiltrate runoff from Crystal Court. A storm water inlet was noted along the south side of Crystal Court and an outlet was noted on the north side of the earthen drainage ditch which was completed filled with soil. The earthen drainage ditch has a five (5) foot tall slope on the east side with an inclination of approximately 1 Horizontal to 1 Vertical (1H:1V) and a two (2) foot tall slope on the west side with an inclination of approximately 1.5H:1V.

The undeveloped areas of the site were noted to have been previously graded with relative flat lots for a residential subdivision. Outside the existing street improvements, the majority of the site was generally covered by silty sand soils with scattered gravel, cobbles, and boulders (see photograph No. 2 in Appendix A). Some of the boulders were piled together, possibly a result of sorting oversized rock during previous grading of the site. Scattered trash and debris were also noted at the site, including materials such as a chain link fence, a shopping cart, a trash can, fragments of asbestos pipe, sections of plastic pipe, glass debris, metal debris, and fragments of concrete, wood, brick and asphalt debris (photograph No. 5 in Appendix A shows fragment of asbestos pipe with cobbles and boulders). Scattered brush was also noted across the vacant portion of the site. Trees were noted along portions of the southern property boundary. A stockpile of soil, weeds, cobbles and boulders, concrete debris, wood, metal debris was also noted (see photograph No. 1 in Appendix A).

A concrete masonry unit screen wall (see photograph No. 6 in Appendix A) was noted on the west and south sides of the existing apartment complex that borders the subject site (vacant lot and three (3) single-family residential properties).

A Topographic Survey, dated February 14, 2017, prepared by RdM Surveying, Inc., was reviewed. The Topographic Survey indicates that the site ranges in elevation from about 1,424 feet above mean sea level (AMSL) at the bottom of the earthen drainage ditch located in the southwestern portion of the site, adjacent to Occidental Drive, to about 1,443 feet AMSL in the eastern portion of the site, adjacent to North University Street.

3.2 **<u>Site History</u>:** Historical aerial photographs, dated various years between 1930 and 2012, prepared by Environmental Data Resources, Inc., were reviewed. Based on our review of the 1930 and 1938 aerial photographs, the eastern portion of the site was vacant. The 1949, 1953 and 1959 aerial photographs show a residence in the eastern portion of the site. An aerial photograph, dated 1966, shows the eastern portion of the site as being subdivided with three single-family residences as currently located at 1219, 1215 and 1205 North University Street. These residential improvements appear relatively similar to those noted during our site observations.

In the remainder of the site, the 1930 historical aerial photograph shows a residence in the southwestern portion of the site and the northwestern portion of the site appeared to be occupied by several single-family residences. The 1938, 1949, 1953, 1959, 1966, 1968, and 1975 aerial photographs show the single-family residence in the southwestern portion of the site as being surrounded by rows of trees while the northwestern portion of the site shows the area occupied by single-family residences with some trees which may be associated with agricultural use. The 1985 aerial photograph indicates the trees had been removed, but a single-family residence is still shown as existing in both the southwestern and northwestern portions of the site. The 1995 aerial photograph shows the northwestern portion of the site as being vacant while the single-family residence is still shown as remaining in the southwestern portion of the site. The undeveloped area in the northwestern portion of the L-shaped site generally appears unchanged between the 1995 and 2016 aerial satellite images of the site.

The May 2002 aerial satellite image of the site shows the residence in the southwestern portion of the site as still remaining. However, the residence in the southwestern portion of the site appears to have been removed in an October 2003 aerial satellite image of the site, and a portion of the site surrounding the former residence appears to have been graded. A January 2004 aerial satellite image of the site shows the area directly west of the three (3) residences bordering North University Street as being graded for construction of Crystal Court (street providing access to the site from Occidental Drive) and associated residential lots. The April 2007 aerial satellite image of the site shows Crystal Court as having been paved with asphalt concrete with a concrete sidewalk along the sides of the street, and the area directly surrounding Crystal Court appears to have been graded again to remove any vegetation growth. A man-made drainage ditch in the southwestern corner of the site also appears to have been excavated at the time of the paving of Crystal Court in 2007 for collection and infiltration of runoff from Crystal Court. More recent satellite images of the site appear generally the same, with the exception that the site appears to be periodically disced or graded to remove vegetation growth.

3.3 **Previous Studies:** At the time this report was being prepared, Moore Twining's environmental division was preparing a Phase I Environmental Site Assessment report for the subject site. Historic photographs obtained as part of the Phase I Environmental Site Assessment were reviewed and are discussed above in the Site History section of this report.

No other 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 **Anticipated Construction and Grading:** It is our understanding the proposed multi-family residential development will include a total of 147 multi-family residential units in three (3) buildings identified as Buildings 1, 2, and 3 as depicted on the Conceptual Site Plan, prepared by Architects Orange, dated April 5, 2019. The proposed buildings are anticipated to include 2 and 3 story wood-framed construction with concrete slabs-on-grade. In addition, the proposed improvements will include an in-ground swimming pool, in-ground spa, carports, multi-purpose room, leasing office, courtyard areas and a car wash area. Appurtenant construction is anticipated to include asphalt concrete and concrete pavements, concrete flatwork, underground utilities, and landscaped areas.

In addition, it is understood that the project will include construction of onsite BMP /infiltration system(s). At the time of our January 2018 field exploration, it was understood that the BMP / infiltration systems may extend about 3 feet below grade. The final location of the proposed BMP infiltration system(s) were not known at the time of our January 2018 field exploration. However, based on our discussions with Mr. Howard Hardin (The Planning Associates Group) prior to our investigation in January 2018, the BMP infiltration systems were expected to be located in landscaped areas on the east side of Building 1, adjacent to North University Street, and on the east side of Building 4, adjacent to the existing apartment complex. After our January 2018 field exploration, Moore Twining was provided a Conceptual Utility Plan (Sheet 2 of 2) for the proposed project, prepared by DRC Engineering, Inc., dated December 13, 2018, showing underground infiltration chambers were proposed for storm water infiltration in the southwest portion of the site. Moore Twining conducted a supplemental investigation to conduct double-ring infiltration testing in the area of the proposed underground infiltration chambers. The results of this supplemental investigation and supplemental recommendations are included in our separate document entitled, "Results of Double-Ring Infiltrometer Test and Supplemental Recommendations, Storm Water Infiltration System, Proposed Residence at Casa Loma, Southeast of East Lugonia Avenue and Occidental Drive, Redlands, San Bernardino County, California," dated April 5, 2019.

For the purpose of this report, maximum column loads of about 40 kips and maximum perimeter wall loads of 3 kips per linear foot were assumed as preliminary structural loads for the purpose of this report. 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.

Based on our review of the referenced Conceptual Grading Plan and Topographic Survey, Building 1 is shown to have finished floor elevations ranging from 1,438.3 to 1,440.0 feet above mean sea level (AMSL), which would require fills of up to about 2 to 6 feet to achieve the finished floor elevation. Building 2 is shown to have finished floor elevations ranging from 1,435.2 to 1,437.1 feet AMSL, which would require fills on the order of about 4 to 6 feet to achieve the finished floor elevation. Building 3 is shown to have a finished floor elevation of 1,433.3 feet AMSL, which would require fills on the order of about 1 to 2 feet to achieve the finished floor elevation. The greatest amount of fill planned at the site appear to be in the southwest corner of the site (proposed pavement areas) which would require up to about 6 to 7 feet of fill in order to fill in the existing man-made drainage ditch. The Conceptual Grading Plan, dated December 13, 2018, prepared by DRC Consultants, Inc., indicates the site will require import.

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, 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. Allen Harker of Moore Twining on January 17 and 18, 2018. 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.

On January 17 and 18, 2018, seven (7) test borings were drilled at the site in the proposed building areas to depths ranging from about 20 to 51½ feet below site grades (BSG). In addition, four (4) percolation test borings were drilled at the site in the anticipated proposed BMP infiltration areas to depths ranging from about 3 to 5 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 brass 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>Percolation Testing</u>: Percolation tests were conducted in the four (4) borings (B-7/P-1, B-8/P-2, B-10/P-3 and B-11/P-4) where shown on Drawing No. 2 in Appendix A of this report. The percolation test borings were drilled to depths of approximately 3 to 5 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 January 18 and 19, 2018 in accordance with the percolation test procedure noted in section VII.3.8 from the Technical Guidance Document Appendices (Appendix 7: Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations), dated May 19, 2011, prepared by Riverside County. The test holes were cylindrical with a diameter of about 8 inches. Gravel packing was used to protect the sidewalls of the holes from washout during refilling. A 2-inch diameter perforated PVC pipe was placed in the boreholes and used to transmit poured water to the bottom of the holes. Granular soils were encountered at the bottom of the percolation tests holes. The percolation holes were presoaked with about 5 gallons of water. Two (2) consecutive measurements were recorded that indicated at least 6 inches of water had seeped away in less than 25 minutes; and thus, the percolation tests in the sandy soils were run immediately after the presoak water had seeped away. The percolation tests were run for an additional hour with measurements taken every 1 to 2 minutes.

Percolation testing included adding water to the test holes periodically and measuring the drop in water level over time. Measurements of water levels and the time of each reading were recorded during testing. The percolation test holes were filled with water to a depth equal to at least 5 times the test hole radius above the gravel at the bottom of the test hole. The head of the water in the test holes during the percolation tests generally ranged from approximately 1¾ to 2½ feet when filled or refilled with water. In accordance with Riverside County's Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations document referenced above, the drop that occurs in the final reading was used to calculate the percolation rate.

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>Surface Conditions</u>:** At the time of our investigation, the eastern portion of the site was developed with residential structures and associated improvements including asphalt concrete paved driveways, concrete flatwork, fences, screen walls, landscaped areas, including several juvenile and mature trees. In addition, public street and sidewalk improvements (Crystal Court) are present within the site. Underground utilities have been installed as part of the existing street improvements. In addition, the vacant portion of the site was generally covered with scattered gravel, cobbles, boulders, scattered trash, construction debris, and fragments of concrete, wood, brick and asphalt debris. Scattered brush was noted across the undeveloped portion of the site and several mature trees were located within the site area. Additional information regarding the existing site features is included in the background portion of this report.

5.2 **Soil Profile:** The soils encountered in the borings conducted for this investigation consisted of silty sands with varying amounts of gravel and potential cobbles extending to depths of about 1½ to 13½ feet BSG. The silty sands were underlain by interbedded layers of poorly graded sands with silt, silty sands and silty sands with gravel, well graded sands with silt and gravel, silty gravel with potential cobbles, poorly graded sands, and poorly graded gravel with potential cobbles extending to the maximum depth explored, about 51½ feet BSG. The soil described above are also anticipated to contain localized boulders; however, the presence of boulders could not be confirmed due to the small size of the boreholes (6⅝ inches and 8 inches in diameter) that were drilled. Based on our observations of scattered cobbles and boulders across the ground surface of the site, boulders are anticipated to be present within the near surface soils. The USDA soil survey indicates the near surface soils contain 3 to 4 percent cobbles. Fill soils were not readily distinguishable from the native soils due to the granular nature of the soils and the absence of any construction debris. However, due to previous development and grading of the site, the near surface soils within the upper about 2 to 3 feet are likely to be fill soils.

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.

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

Silty Sands and Silty Sands with Gravel and Potential Cobbles: The silty sands with varying amounts of gravel and cobbles were described as medium dense to very dense, as determined by standard penetration resistance, N-values, ranging from 5 to greater than 50 blows per foot. The moisture content of the samples tested ranged from about 3 to 8 percent. Five (5) relatively undisturbed samples revealed dry densities of 116.1, 120.8, 112.1, 117.4, and 116.3 pounds per cubic foot. Two expansion index tests indicated expansion index values of 0. A direct shear test conducted on a sample collected from boring B-5 at depths of 3½ to 5 feet BSG indicated an internal angle of friction of 43 degrees and 100 pounds per square foot of cohesion. Four (4) consolidation tests conducted on near surface samples collected in the upper five (5) feet BSG from borings B-1, B-3, B-5 and B-9 indicated low compressibility characteristics (2.1, 3.3, 2.5, and 1.9 percent consolidation under a load of 8 kips per square foot).

Poorly Graded Sands and Poorly Graded Sands with Silt: The poorly graded sands and poorly graded sands with silt and varying amounts of gravel and cobbles were described as loose to dense, as determined by standard penetration resistance, N-values ranging from 7 to greater than 50 blows per foot. The moisture content of the samples tested ranged from 2 to 6 percent. Three (3) relatively undisturbed samples revealed dry densities of 122.2, 107.3, and 123.0 pounds per cubic foot. A consolidation test conducted on a sample collected from boring B-6 at depths of 2 to 3½ feet BSG indicated low compressibility characteristics (2.1 percent consolidation under a load of 8 kips per square foot).

Well Graded Sand with Silt and Gravel: The well graded sand with silt and gravel was described as medium dense, as determined by a standard penetration resistance, N-value, of 28 blows per foot.

Silty Gravel with Sand and Potential Cobbles: The silty gravel with sand and potential cobbles was described as dense, as determined by a standard penetration resistance, N-value, of 34 blows per foot. The moisture content of a silty gravel sample with sand and potential cobbles was about 3 percent. One (1) relatively undisturbed sample revealed a dry density of 123.7 pounds per cubic foot.

Poorly Graded Gravel with Sand and Potential Cobbles: The poorly graded gravel with sand and potential cobbles were described as medium dense, as determined by an SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 16 blows per foot. The moisture content of a sample tested was about 1 percent. One (1) sample revealed a dry density of 114.8 pounds per cubic foot.

Maximum Density/Optimum Moisture Content Determination: The results of a maximum density/optimum moisture content determination from a sample collected at depths of 0 to 5 feet BSG from boring B-4 indicated a maximum dry density of 127.8 pounds per cubic foot at an optimum moisture content of 7.1 percent.

R-value Tests: Two R-value tests conducted on a near surface silty sand sample and a near surface sample containing a mixture of silty sand and poorly graded sand collected from depths of about 0 to 5 feet BSG in borings B-2 and B-6 indicated R-values of 68 and 67, respectively.

Chemical Tests: Chemical tests performed on a near surface soil sample collected at depths of 2 to 5 feet BSG from boring B-1 indicated a pH value of 7.4; a minimum resistivity value of 4,736 ohms-centimeter; 0.0027 percent by weight concentrations of sulfate; and 0.00077 percent by weight concentrations of chloride. Chemical tests performed on a near surface soil sample collected at depths of 0 to 2½ feet BSG from boring B-11/P-4 indicated a pH value of 7.5; a minimum resistivity value of 4,135 ohms-centimeter; 0.0011 percent by weight concentrations of sulfate; and 0.0011 percent by weight concentrations of chloride.

5.4 **<u>Groundwater Conditions</u>:** Groundwater was not encountered in the test borings drilled at the time of our January 17 and 18, 2018 field exploration to the maximum depth explored, about 51½ feet BSG. Based on our review of water well data on the Department of Water Resources website, groundwater is anticipated at depths greater than 100 feet BSG.

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.

5.5 Percolation Test Results: The results of the percolation tests are summarized in Table No. 1 below. For the proposed on-site BMP / infiltration systems, the percolation tests were conducted at a depth of about 3 to 5 feet BSG within silty sand, poorly graded 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.

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
B-7/P-1 at 5 feet BSG	0.6	12	Poorly Graded Sand
B-8/P-2 at 3 feet BSG	0.5	15	Poorly Graded Sand
B-10/P-3 at 4.3 BSG	0.5	15	Poorly Graded Sand
B-11/P-4 at 3.75 feet BSG	0.7	7	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 subject site included single-family residences in the easternmost portion of the site. The developed areas included asphalt concrete paved driveways, concrete flatwork, fences, screen walls, underground utilities and landscaped areas, including several juvenile and mature trees.

The subject site includes a public street (Crystal Court) and associated underground utilities. Outside the existing street improvements, the majority of the site was generally covered by silty sands with scattered gravel, cobbles, and boulders. Some of the boulders were piled together, possibly a result of sorting oversized rock during previous grading at the site. Scattered trash and construction debris were also noted at the site, including materials such as a chain link fence, a shopping cart, a trash can, fragments of asbestos pipe, sections of plastic pipe, glass debris, metal debris, and fragments of concrete, wood, brick and asphalt debris. Scattered brush was also noted across the vacant portion of the site. Trees were noted along portions of the southern property boundary. A stockpile of soil, weeds, cobbles and boulders, concrete debris, wood, metal debris was also noted. As part of the site preparation, oversized rock materials and all debris should be removed from the site. Additional discussion regarding oversized rock materials is included in Section 6.2 of this report.

Where existing vegetation and landscaping is present, these areas should be stripped of all vegetation and top soil, and removal of trees and vegetation should remove all root balls and roots greater than ¼ inch in diameter.

As part of site preparation, underground utilities, including those associated with the Crystal Court construction, and all associated backfill soils within areas of proposed improvements which are sensitive to settlement should be removed and the excavations should be backfilled with engineered fill. If any of the existing underground utilities are planned to remain below new structural improvements, reports of compaction testing of the trench backfill should be provided to our firm for review and consideration.

Due to the granular nature of the on-site soils and absence of debris, fill soils could not be differentiated from native soils in the borings that were drilled. However, fill soils are anticipated at the site due to prior site grading and as part of removal of former structures. As part of the site preparation, existing undocumented fill soils encountered should be over-excavated and placed back as engineered fill in accordance with the recommendations of this report.

Due to the presence of existing and former residential structures, existing buried septic systems may be present at the site. In addition, foundations will need to be removed. During site preparation, the existing residences will need to be removed along with any subsurface structures/foundations/septic systems associated with the existing and former residences. Over-excavation should remove any 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.

6.2 **<u>Processing Onsite Soils with Gravel, Cobbles and Boulders for Use As Engineered Fill</u>:** Scattered cobbles and boulders were noted across the site. The near surface soils and soils at depths where cuts are planned are anticipated to contain coarse gravel, cobbles and potentially some boulders. The USDA soil survey indicates the near surface soils contain 3 to 4 percent cobbles. As part of site grading, the oversize cobbles larger than 6 inches and boulders will need to be removed prior to use of the soils as engineered fill.

Based on these conditions, the contractor will need to determine the methods they will use to remove the oversized rock and achieve the specified requirements for engineered fill.

6.3 **<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 expansive soil conditions are not anticipated. Therefore, special procedures to address expansive soils concerns are not anticipated for the project.

6.4 **<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 previous site grading and demolition and removal of structures and subsurface improvements, this report recommends that footings for the proposed buildings be supported on 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.

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.

6.5 **Seismic Ground Rupture and Design Parameters:** The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active fault is the San Andreas Fault, which is located approximately 3.6 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 0.875g 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).

6.6 **Liquefaction and Seismic Settlement:** Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction. 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 0.875g, a maximum considered earthquake magnitude of 8.1 and a groundwater depth of 100 feet were used in the analysis of the soils encountered in boring B-1 to a depth of about 51½ 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, liquefaction is not considered a concern. However, the analyses indicated that some of the granular soil layers encountered in boring B-1 would be subject to potential dry seismic settlement. The majority of the dry seismic settlement occurs from the granular soil layers encountered at depths of greater than 30 feet BSG. Seismic settlements were estimated to be about 1 inch total and ½ inch differential in 40 feet.

6.7 **<u>Asphaltic Concrete (AC) Pavements</u>:** 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.

Two (2) R-value tests were conducted on near surface samples, which indicated R-values of 68 and 67. 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.

6.8 **<u>Portland Cement Concrete (PCC) Pavements</u>:** 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.

6.9 **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. 2 below.

Table No. 2
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 "corrosive" potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the "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.

6.10 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 concrete is to perform testing to determine the sulfates present in the soils. The test results are then compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. 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 test results are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

The 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 provide design parameters.

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 consisted of silty sands with varying amounts of gravel and potential cobbles extending to depths of about 1½ to 13½ feet BSG. The silty sands were underlain by interbedded layers of poorly graded sands with silt, silty sands and silty sands with gravel, well graded sands with silt and gravel, silty gravel with potential cobbles, poorly graded sands, and poorly graded gravel with potential cobbles extending to the maximum depth explored, about 51½ feet BSG. The soil layers described above are also anticipated to contain localized boulders; however, the presence of boulders could not be confirmed due to the small size of the boreholes. Fill soils were not readily distinguishable from the native soils due to the absence of any construction debris and the granular nature of the soils; however, due to previous site grading, fill soils are generally anticipated to occur within the upper about 2 or 3 feet at the site.

7.3 Oversized cobbles and boulders were noted at the site. The USDA soil survey indicates the near surface soils contain 3 to 4 percent cobbles. Due to the coarse gravel, cobble and boulders anticipated within the onsite soils, oversize cobbles and boulders would need to be removed prior to placement of the soils as engineered fill.

7.4 The near surface soils encountered are granular in nature and are not considered expansive.

7.5 Groundwater was not encountered in the test borings drilled at the time of our January 17 and 18, 2018 field exploration to the maximum depth explored, about 51½ feet BSG. Based on our review of water well data on the Department of Water Resources website, groundwater is anticipated at depths greater than 100 feet BSG.

7.6 Based on the results of the percolation tests for the proposed BMP / infiltration areas, and the granular nature and low fines content of the on-site soils, infiltration of storm water appears to be feasible from a geotechnical engineering standpoint, provided the recommendations of this report and the recommendations of the April 5, 2019 report entitled "Results of Double-Ring Infiltrometer Test and Supplemental Recommendations" prepared by Moore Twining are followed.

7.7 Based on our field and laboratory investigation, the near surface soils tested possess a very low expansion potential, low compressibility characteristics, and excellent pavement support characteristics when compacted as engineered fill.

7.8 Due to the previous site grading and demolition and removal of existing subsurface improvements, this report recommends over-excavation of the upper soils to support of the new structures on engineered fill and reduce the potential for excessive differential static settlement. Static settlements of 1 inch total and ½ inch differential in 40 linear feet should be anticipated for foundations supported on engineered fill prepared in accordance with the recommendations of this report.

7.9 Due to the depth of groundwater, liquefaction is not considered a potential hazard at this site. However, seismic settlements were estimated to be about 1 inch total and ½ inch differential in 40 feet.

7.10 Chemical testing of soil samples indicated the soils exhibit a "corrosive" corrosion potential.

7.11 Chemical analyses indicated a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

7.12 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 pipe manufacturer 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 foundation loads. 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 Due to the existing and previous residential development at the site, there is potential to encounter septic systems, former foundations, undocumented fills, etc. during grading.

8.1.5 A demolition plan should be developed to identify the existing improvements (i.e., structures, underground utilities, street improvements, trees, etc.) to be removed.

8.1.6 The onsite soils contain gravel and oversized cobbles and potentially boulders. Thus, contractors should anticipate the need to remove oversized rock prior to placement of engineered fill soils.

8.1.7 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.

8.2 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. 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.

8.2.3 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.

8.2.4 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.

8.2.5 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.

8.2.6 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

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.

Recommendations for stormwater infiltration systems for the project are included in the report prepared by Moore Twining entitled, "Results of Double-Ring Infiltrometer Test and Supplemental Recommendations, Storm Water Infiltration System, Proposed Residence at Casa Loma, Southeast of East Lugonia Avenue and Occidental Drive, Redlands, San Bernardino County, California," dated April 5, 2019. In addition to the recommendations included therein, concrete cutoffs/collars should be included at inlet and outlet pipes connecting to infiltration systems (if any) to reduce seepage from migrating along trenches.

8.4 Site Preparation

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 planned excavation. 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 Where trees are to be removed, all roots larger than ¼ inch in diameter and any accumulation of organic matter that will result in an organic content more than 3 percent by weight should be removed and not used as engineered fill. After verification of removal of all roots and organic materials, the bottom of the excavation should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations. Moore Twining should be contacted to observe removal of the tree roots.

8.4.3 Existing underground utilities, including utilities and associated structures within Crystal Court within areas of proposed improvements which are sensitive to settlement should be removed and backfilled with engineered fill. In addition, all existing trench backfill soils and materials should be removed and the excavations should be backfilled with 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.

8.4.4 During site preparation, all existing surface and subsurface structures will need to be removed, including foundations, utilities, septic systems, etc. 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.

8.4.5 After site stripping, removal of root systems and removal of existing surface and subsurface improvements, areas of proposed residential structures and all foundations should be over-excavated to at least 36 inches below preconstruction site grades, to a minimum of 12 inches below the bottom of the footings, to the depth to remove undocumented fill soils, and to at least 12 inches below the bottom of existing improvements 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 optimum to three (3) percent above optimum moisture content and compacted as engineered fill.

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

8.4.7 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.

8.4.8 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 bottom of the footings, to the depth to remove undocumented fill soils, and to at least 12 inches below the bottom of existing improvements to be removed, whichever is greater.

8.4.9 After approval of the pool / spa excavations 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.

8.4.10 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 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 between optimum and three (3) percent above 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.

8.4.11 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; to the depth required to remove existing undocumented fills; and to at least 12 inches below subsurface improvements (structures, utilities, etc.) 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.

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

8.4.13 The contractor should locate all on-site water wells (if any). 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.

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

8.4.15 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.

8.4.16 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, poorly graded sands with silt, silty gravel, and poorly graded gravel. The onsite soils are anticipated to include varying amounts of cobbles and potentially boulders. The USDA soil survey indicates the near surface soils contain 3 to 4 percent cobbles. Recycled materials including asphalt 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, provided the oversize rock is removed from the soils. Oversized rock and irreducible materials greater than 6 inches in dimension should be removed from the soils prior to use as engineered fill. Gravel, cobbles and irreducible materials (i.e., asphalt, concrete, brick) should not be nested in fill materials. 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	> 5,000 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 between optimum moisture content and three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. 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. 4
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 Conventional Shallow Spread Foundations

8.6.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on the estimated settlements. The following static 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) a total seismic settlement of 1 inch, and 4) a differential seismic settlement of ½ inch in 40 feet.

8.6.2 Foundations supported on subgrade 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.

8.6.3 All foundations for two- and three-story structures should have a minimum depth of 18 inches below the lowest adjacent grade. All foundations for single-story structures should have a minimum depth of 12 inches below the lowest adjacent grade. In addition, all footings for the new buildings should have a minimum width of 15 inches, regardless of load.

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

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 34.06875 degrees and a longitude of -117.16805 degrees. The data provided in Table No. 5 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. 5 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. 5

Seismic Factor	2019 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.875g

TABLE NO. 5

Seismic Factor	2019 CBC Value
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration (PGA)	0.796g
Spectral Response At Short Period (0.2 Second), Ss	1.87
Spectral Response At 1-Second Period, S_1	0.763
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}	1.87
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}	1.247
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 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.

8.6.8 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 350 pounds per square foot per foot of depth to a maximum of 3,500 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.

8.6.9 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.

8.6.10 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 <u>Frictional Coefficient and Earth Pressures</u>

8.7.1 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 0.40 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.

8.7.2 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 350 pounds per cubic foot. The upper 6 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.7.3 The active and at-rest pressures of the 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.

8.7.4 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.

8.7.5 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.

8.7.6 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.

8.8 Retaining Walls / Screen Walls

8.8.1 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.

8.8.2 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 3,000 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

8.8.3 Retaining walls should be constructed with imported 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.

8.8.4 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.

8.8.5 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.

8.8.6 The earth pressures provided in this report (Section 8.7) 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.

8.8.7 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.

8.8.8 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.

8.8.9 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.

8.9 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.

8.9.1 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.

8.9.2 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.

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

8.9.4 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The vapor retarder should overlay the compacted aggregate base. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the aggregate base and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. 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 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 selection and 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 the floor covering and floor covering adhesive 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.

8.9.5 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.

8.9.6 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.

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

8.9.8 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.

8.9.9 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.

8.9.10 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.

8.9.11 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.

8.10 <u>Exterior Slabs-On-Grade and Concrete Pool / Spa Decking</u>

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.

8.10.1 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.

8.10.2 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.

8.10.3 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.

8.10.4 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.

8.10.5 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.

8.11 <u>In-Ground Swimming Pool / Spa</u>

8.11.1 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.

8.11.2 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.

8.11.3 If the subgrade is prepared, and then disturbed by equipment workers, weather or other source, we 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.

8.11.4 Due to the granular nature of the onsite soils, excavations for the pool / spa 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.

8.11.5 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 foundations.

8.12 Asphaltic Concrete (AC) Pavements

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

8.12.1 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.

8.12.2 The following pavement sections are based on an R-value of 50 and traffic index values ranging from 5.0 to 7.0 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. 6
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	4.0	12
5.5	3.0	4.0	12
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)

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

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

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

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

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

8.12.8 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.

8.12.9 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.

8.12.10 The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. The Contractor shall provide an asphalt concrete mix design prepared and signed by a California registered civil engineer and approved by Moore Twining prior to construction.

8.13 Portland Cement Concrete (PCC) Pavements

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

8.13.1 The subgrade soils for Portland cement concrete pavements should be over-excavated and compacted as recommended in the "Site Preparation" section of the recommendations in this report.

8.13.2 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. 7
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)	6.5	4.0	12.0
1 truck per day (7 trucks per week)	7.0	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)

8.13.3 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.

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

8.13.5 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.

8.13.6 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.

8.13.7 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.

8.13.8 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.

8.13.9 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.

8.13.10 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.

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

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

8.14 Slopes and Temporary Excavations

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

8.14.2 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.

8.14.3 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.

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

8.14.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the 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.

8.15 Utility Trenches

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

8.15.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned 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.

8.15.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in Table No. 8 (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. 8
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

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

8.15.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned to between optimum and three (3) percent above 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.

8.15.6 On-site soils and approved imported engineered fill may be used as final backfill (12 inches above the pipe to the ground surface) in trenches

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

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

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

8.15.10 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.

8.15.11 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.

8.16 <u>Corrosion Protection</u>

8.16.1 The analytical results of sample analyses indicate the samples had resistivity values of 4,736 and 4,135 ohms-centimeter, with pH values of 7.4 and 7.5, respectively. Based on the resistivity values and the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 2 of section 6.9 of this report, the soils exhibit a "corrosive" corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "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.

8.16.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (0.0027 and 0.0011 percent by dry weight concentrations of sulfate). According to provisions of ACI 318, section 4.3 , the sulfate concentration falls in the negligible classification (0.00 to 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.

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

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 afforded the opportunity 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 The Planning Associates Group. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

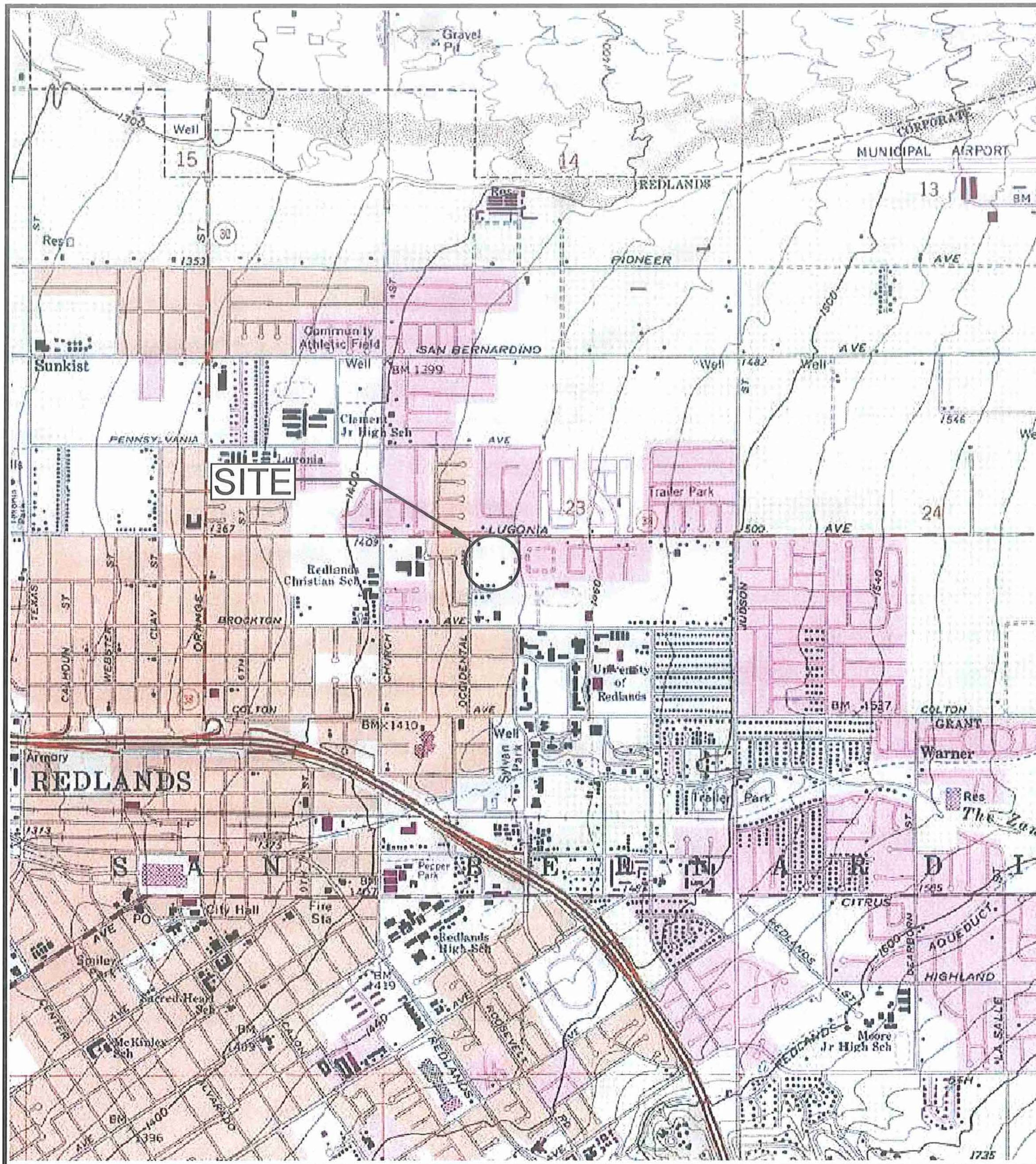
Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Allen H. Harker, PG
Professional Geologist

Read L. Andersen, RGE
Manager

APPENDIX A

DRAWINGS

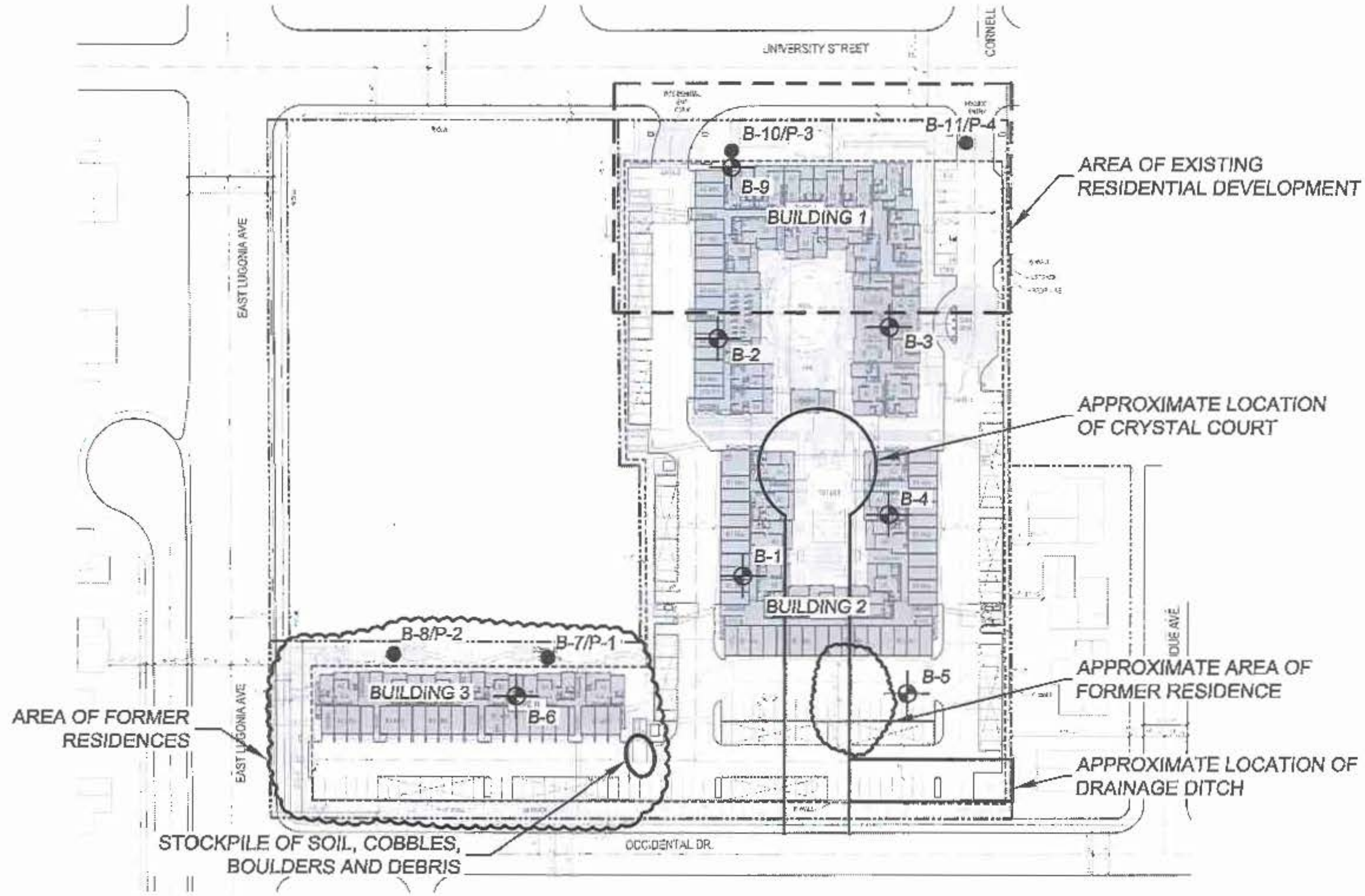


SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
REDLANDS, CALIFORNIA QUADRANGLE 1996

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED MULTIFAMILY RESIDENCE AT CASA LOMA SOUTHEAST OF INTERSECTION OF EAST LUGONIA AVENUE AND OCCIDENTAL DRIVE REDLANDS, CALIFORNIA	FILE NO.: 57101-01-01	DATE: 01/24/18
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. G57101.01	DRAWING NO. 1

MOORE TWINING ASSOCIATES, INC.



APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

APPROXIMATE LOCATION OF EXISTING RESIDENTIAL DEVELOPMENT

TEST BORING, PERCOLATION TEST AND CPT LOCATION MAP PROPOSED MULTIFAMILY RESIDENCE AT CASA LOMA SOUTHEAST OF INTERSECTION OF EAST LUGONIA AVENUE AND OCCIDENTAL DRIVE REDLANDS, CALIFORNIA	FILE NO. 57101-01-02	DATE DRAWN: 11/26/19
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. G57101.01	DRAWING NO. 2

MOORE TWINING ASSOCIATES, INC.

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of borings. 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 locations. Also, the passage of time may result in changes in the soil conditions at these test boring 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.

Since 1898

Test Boring: B-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

Drill Type: CME 75

Elevation: 1,432 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1432 — 0	2/6 18/6 20/6	SM	SILTY SAND; very loose, moist, fine to coarse grained, brown, with rootlets At 0.5 feet - Becomes dense At 1 foot - With quartz rock fragments		38	6.8
1430 — 2	13/6 18/6 22/6		Medium dense, with a little fine gravel	DD = 116.1 pcf From 2-5' pH = 7.4 SR = 4,736 ohm-cm SS = 0.0027% Cl = 0.00077%	40	4.7
1428 — 4	14/6 11/6 18/6		Grayish-brown, trace fine to coarse gravel, decrease in fines content		29	
1426 — 6						
1424 — 8	15/6 18/6 20/6		Dense, increase in fine to coarse gravel content, with potential cobbles		38	
1422 — 10						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

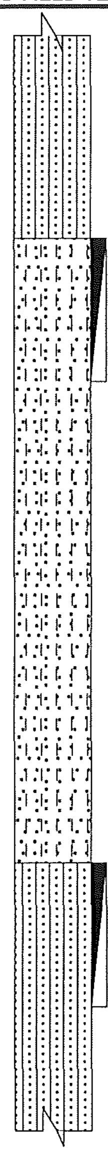
Drill Type: CME 75

Elevation: 1,432 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1420 — 12						
1418 — 14	16/6 18/6 18/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, fine to coarse grained, grayish-brown, with a little fine to coarse gravel		36	
1416 — 16						
1414 — 18						
1412 — 20	10/6 14/6 20/6	SM	SILTY SAND; dense, damp, fine to coarse grained, brown, with some fine gravel		34	
1410 — 22						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

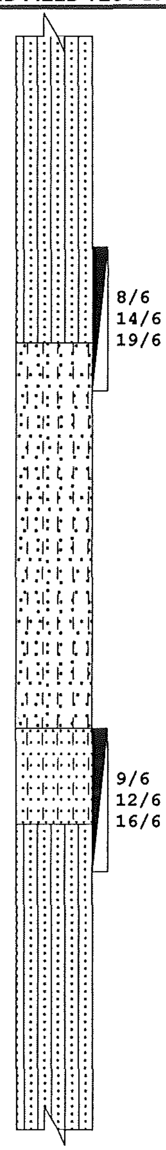
Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,432 feet AMSL

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 %
1408 — 24						
	8/6 14/6 19/6		Decrease in fines content and gravel, trace fine gravel, grayish-brown		33	
1406 — 26		SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, fine to coarse grained, grayish-brown, trace fine gravel			
1404 — 28						
1402 — 30	9/6 12/6 16/6	SW-SM	WELL GRADED SAND WITH SILT AND GRAVEL; medium dense, fine to coarse, grayish-brown, with sand, and with some interbedded silty sand lenses	From 30-31.5: Gravel=15.4% Sand = 76.3% -200 = 8.3%	28	
		SM	SILTY SAND; medium dense, damp, fine to coarse grained, grayish-brown, with trace fine gravel			
1400 — 32						
1398 — 34						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,432 feet AMSL

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 %
	9/6 12/6 18/6		Increase in gravel content, with a little fine gravel	From 35-36.5': Gravel = 9.8% Sand = 77.1% -200 = 13.1%	30	
1396 — 36						
1394 — 38						
1392 — 40	12/6 11/6 15/6		With some fine gravel	From 40-41.5': Gravel = 3.5% Sand = 84.2% -200 = 12.3%	26	
1390 — 42						
1388 — 44						
			Dense, decrease in fines content, increase in fine gravel content		39	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

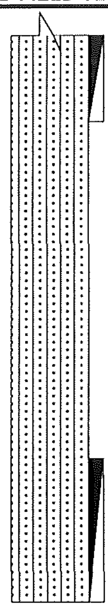
Drill Type: CME 75

Elevation: 1,432 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1386 — 46	9/6 18/6 21/6					
1384 — 48						
1382 — 50	15/6 38/6 42/6		Very dense, with some fine gravel and trace coarse gravel		80	
			Bottom of Boring B-1 at 51.5 feet			
1380 — 52						
1378 — 54						
1376 — 56						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-2

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

Drill Type: CME 75

Elevation: 1,434 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1434 — 0	6/6 7/6 6/6	SM	SILTY SAND; medium dense, moist, fine grained, brown, trace medium to coarse sand and fine gravel		13	7.6
1432 — 2						
1430 — 4	50/4		Very dense, fine to coarse grained, light brown, with a little fine gravel and some coarse gravel rock fragments and potential cobbles		>50	
1428 — 6	28/6 42/6 50/3	GM	SILTY GRAVEL; very dense, damp, fine to coarse, grayish-brown, with sand and potential cobbles	DD = 123.7 pcf	>92	2.5
1426 — 8						
1424 — 10	7/6 9/6 13/6	SM	SILTY SAND; medium dense, damp, fine to medium grained, grayish-brown, with some coarse sand and fine gravel		22	
		SP-SM				

Notes:

Figure Number

Since 1898

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,434 feet AMSL

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 %
1422 — 12 1420 — 14			POORLY GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, grayish-brown, with a little fine gravel			
	8/6 16/6 17/6	SP	POORLY GRADED SAND; dense, damp, fine to coarse grained, grayish-brown, with some fine to coarse gravel		33	
		CL	LEAN CLAY; hard, moist, low plasticity, brown, high silt content (3 inch thick layer)			
1418 — 16 1416 — 18		SM	SILTY SAND; dense, damp, fine to coarse grained, grayish-brown, with some fine to coarse gravel			
1414 — 20	12/6 26/6 24/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, fine to coarse grained, grayish-brown, with some fine to coarse gravel		50	
1412 — 22			Bottom of Boring B-2 at 21.5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-3

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

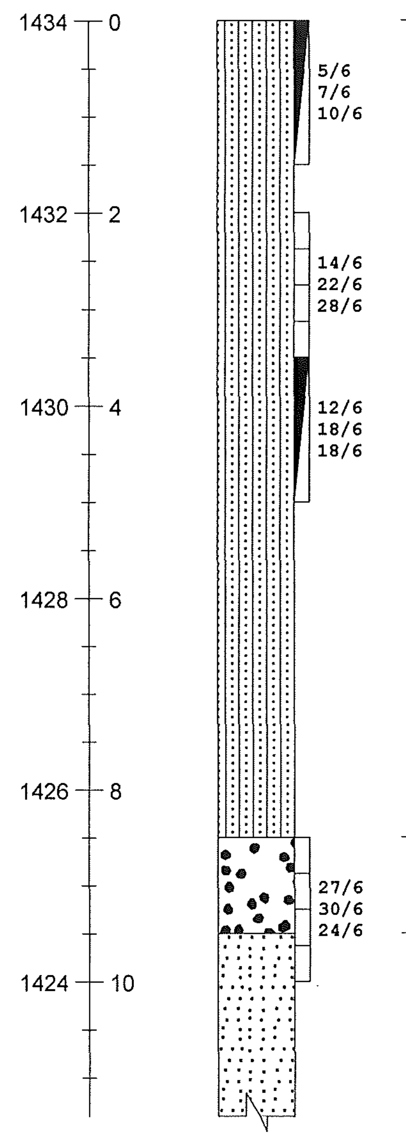
Drill Type: CME 75

Elevation: 1,434 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1434 / 0	5/6, 7/6, 10/6	SM	SILTY SAND; medium dense, moist, fine grained, brown, trace fine gravel, with some fine to coarse grained lenses		17	6.5
1432 / 2	14/6, 22/6, 28/6		Dense, fine to coarse grained, with fine gravel, coarse gravel rock fragments and potential cobbles	DD = 120.8 pcf	50	3.1
1430 / 4	12/6, 18/6, 18/6		Decrease in fines content, grayish-brown		36	
1426 / 8.5	27/6, 30/6, 24/6	GP	POORLY GRADED GRAVEL; dense, damp, fine to coarse, grayish- brown, with sand and potential cobbles	DD = 122.2 pcf	54	2.3
1424 / 9.5		SP	POORLY GRADED SAND; dense, damp, fine grained, grayish-brown, trace medium to coarse sand and fine gravel			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-3

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,434 feet AMSL

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 %
1422 / 12						
1420 / 14	6/6 11/6 16/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, grayish-brown, with some fine gravel		27	
1418 / 16						
1416 / 18	9/6 12/6 12/6		Increase in gravel content, with some fine to coarse gravel		24	
1414 / 20			Bottom of Boring B-3 at 20 feet			
1412 / 22						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 17, 2018

Drill Type: CME 75

Elevation: 1,432 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1432 / 0		SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, with trace coarse sand and some fine gravel	From 0-5' Gravel=12.4% Sand = 72% -200 = 15.6% EI = 0 DD = 112.1 pcf		
	12/6 13/6 15/6				28	7.1
1430 / 2						
	5/6 5/6 5/6		Loose		10	
1428 / 4						
	9/6 13/6 11/6	GP	POORLY GRADED GRAVEL; medium dense, damp, fine to coarse, grayish-brown, with sand and cobbles	DD = 114.8 pcf	24	1.4
1426 / 6						
1424 / 8						
1422 / 10	21/6 19/6 15/6	GM	SILTY GRAVEL; dense, damp, fine to coase, grayish-brown, with sand and cobbles		34	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,432 feet AMSL

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 %
1420 — 12						
1418 — 14						
	13/6 13/6 9/6	SM	SILTY SAND; medium dense, damp, fine to coarse grained, grayish-brown, with a little fine gravel		22	
1416 — 16						
1414 — 18						
1412 — 20	7/6 8/6 14/6		Brown		22	
			Bottom of Boring B-4 at 21.5 feet			
1410 — 22						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-5

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,430 feet AMSL

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 %
1430 — 0	3/6 7/6 9/6	SM	SILTY SAND; medium dense, moist, fine grained, brown, trace medium to coarse sand and fine to coarse gravel		16	6.2
1428 — 2						
1426 — 4	21/6 27/6 42/6		Dense, fine to medium grained, with a little fine gravel	DD = 117.4 pcf ø = 43° c = 100 psf	69	4.0
1424 — 6						
1422 — 8	8/6 12/6 11/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to medium grained, grayish-brown, trace coarse sand and fine gravel		23	
1420 — 10						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-5

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 17, 2018

Elevation: 1,430 feet AMSL

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 %
1418 — 12						
1416 — 14	9/6 7/6 12/6				19	
1414 — 16						
1412 — 18	10/6 10/6 14/6		Fine to coarse grained, trace fine gravel		24	
1410 — 20			Bottom of Boring B-5 at 20 feet			
1408 — 22						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-6

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 18, 2018

Elevation: 1,432 feet AMSL

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 %
1432 — 0	4/6 4/6 5/6	SM	SILTY SAND; loose, moist, fine to medium grained, brown, trace fine gravel	From 0-5' EI = 0	9	5.8
1430 — 2	5/6 6/6 13/6		Medium dense	DD = 107.3 pcf	18	6.2
		SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, light brown, trace fine gravel			
1428 — 4						
	13/6 13/6 21/6		With potential cobbles	DD = 123.0 pcf	34	2.2
1426 — 6						
1424 — 8						
	17/6 18/6 28/6	SM	SILTY SAND WITH GRAVEL; dense, damp, fine to coarse grained, brown, grayish-brown, with potential cobbles		46	
1422 — 10						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: B-6

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 18, 2018

Elevation: 1,432 feet AMSL

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 %
1420 — 12						
1418 — 14	13/6 15/6 12/6		Medium dense		27	
1416 — 16						
1414 — 18	8/6 12/6 13/6		SILTY SAND; fine grained, trace fine gravel, brown		25	
1412 — 20		SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, grayish-brown, with a little fine gravel			
			Bottom of Boring B-6 at 20 feet			
1410 — 22						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: B-7/P-1

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 18, 2018

Elevation: 1,432-1/2 feet AMSL

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 %
1432 — 0	6/6 6/6 8/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace coarse sand and fine gravel		14	6.0
	6/6 7/6 7/6		Decrease in gravel content		14	
1430 — 2		SP	POORLY GRADED SAND; medium dense, damp, fine to coarse grained, light brown, trace fine gravel			
	6/6 8/6 15/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown, trace coarse sand and fine gravel		23	
1428 — 4		SP	POORLY GRADED SAND; medium dense, damp, fine to coarse grained, light brown, trace fine gravel			
			Bottom of Percolation Test Boring B-7/P-1 at 5 feet			
1426 — 6						
1424 — 8						
1422 — 10						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-8/P-2

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 18, 2018

Drill Type: CME 75

Elevation: 1,432-1/2 feet AMSL

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip Hammer

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 %
1432 — 0	1/6 2/6 3/6	SM	SILTY SAND; loose, moist, fine to medium grained, brown, trace coarse sand and fine gravel		5	5.4
1430 — 2	4/6 4/6 3/6	SP	POORLY GRADED SAND; loose, moist, fine to medium grained, light brown, trace coarse sand and with some fine to coarse gravel	From 1.5-3': Gravel=11.0% Sand = 85.0% -200 = 4.0%	7	
			Bottom of Percolation Test Boring B-8/P-2 at 3 feet			
1428 — 4						
1426 — 6						
1424 — 8						
1422 — 10						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-9

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 18, 2018

Drill Type: CME 75

Elevation: 1,439 feet AMSL

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

Hammer Type: 140 Pound Auto Trip Hammer

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 1438		SM	SILTY SAND; moist, fine grained, brown, with trace fine gravel			3.9
2 1436		SP-SM	POORLY GRADED SAND WITH SILT; moist, fine to coarse grained, light brown, trace fine gravel Trace fine to coarse gravel Increase in fine gravel			
4	16/6 14/6 39/6	SM	SILTY SAND; dense, moist, fine to medium grained, brown, with a little fine to coarse gravel, low fines content	DD = 116.3 pcf	53	2.7
1434 6 1432 8 1430	16/6 21/6 24/6		SILTY SAND WITH GRAVEL; increaes in fines content and fine to coarse gravel content, potential cobbles		45	
10 1428	9/6 19/6 19/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, fine to coarse grained, grayish-brown, with some fine to coarse gravel		38	

Notes: *Hand augered the upper 3.5 feet due to potential utility conflicts*

Figure Number

MOORE TWINING ASSOCIATES, INC. Since 1898

Test Boring: B-9

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 18, 2018

Elevation: 1,439 feet AMSL

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 %
1424 / 15	12/6 16/6 17/6	SM	SILTY SAND; dense, damp, fine grained, grayish-brown, trace medium to coarse sand and fine to coarse gravel	Gravel stuck in shoe of sampler, little recovery	33	
20	10/6 17/6 16/6				33	
21.5			Bottom of Boring B-9 at 21.5 feet			

Notes: *Hand augered the upper 3.5 feet due to potential utility conflicts*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-10/P-3

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Drilled By: J.T.

Drill Type: CME 75

Auger Type: 8" O.D. Hollow Stem Augers

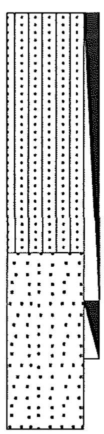
Hammer Type: 140 Pound Auto Trip Hammer

Logged By: A.H.

Date: January 18, 2018

Elevation: 1,439 feet AMSL

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		SM	SILTY SAND; moist, fine grained, brown, trace fine gravel			5.2
1438						
2						
1436		SP	POORLY GRADED SAND; damp, fine to coarse grained, light brown, with some fine gravel and trace coarse gravel	From 3-3.6': Gravel=10.6% Sand = 85.0% -200 = 4.4%		
4						
1434			Bottom of Percolation Test Boring B-10/P-3 at 4.3 feet			
6						
1432						
8						
1430						
10						
1428						

Notes: *Hand augered the upper 3.6 feet due to potential utility conflicts*

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-11/P-4

Project: Proposed Multi-Family Residential Development in Redlands

Project Number: G57101.01

Logged By: A.H.

Drilled By: J.T.

Date: January 18, 2018

Drill Type: CME 75

Elevation: 1,439 feet AMSL

Auger Type: 8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip Hammer

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		SM	SILTY SAND; moist, fine grained, brown, trace fine gravel	From 0-2.5': pH = 7.5 SR = 4,135 ohm-cm SS = 0.0011% CL = 0.0011%		6.8
3.75			Bottom of Percolation Test Boring B-11/P-4 at 3.75 feet			

Elevation/depth scale: 0, 1438, 2, 1436, 4, 1434, 6, 1432, 8, 1430, 10, 1428

Notes: *Hand augered the upper 2.5 feet due to potential utility conflicts. Could not hand auger past coarse gravel at 2.5 feet.*

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Silty sand
	Poorly graded sand with silt
	Well graded sand with silt
	Silty gravel
	Poorly graded sand
	Lean clay
	Poorly graded gravel

Symbol	Description
Misc. Symbols	
	Boring continues
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler
	Bulk/Grab sample

Notes:

1. Exploratory borings were drilled on 1/17/18 and 1/18/18 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. Elevations are noted from Topographic Survey, dated February 14, 2017, prepared by RdM Surveying, Inc. Elevations were interpolated to the nearest 1/2 foot.

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
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

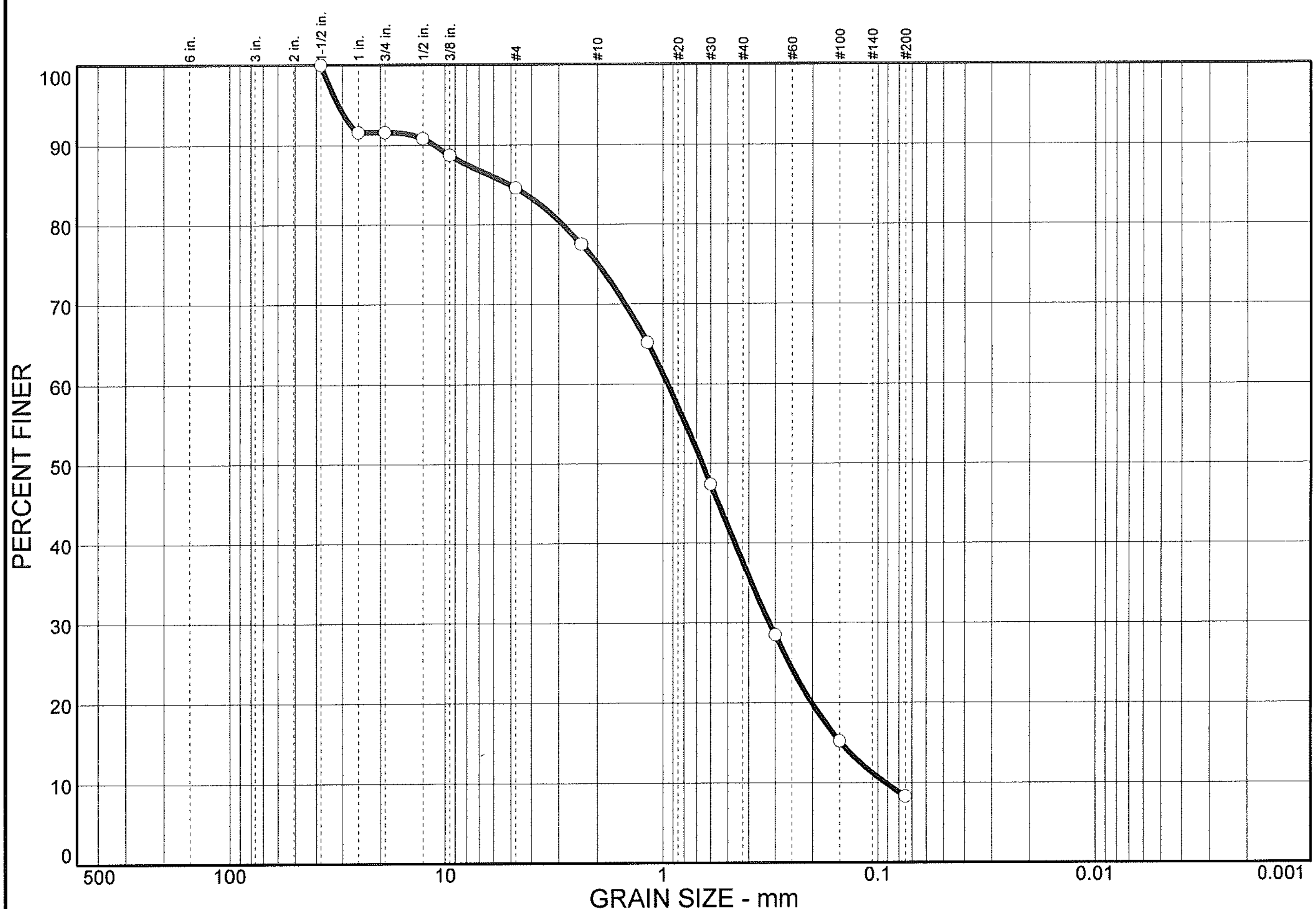
APPENDIX C

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 D2216)	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 D1125)	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	15.4	76.3	8.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	91.6		
3/4 in.	91.6		
1/2 in.	90.8		
3/8 in.	88.7		
#4	84.6		
#8	77.5		
#16	65.2		
#30	47.4		
#50	28.5		
#100	15.2		
#200	8.3		

* (no specification provided)

Material Description

Well-graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 5.05 D_{60}= 0.951 D_{50}= 0.658
D_{30}= 0.319 D_{15}= 0.148 D_{10}= 0.0922
C_u= 10.31 C_c= 1.16

Classification

USCS= SW-SM AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 30-31.5'

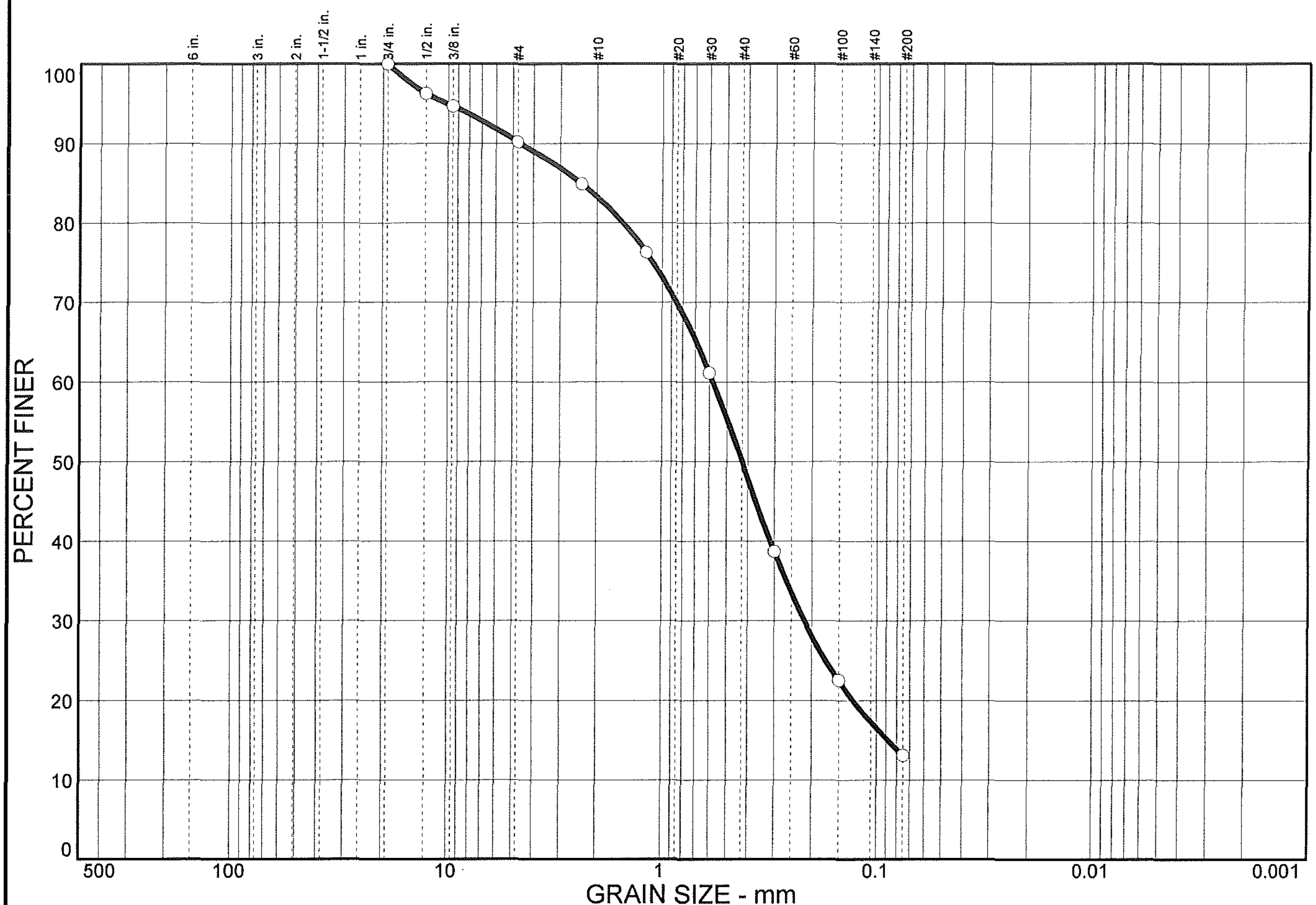
Moore Twining Associates, Inc.

Fresno, CA

Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands

Project No: G57101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	9.8	77.1	13.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	96.3		
3/8 in.	94.7		
#4	90.2		
#8	84.9		
#16	76.3		
#30	61.1		
#50	38.7		
#100	22.5		
#200	13.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.39 D_{60}= 0.578 D_{50}= 0.425
D_{30}= 0.217 D_{15}= 0.0881 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 35-36.5'

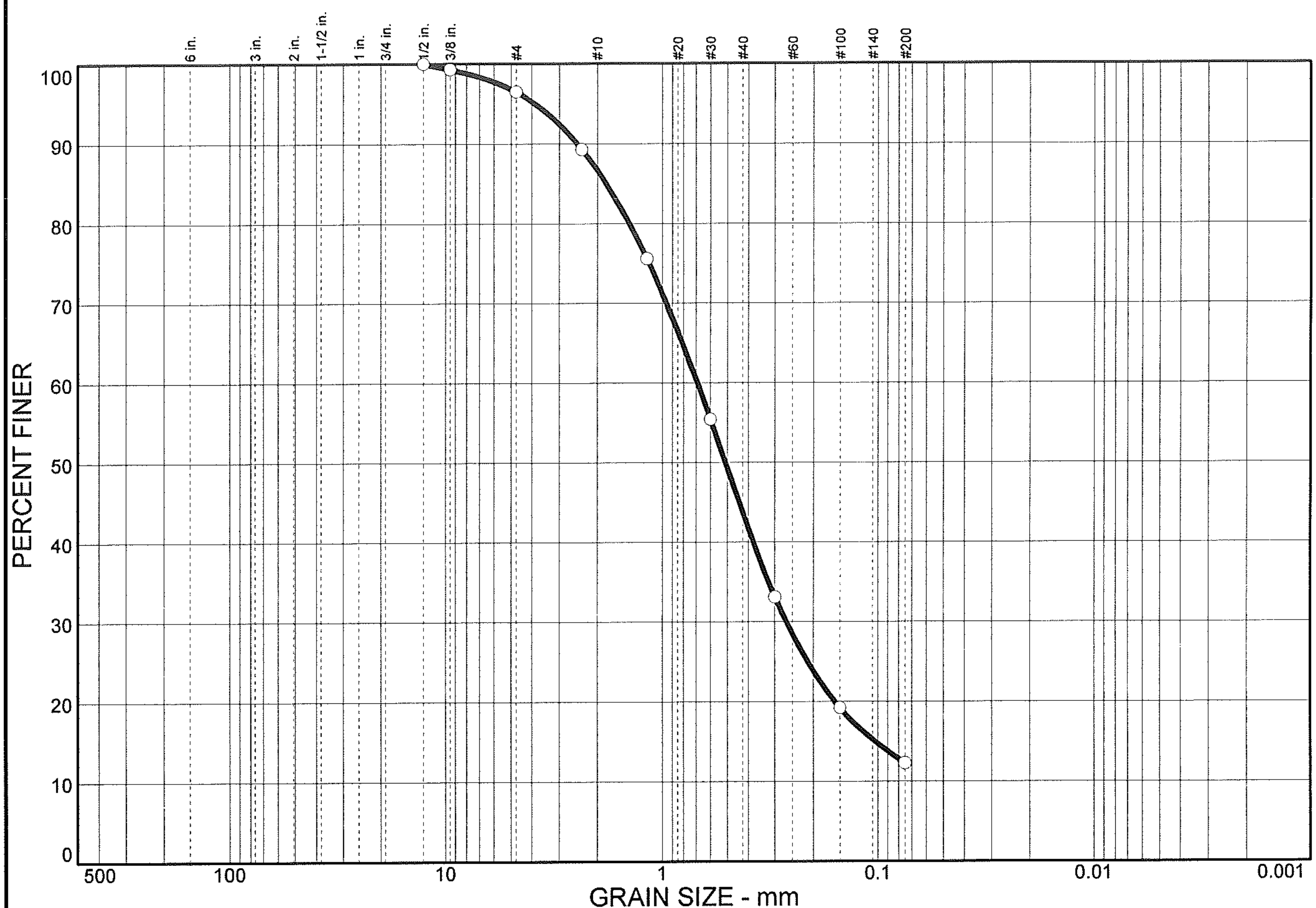
Moore Twining Associates, Inc.

Fresno, CA

Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands

Project No: G57101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.5	84.2	12.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.3		
#4	96.5		
#8	89.3		
#16	75.6		
#30	55.4		
#50	33.1		
#100	19.2		
#200	12.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.82 D_{60}= 0.690 D_{50}= 0.511
D_{30}= 0.266 D_{15}= 0.103 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

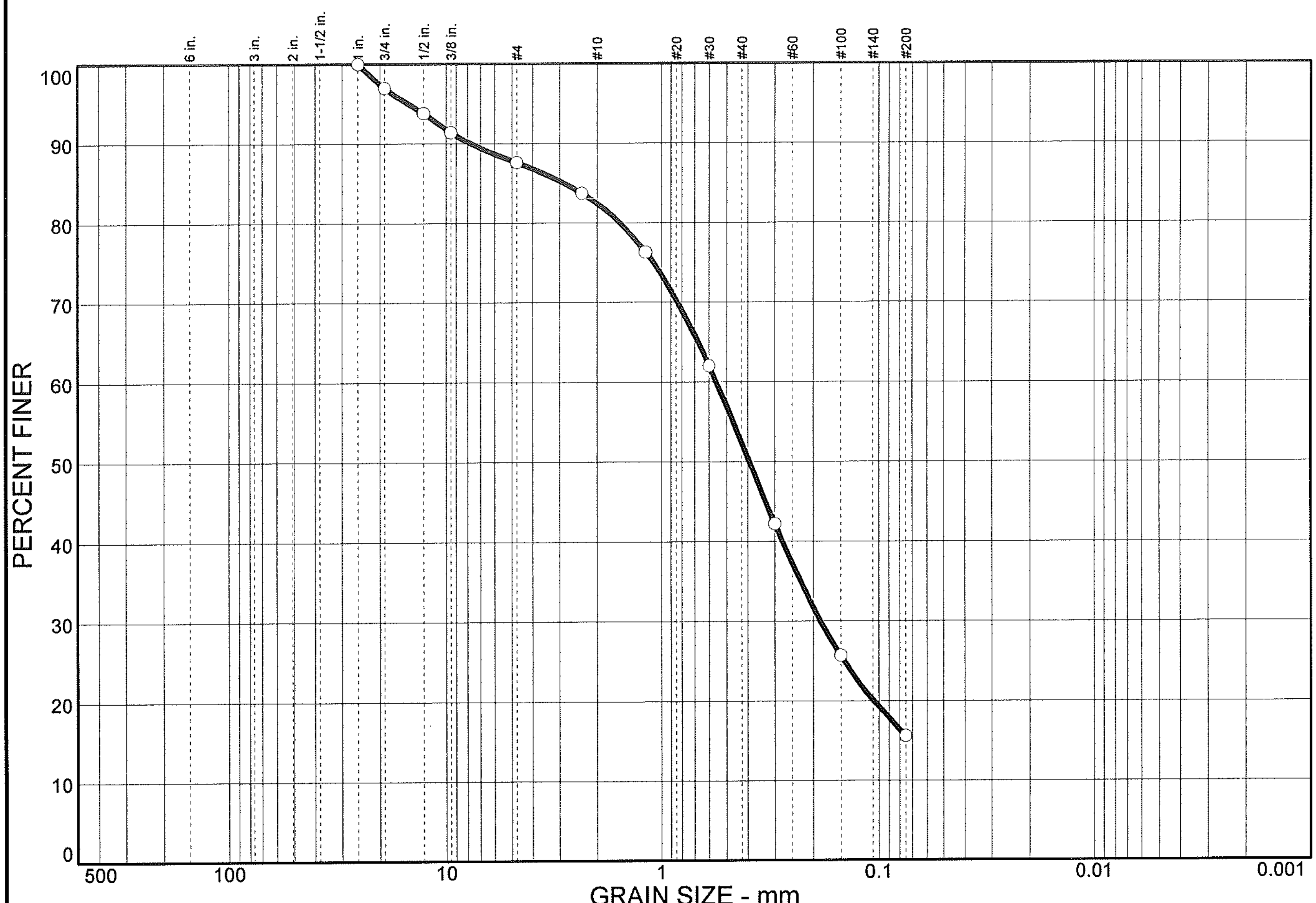
Sample No.: B-1 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 40-41.5'

Moore Twining Associates, Inc.

Fresno, CA

Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands
Project No: G57101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	12.4	72.0	15.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	97.0		
1/2 in.	93.8		
3/8 in.	91.4		
#4	87.6		
#8	83.7		
#16	76.3		
#30	62.0		
#50	42.2		
#100	25.7		
#200	15.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.87 D_{60}= 0.557 D_{50}= 0.393
D_{30}= 0.185 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 0-5'

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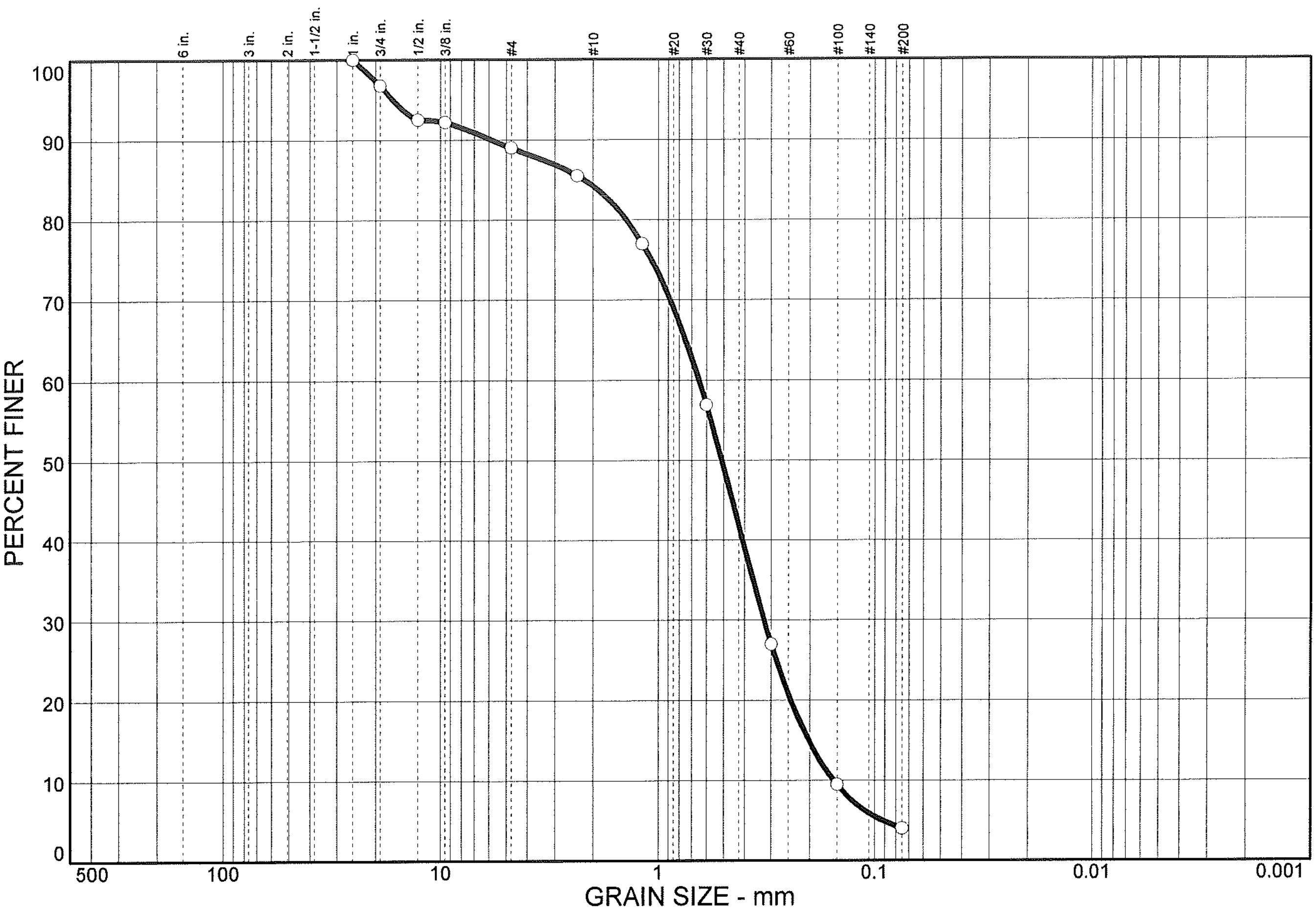
Fresno, CA

Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands

Project No: G57101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	11.0	85.0	4.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
3/4 in.	96.8		
1/2 in.	92.5		
3/8 in.	92.2		
#4	89.0		
#8	85.5		
#16	77.0		
#30	56.9		
#50	27.0		
#100	9.5		
#200	4.0		

* (no specification provided)

Material Description

Poorly graded sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.20 D_{60}= 0.650 D_{50}= 0.510
D_{30}= 0.324 D_{15}= 0.202 D_{10}= 0.155
C_u= 4.19 C_c= 1.04

Classification

USCS= SP AASHTO=

Remarks

Sample No.: B-8/P-2 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 1.5-3'

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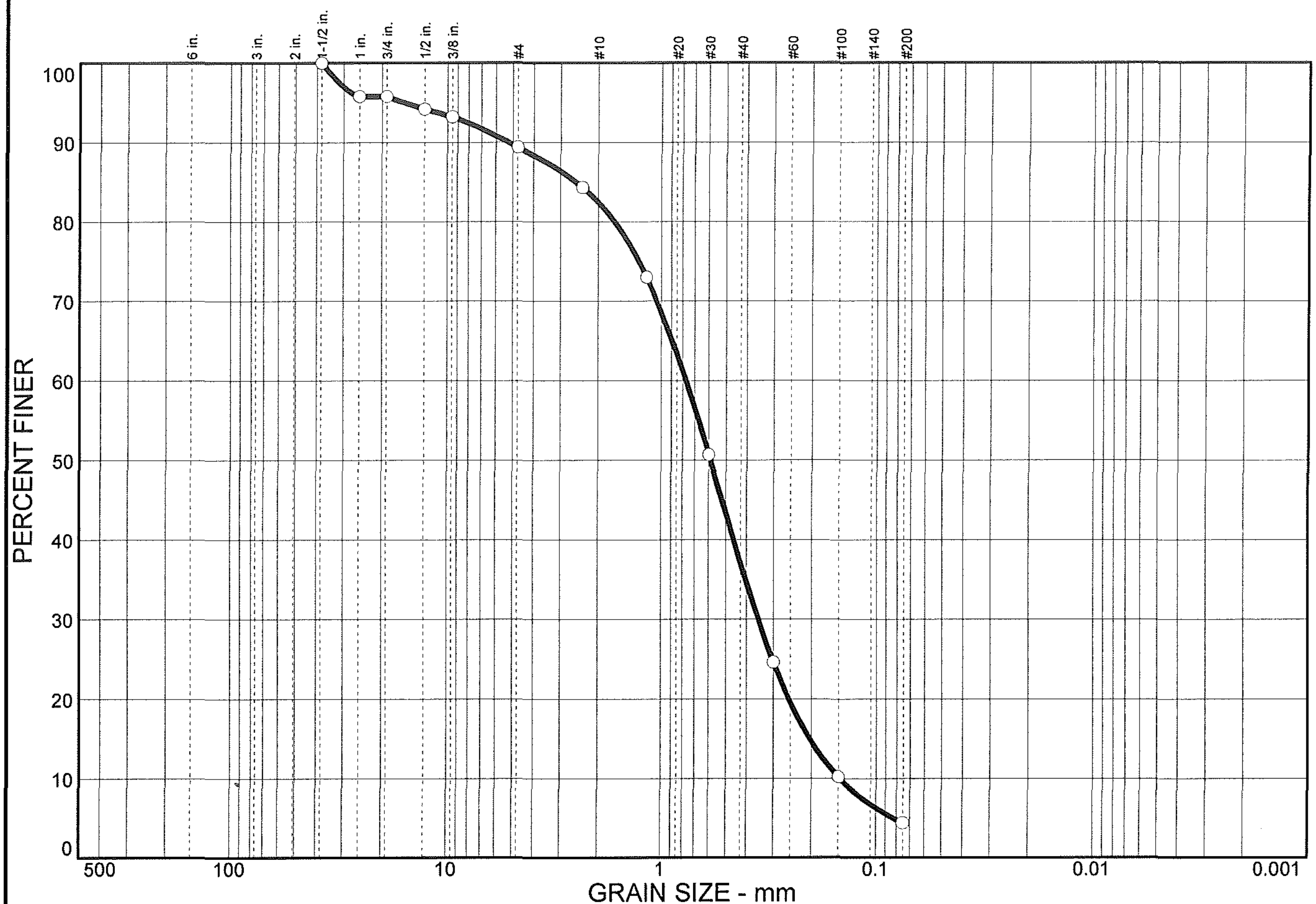
Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands

Project No: G57101.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	10.6	85.0	4.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2 in.	100.0		
1 in.	95.8		
3/4 in.	95.8		
1/2 in.	94.2		
3/8 in.	93.2		
#4	89.4		
#8	84.3		
#16	73.0		
#30	50.7		
#50	24.6		
#100	10.2		
#200	4.4		

* (no specification provided)

Material Description

Poorly graded sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.54 D_{60}= 0.770 D_{50}= 0.589
D_{30}= 0.353 D_{15}= 0.204 D_{10}= 0.148
C_u= 5.22 C_c= 1.09

Classification

USCS= SP AASHTO=

Remarks

Sample No.: B-10/P-3 **Source of Sample:** **Date:** 1/17/18
Location: **Elev./Depth:** 3-3.6'

Moore Twining Associates, Inc.

Fresno, CA

Client: The Planning Associates Group
Project: Proposed Multi-Family Residential Development, Redlands
Project No: G57101.01 **Figure**

MOORE TWINING ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Multi-Family Residential Development, Redlands	REPORT DATE:	1/25/2018
		TEST DATE:	1/23/2018
MTA PROJECT NO.:	G57101.01		
SAMPLE I.D.:	B-4 @ 0-5'		
SAMPLED BY:	AH		
SAMPLE DATE:	1/17/2018	TESTED BY:	PV

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	1.0098
Pan + Dry Soil Wt., gm	233.2	Dry Soil Wt., lbs	0.8865
Pan Wt., gm	0.0		
Initial % Moisture Content	7.2	Final % Moisture Content	13.9

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9504	Ring + Sample Wt., lbs	1.0098
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9504	Remolded Wt., lbs	1.0098
Remolded Wet Density, pcf	130.7	Remolded Wet Density, pcf	138.9
Remolded Dry Density, pcf	121.9	Remolded Dry Density, pcf	121.9

Initial Volume	Final Volume
0.00727222	0.007271

Expansion Data:	
Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.0499
Expansion, in:	-0.0001
Expansion Index	0

Comments: **Vey 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

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EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Multi-Family Residential Development, Redlands	REPORT DATE:	1/25/2018
		TEST DATE:	1/23/2018
MTA PROJECT NO.:	G57101.01		
SAMPLE I.D.:	B-6 @ 0-5'		
SAMPLED BY:	AH		
SAMPLE DATE:	1/17/2018	TESTED BY:	PV

MATERIALS DESCRIPTION: Silty sand with poorly graded sand mix

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	1.0101
Pan + Dry Soil Wt., gm	233.2	Dry Soil Wt., lbs	0.8869
Pan Wt., gm	0.0		
Initial % Moisture Content	7.2	Final % Moisture Content	13.9

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9508	Ring + Sample Wt., lbs	1.0101
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9508	Remolded Wt., lbs	1.0101
Remolded Wet Density, pcf	130.7	Remolded Wet Density, pcf	138.9
Remolded Dry Density, pcf	122.0	Remolded Dry Density, pcf	122.0

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007272
Initial Gage Reading, in:	0.0500		
Final Gage Reading, in:	0.0500		
Expansion, in:	0.0000		
Expansion Index	0	**Comments:**	**Vey 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

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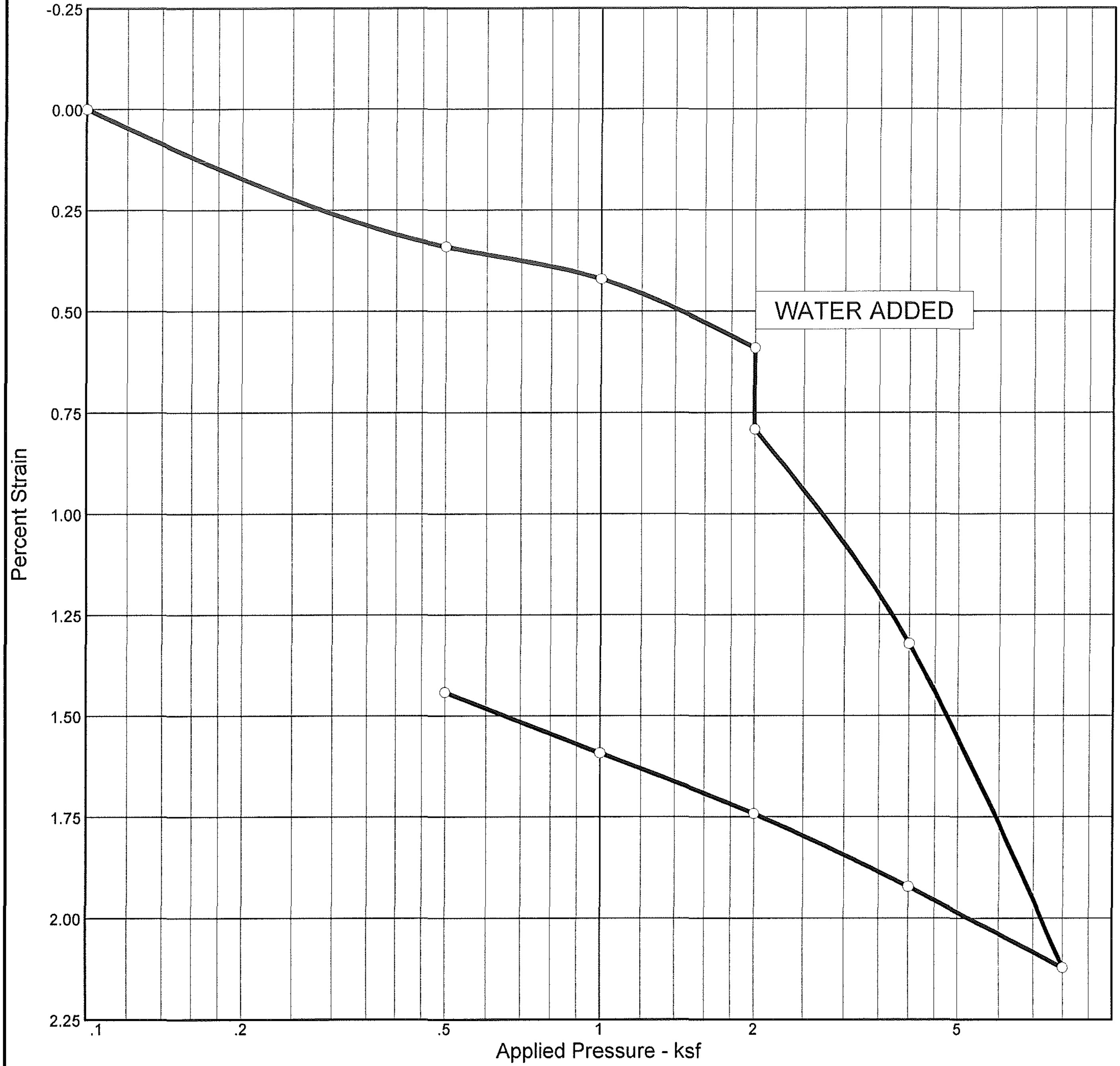
CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
26.2 %	4.1 %	116.6			2.65		2.02	0.04	0.01		0.2	0.419

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

Source: **Sample No.:** B-1 **Elev./Depth:** 2-3.5'

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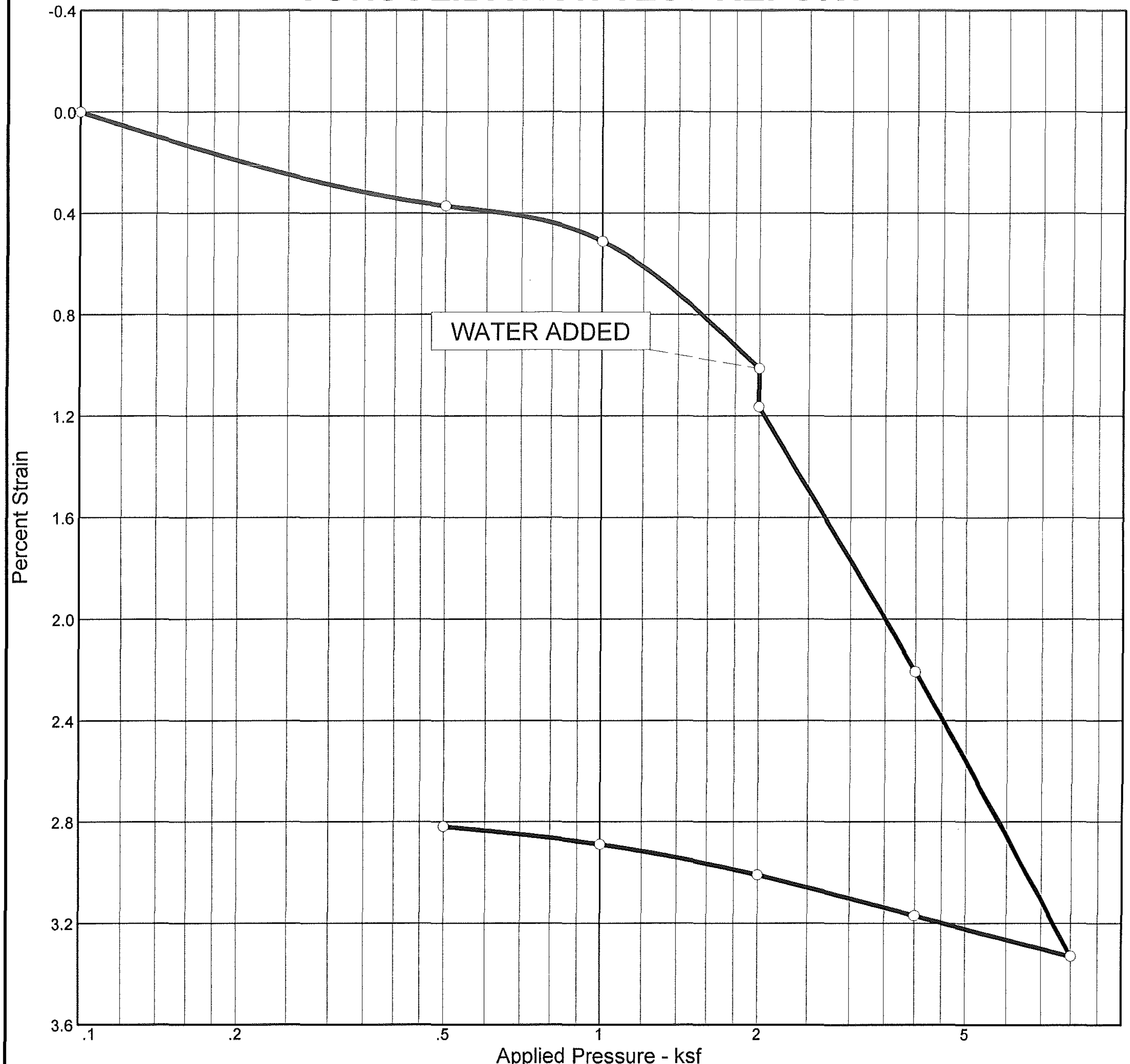
Remarks:

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.											
9.4 %	1.6 %	113.7			2.65		1.50	0.05	0.01		0.2	0.455

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

Source: **Sample No.:** B-3 **Elev./Depth:** 2-3.5'

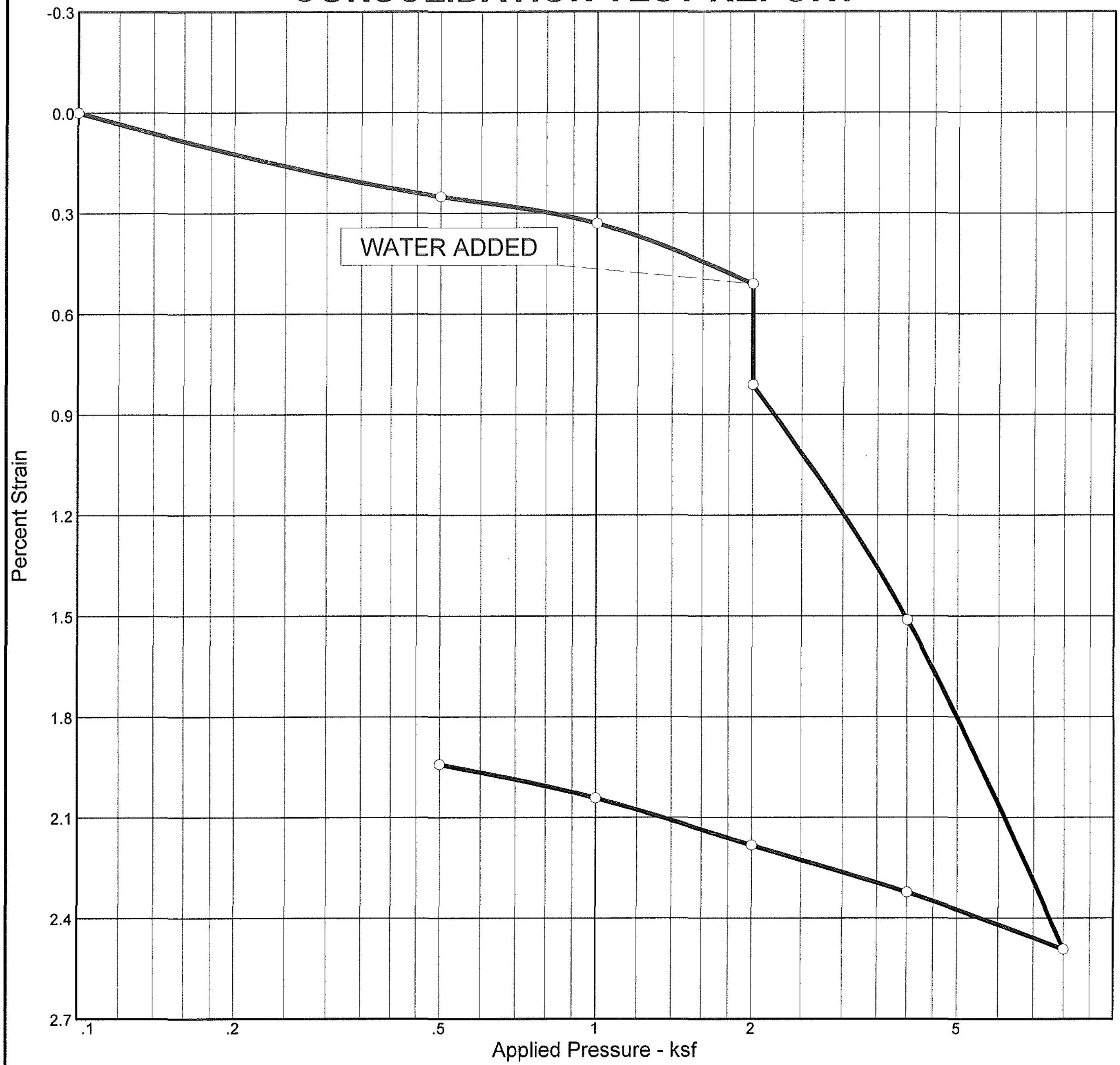
Moore Twining Associates, Inc.

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.											
24.1 %	3.8 %	117.0			2.65		1.88	0.05	0.01		0.3	0.415

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

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

Remarks:

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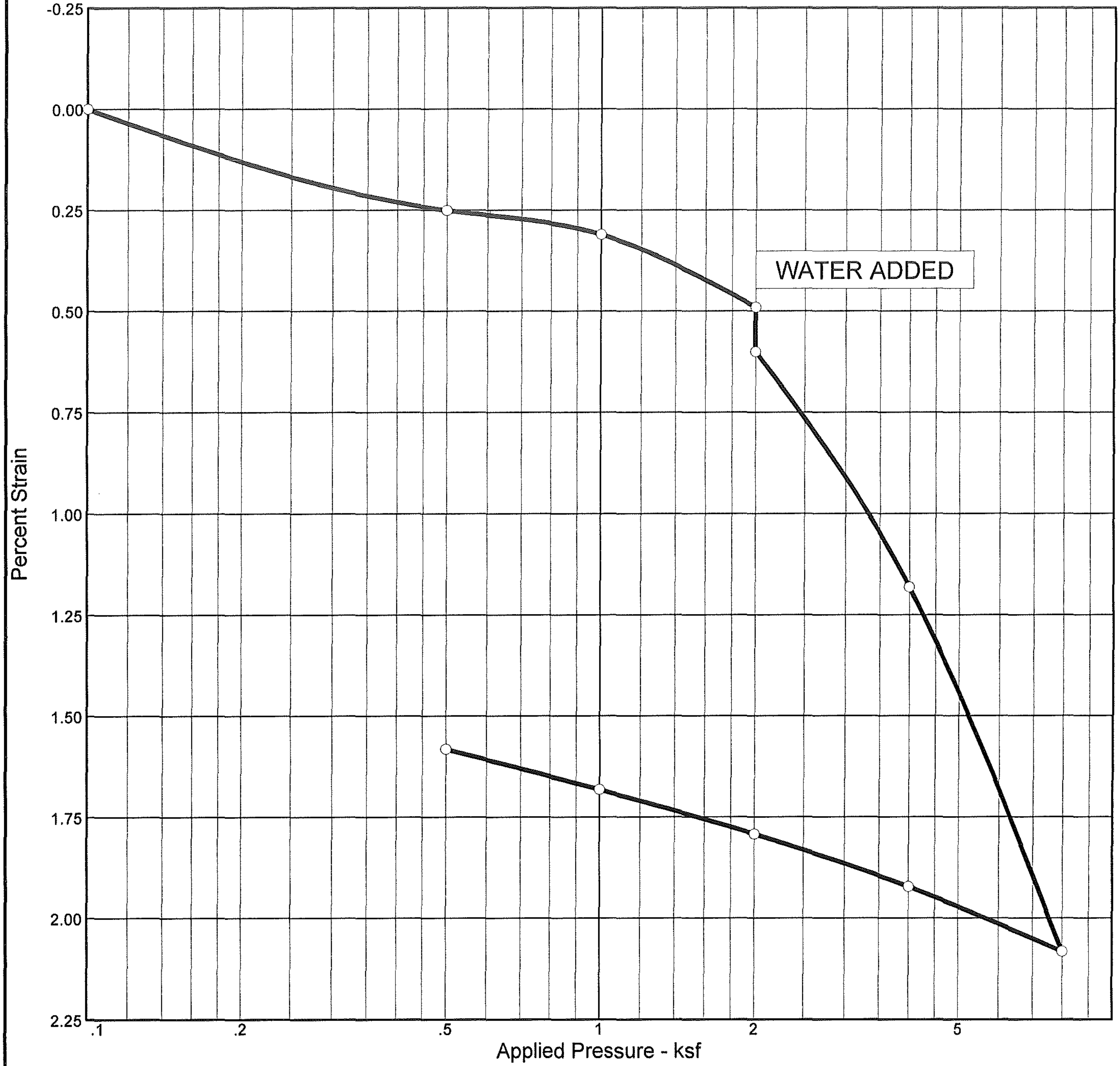
Figure

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.											
23.4 %	4.7 %	107.9			2.65		2.25	0.05	0.01		0.1	0.533

MATERIAL DESCRIPTION	USCS	AASHTO
Poorly graded sand with silt	SP-SM	

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

Source: **Sample No.:** B-6 **Elev./Depth:** 2-3.5'

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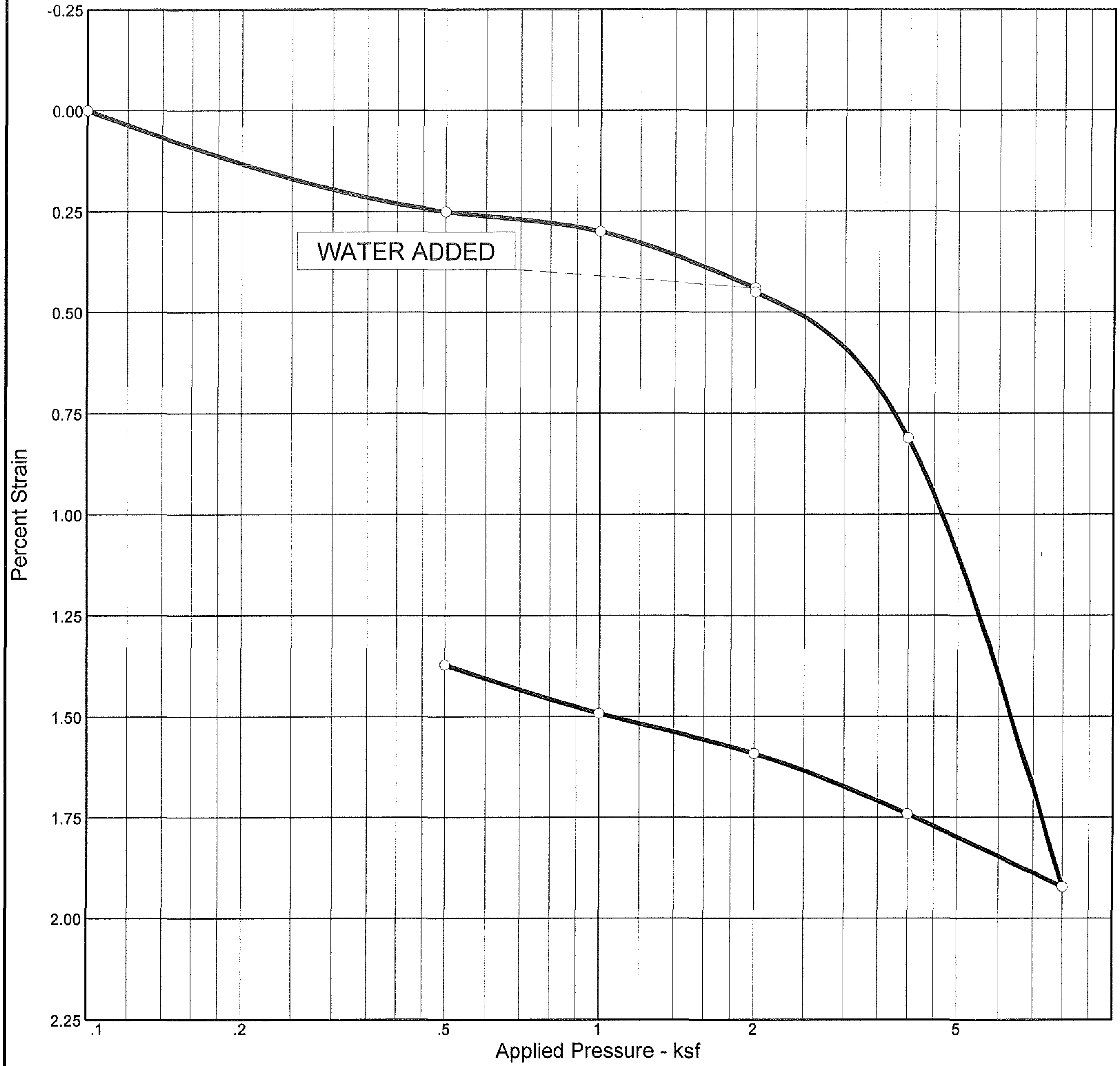
Remarks:

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.											
22.1 %	3.6 %	115.5			2.65		3.87	0.06	0.01		0.1	0.432

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

Source: **Sample No.:** B-9 **Elev./Depth:** 3.5-5'

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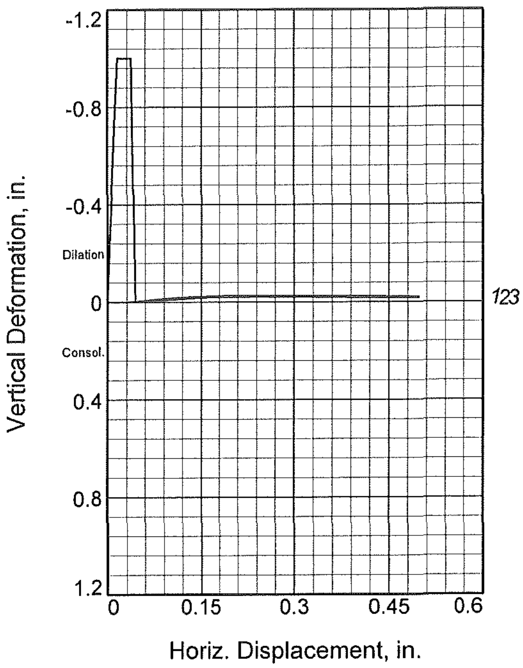
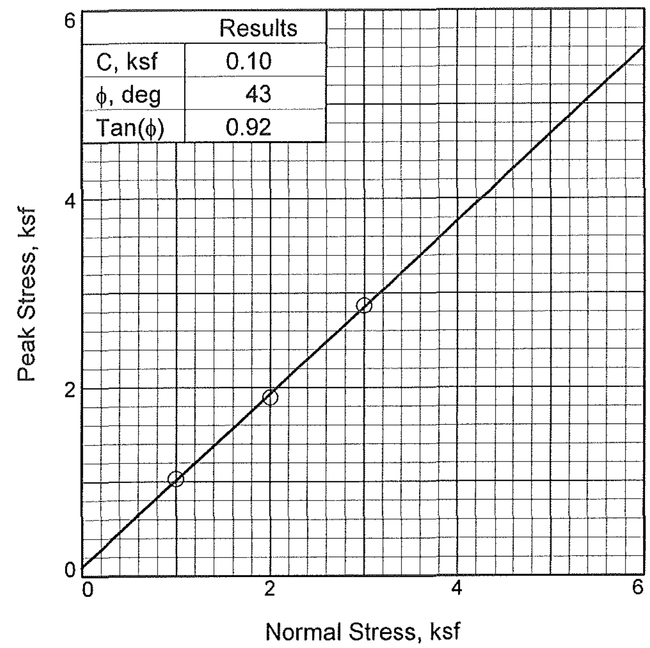
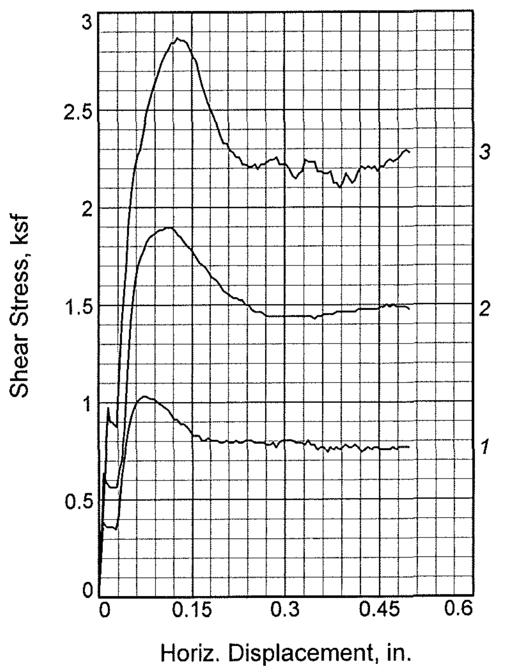
Remarks:

Figure







	Sample No.	1	2	3
Initial	Water Content, %	4.0	4.5	3.5
	Dry Density, pcf	116.4	118.9	116.5
	Saturation, %	25.2	30.3	22.1
	Void Ratio	0.4210	0.3912	0.4200
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	15.2	14.3	14.4
	Dry Density, pcf	117.0	119.0	118.2
	Saturation, %	97.2	97.4	95.4
	Void Ratio	0.4143	0.3899	0.4001
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	0.99
Normal Stress, ksf		1.00	2.00	3.00
Peak Stress, ksf		1.03	1.90	2.87
Displacement, in.		0.07	0.11	0.13
Residual Stress, ksf				
Displacement, in.				
Strain at peak, %		2.9	4.4	5.3

Sample Type:
Description: Silty sand

Specific Gravity= 2.65
Remarks:

Figure ________

Client: The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

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

Proj. No.: G57101.01 **Date Sampled:** 1/17/18

DIRECT SHEAR TEST REPORT
Moore Twining Associates, Inc.
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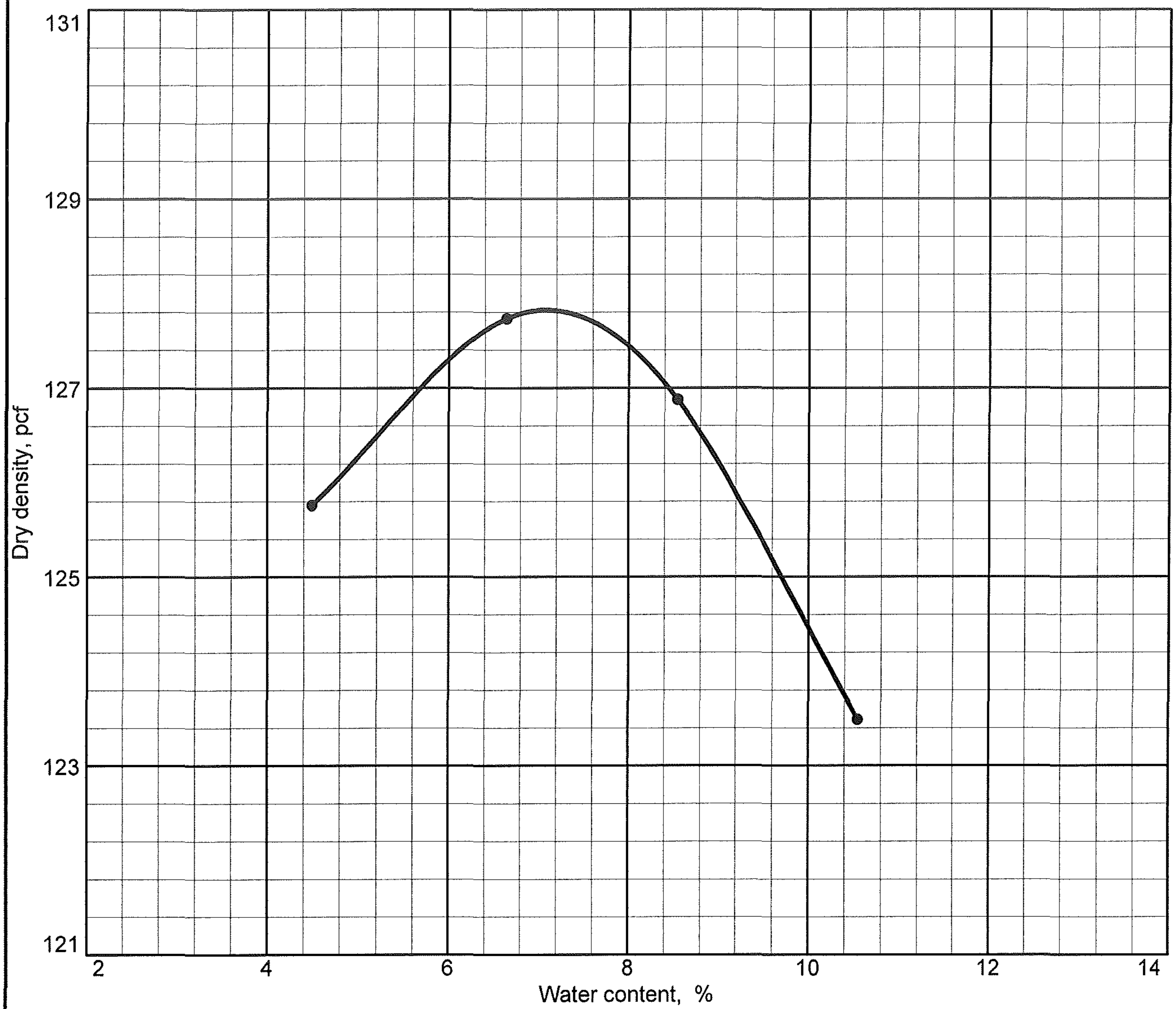
COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
0-5'	SM						8.6	15.6

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 127.8 pcf Optimum moisture = 7.1 %	Silty sand

Project No. G57101.01 **Client:** The Planning Associates Group

Project: Proposed Multi-Family Residential Development, Redlands

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 0-5'

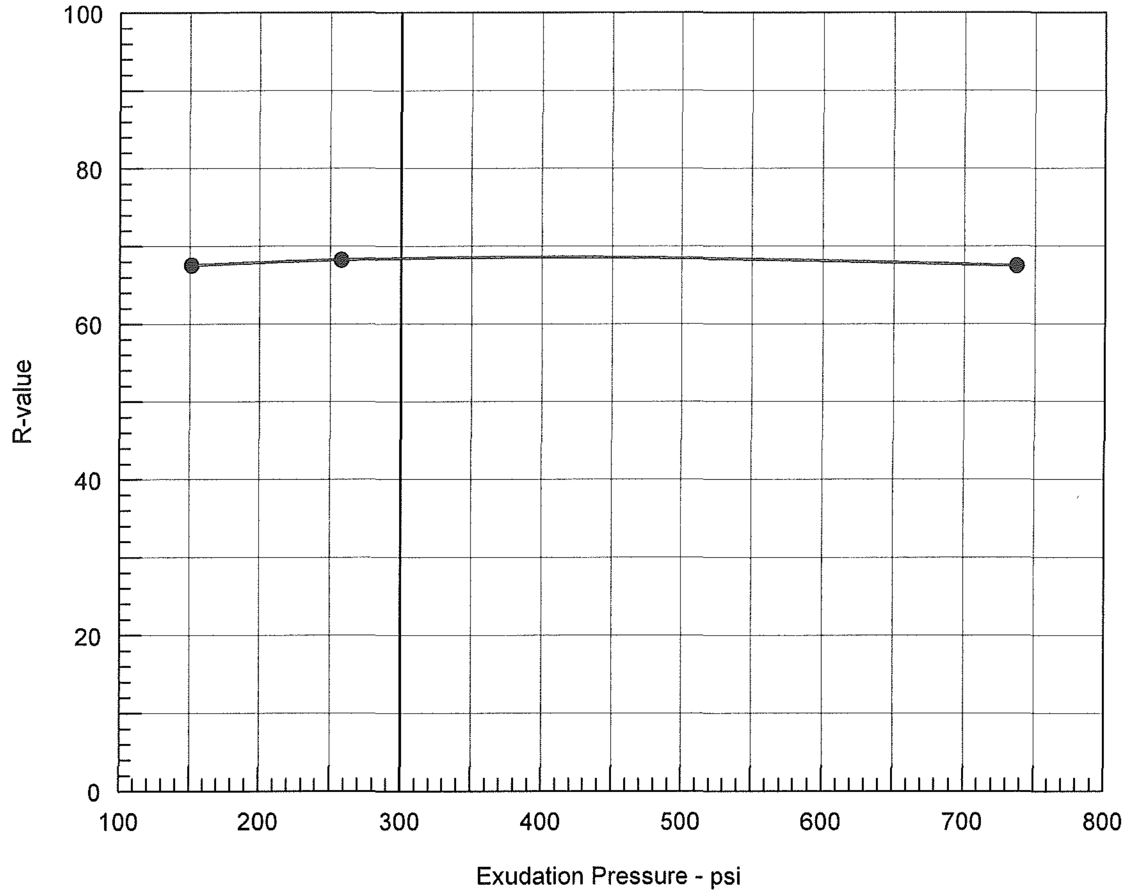
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	128.0	8.5	0.00	30	2.48	151	68	68
2	350	130.5	7.5	0.00	26	2.44	737	69	68
3	350	126.3	8.0	0.00	28	2.46	258	68	68

Test Results	Material Description
R-value at 300 psi exudation pressure = 68	Silty sand

Project No.: G57101.01

Project: Proposed Multi-Family Residential Development, Redlands

Sample Number: B-2 **Depth:** 0-3.5'

Date: 1/26/2018

Tested by:

Checked by:

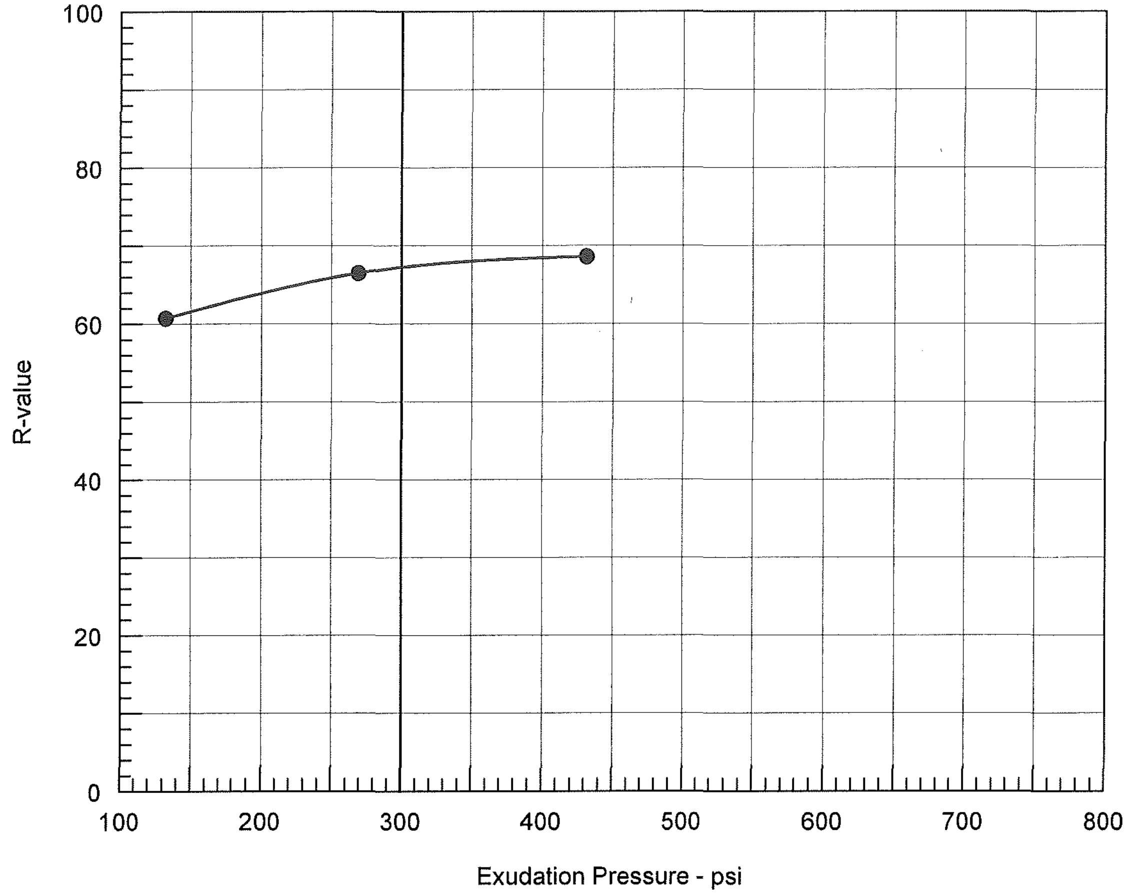
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

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	126.5	9.1	0.00	35	2.52	132	61	61
2	350	126.1	8.1	0.00	28	2.50	431	69	69
3	350	125.7	7.6	0.00	29	2.51	269	66	66

Test Results	Material Description
R-value at 300 psi exudation pressure = 67	Silty sand with poorly graded sand mix

Project No.: G57101.01

Project: Proposed Multi-Family Residential Development, Redlands

Sample Number: B-6 **Depth:** 0-5'

Date: 1/26/2018

Tested by:

Checked by:

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure ______

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(559) 268-0740 Fax

January 30, 2018

Work Order #: EA22018

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

RE: Proposed Multi-Family Resid. Development-Redland

Enclosed are the analytical results for samples received by our laboratory on 01/22/18 . For your reference, these analyses have been assigned laboratory work order number EA22018.

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.

Julio Morales
Client Services Supervisor

MOORE TWINING ASSOCIATES, INC.

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MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Multi-Family Resid. Development-Redland
Project Number: G57101.01
Project Manager: Allen Harker

Reported:
01/30/18 11:48

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B-1 @ 2 - 5	EA22018-01	Soil	01/17/18 00:00	01/22/18 11:40
B-11/ P-4 @ 0 - 2.5	EA22018-02	Soil	01/18/18 00:00	01/22/18 11:40

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MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Multi-Family Resid. Development-Redland
Project Number: G57101.01
Project Manager: Allen Harker

Reported:
01/30/18 11:48

B-1 @ 2 - 5

EA22018-01 (Soil)   Sampled:01/17/18 00:00

Analyte	Notes.	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**7.7**	6.0	mg/kg	3	U8A2422	01/24/18	01/25/18	ASTM D4327-84
Chloride		**0.00077**	0.00060	% by Weight	3	[CALC]	01/25/18	01/25/18	ASTM D4327-84
Sulfate as SO4		**0.0027**	0.00060	% by Weight	3	[CALC]	01/25/18	01/25/18	ASTM D4327-84
pH		**7.4**	0.10	pH Units	1	U8A2422	01/24/18	01/26/18	ASTM D4972-89 Mod
Sulfate as SO4		**27**	6.0	mg/kg	3	U8A2422	01/24/18	01/25/18	ASTM D4327

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California ELAP Certificate #1371

MTA Geotechnical Division	Project: Proposed Multi-Family Resid. Development-Redland	
2527 Fresno Street	Project Number: G57101.01	**Reported:**
Fresno CA, 93721	Project Manager: Allen Harker	01/30/18 11:48

B-11/ P-4 @ 0 - 2.5

EA22018-02 (Soil) Sampled:01/18/18 00:00

Analyte	Notes.	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**11**	6.0	mg/kg	3	U8A2422	01/24/18	01/25/18	ASTM D4327-84
Chloride		**0.0011**	0.00060	% by Weight	3	[CALC]	01/25/18	01/25/18	ASTM D4327-84
Sulfate as SO4		**0.0011**	0.00060	% by Weight	3	[CALC]	01/25/18	01/25/18	ASTM D4327-84
pH		**7.5**	0.10	pH Units	1	U8A2422	01/24/18	01/26/18	ASTM D4972-89 Mod
Sulfate as SO4		**11**	6.0	mg/kg	3	U8A2422	01/24/18	01/25/18	ASTM D4327

Notes and Definitions

ug/L	micrograms per liter (parts per billion concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
mg/L	milligrams per Liter (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.

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Project Name:	Proposed Multi-Family Residential Development, Redlands	Report Date:	1/25/2018
		Sample Date:	1/17/2018
Project Number:	G57101.01		
		Sampled By:	AH
Subject:	Minimum Resistivity, ASTM G187	Tested By:	PV
Material Description:	Silty sand	Test Date:	1/23/2018
Location:	B-1 @ 2-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	35,351
100 mls	24,012
150 mls	5,936
200 mls	5,003
250 mls	4,736
300 mls	4,802

Remarks: Min. Resistivity is 4,736 Ohm-cm

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MOORE TWINING ASSOCIATES, INC.

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Project Name:	Proposed Multi-Family Residential Development, Redlands	Report Date:	1/25/2018
		Sample Date:	1/17/2018
Project Number:	G57101.01		
		Sampled By:	AH
Subject:	Minimum Resistivity, ASTM G187	Tested By:	PV
Material Description:	Silty sand	Test Date:	1/23/2018
Location:	B-11/P-4 @ 0-2.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	23,345
100 mls	6,670
150 mls	4,536
200 mls	4,135
250 mls	4,202

Remarks: Min. Resistivity is 4,135 Ohm-cm

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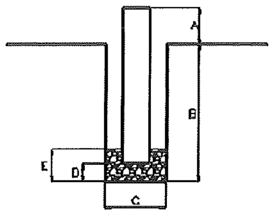
APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-7/P-1

Project:	Proposed Multi-Family Residential Development, Redland	**Project No.**	G57101.01
Location:	SEC of East Lugonia Avenue and Occidental Drive, Redlands, California	**Test Date:**	1/19/2018



A. Top of Pipe Above Ground	27.5	Inches
B. Depth of Hole	60	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	4	Inches
E. Total Gravel Layer Depth	24	Inches
F. Pipe Length	83.5	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: 5 gallons to 22.7 inches from bottom
Checked Presoak Water Dropped 6 inches Twice in a Row in Less than 30 Minutes

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	1/19/2018	7:33:37	5.4				
Presoak	1/19/2018	7:34:06	5.9	0.5	6	0.2	26.8
2	1/19/2018	7:34:06	5.9				
Presoak	1/19/2018	7:35:04	6.4	1	6	0.4	18.5
3	1/19/2018	7:36:57	5.45				
Start Test	1/19/2018	7:37:57	6.14	1	8.28	0.3	19.4
4	1/19/2018	7:37:57	6.14				
	1/19/2018	7:38:57	6.5	1	4.32	0.6	14.8
5	1/19/2018	7:39:50	5.4				
	1/19/2018	7:40:50	6.07	1	8.04	0.3	18.2
6	1/19/2018	7:40:50	6.07				
	1/19/2018	7:41:50	6.45	1	4.56	0.6	14.9
7	1/19/2018	7:42:35	5.35				
	1/19/2018	7:43:35	6.05	1	8.4	0.3	18.7
8	1/19/2018	7:43:35	6.05				
	1/19/2018	7:44:35	6.4	1	4.2	0.6	13.3
9	1/19/2018	7:45:19	5.35				
	1/19/2018	7:46:19	6.01	1	7.92	0.3	17.4
10	1/19/2018	7:46:19	6.01				
	1/19/2018	7:47:19	6.38	1	4.44	0.6	13.7
11	1/19/2018	7:48:09	5.5				
	1/19/2018	7:49:09	6.09	1	7.08	0.4	16.6
12	1/19/2018	7:49:09	6.09				
	1/19/2018	7:50:09	6.43	1	4.08	0.6	13.3
13	1/19/2018	7:50:57	5.33				
	1/19/2018	7:51:57	6	1	8.04	0.3	17.5
14	1/19/2018	7:51:57	6				
	1/19/2018	7:52:57	6.34	1	4.08	0.6	12.4
15	1/19/2018	7:53:46	5.3				
	1/19/2018	7:54:46	5.92	1	7.44	0.3	15.7

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-7/P-1

Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/19/2018

16	1/19/2018	7:54:46	5.92				
	1/19/2018	7:55:46	6.3	1	4.56	0.6	13.2
17	1/19/2018	7:56:37	5.37				
	1/19/2018	7:57:37	6.05	1	8.16	0.3	18.2
18	1/19/2018	7:57:37	6.05				
	1/19/2018	7:58:37	6.38	1	3.96	0.6	12.4
19	1/19/2018	7:59:25	5.3				
	1/19/2018	8:00:25	5.92	1	7.44	0.3	15.7
20	1/19/2018	8:00:25	5.92				
	1/19/2018	8:01:25	6.3	1	4.56	0.6	13.2
21	1/19/2018	8:02:11	5.35				
	1/19/2018	8:03:11	5.97	1	7.44	0.3	16.2
22	1/19/2018	8:03:11	5.97				
	1/19/2018	8:04:11	6.32	1	4.2	0.6	12.5
23	1/19/2018	8:04:52	5.35				
	1/19/2018	8:05:52	5.97	1	7.44	0.3	16.2
24	1/19/2018	8:05:52	5.97				
	1/19/2018	8:06:52	6.32	1	4.2	0.6	12.5
25	1/19/2018	8:07:30	5.3				
	1/19/2018	8:08:30	5.88	1	6.96	0.4	14.6
26	1/19/2018	8:08:30	5.88				
	1/19/2018	8:09:30	6.26	1	4.56	0.6	12.8
27	1/19/2018	8:10:18	5.37				
	1/19/2018	8:11:18	5.99	1	7.44	0.3	16.3
28	1/19/2018	8:11:18	5.99				
	1/19/2018	8:12:18	6.34	1	4.2	0.6	12.7
29	1/19/2018	8:14:27	5.32				
	1/19/2018	8:15:27	5.95	1	7.56	0.3	16.2
30	1/19/2018	8:15:27	5.95				
	1/19/2018	8:16:27	6.32	1	4.44	0.6	13.1
31	1/19/2018	8:17:10	5.33				
	1/19/2018	8:18:10	5.92	1	7.08	0.4	15.1
32	1/19/2018	8:18:10	5.92				
	1/19/2018	8:19:10	6.28	1	4.32	0.6	12.4
33	1/19/2018	8:19:53	5.3				
	1/19/2018	8:20:53	5.87	1	6.84	0.4	14.3
34	1/19/2018	8:20:53	5.87				
	1/19/2018	8:21:53	6.24	1	4.44	0.6	12.4
35	1/19/2018	8:22:38	5.3				
	1/19/2018	8:23:38	5.87	1	6.84	0.4	14.3

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-7/P-1

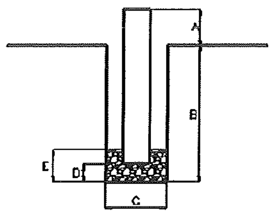
Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/19/2018

36	1/19/2018	8:23:38	5.87				
	1/19/2018	8:24:38	6.24	1	4.44	0.6	12.4
37	1/19/2018	8:25:18	5.33				
	1/19/2018	8:26:18	5.94	1	7.32	0.3	15.7
38	1/19/2018	8:26:18	5.94				
	1/19/2018	8:27:18	6.29	1	4.2	0.6	12.2
39	1/19/2018	8:28:10	5.33				
	1/19/2018	8:29:10	5.92	1	7.08	0.4	15.1
40	1/19/2018	8:29:10	5.92				
	1/19/2018	8:30:10	6.28	1	4.32	0.6	12.4
41	1/19/2018	8:30:54	5.35				
	1/19/2018	8:31:54	5.95	1	7.2	0.4	15.6
42	1/19/2018	8:31:54	5.95				
	1/19/2018	8:32:54	6.28	1	3.96	0.6	11.5
43	1/19/2018	8:33:36	5.35				
	1/19/2018	8:34:36	5.95	1	7.2	0.4	15.6
44	1/19/2018	8:34:36	5.95				
	1/19/2018	8:35:36	6.29	1	4.08	0.6	11.9
45	1/19/2018	8:36:14	5.33				
	1/19/2018	8:37:14	5.91	1	6.96	0.4	14.8
46	1/19/2018	8:37:14	5.91				
	1/19/2018	8:38:14	6.26	1	4.2	0.6	11.9

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-8/P-2

Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018



A. Top of Pipe Above Ground	16.5 Inches
B. Depth of Hole	36.5 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	26.5 Inches
F. Pipe Length	51 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 5 gallons to 20.6 inches from bottom
Checked Presoak Water Dropped 6 inches Twice in a Row in Less than 30 Minutes

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	1/18/2018	4:07:40	2.7				
Presoak	1/18/2018	4:08:08	3.5	0.5	9.6	0.1	54.2
2	1/18/2018	4:08:08	3.5				
Presoak	1/18/2018	4:08:20	3.7	0.2	2.4	0.2	47.7
3	1/18/2018	4:22:50	2.45				
Start Test	1/18/2018	4:23:50	3.52	1	12.84	0.2	31.4
4	1/18/2018	4:23:50	3.52				
	1/18/2018	4:24:45	3.95	1	5.16	0.5	25.9
5	1/18/2018	4:26:00	2.25				
	1/18/2018	4:27:00	3.27	1	12.24	0.2	26.2
6	1/18/2018	4:27:00	3.27				
	1/18/2018	4:28:00	3.78	1	6.12	0.4	22.6
7	1/18/2018	4:28:35	2.2				
	1/18/2018	4:29:35	3.15	1	11.4	0.2	23.3
8	1/18/2018	4:29:35	3.15				
	1/18/2018	4:30:35	3.72	1	6.84	0.4	23.3
9	1/18/2018	4:31:13	2.3				
	1/18/2018	4:32:13	3.24	1	11.28	0.2	24.3
10	1/18/2018	4:32:13	3.24				
	1/18/2018	4:33:13	3.72	1	5.76	0.4	20.4
11	1/18/2018	4:34:00	2.3				
	1/18/2018	4:35:00	3.2	1	10.8	0.2	23.0
12	1/18/2018	4:35:00	3.2				
	1/18/2018	4:36:00	3.68	1	5.76	0.4	19.7
13	1/18/2018	4:36:40	2.2				
	1/18/2018	4:37:40	3.15	1	11.4	0.2	23.3
14	1/18/2018	4:37:40	3.15				
	1/18/2018	4:38:40	3.67	1	6.24	0.4	20.8
15	1/18/2018	4:39:48	2.2				
	1/18/2018	4:40:48	3.12	1	11.04	0.2	22.4

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-8/P-2

Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018

16	1/18/2018	4:40:48	3.12				
	1/18/2018	4:41:48	3.64	1	6.24	0.4	20.3
17	1/18/2018	4:42:24	2.2				
	1/18/2018	4:43:24	3.09	1	10.68	0.2	21.5
18	1/18/2018	4:43:24	3.09				
	1/18/2018	4:44:24	3.59	1	6	0.4	18.9
19	1/18/2018	4:45:00	2.21				
	1/18/2018	4:46:00	3.08	1	10.44	0.2	21.0
20	1/18/2018	4:46:00	3.08				
	1/18/2018	4:47:00	3.58	1	6	0.4	18.7
21	1/18/2018	4:47:39	2.2				
	1/18/2018	4:48:39	3.07	1	10.44	0.2	20.9
22	1/18/2018	4:48:39	3.07				
	1/18/2018	4:49:39	3.58	1	6.12	0.4	19.0
23	1/18/2018	4:50:15	2.3				
	1/18/2018	4:51:15	3.13	1	9.96	0.3	20.8
24	1/18/2018	4:51:15	3.13				
	1/18/2018	4:52:15	3.61	1	5.76	0.4	18.5
25	1/18/2018	4:52:53	2.3				
	1/18/2018	4:53:53	3.14	1	10.08	0.3	21.1
26	1/18/2018	4:53:53	3.14				
	1/18/2018	4:54:53	3.61	1	5.64	0.5	18.2
27	1/18/2018	4:55:30	2.2				
	1/18/2018	4:56:30	3.06	1	10.32	0.2	20.6
28	1/18/2018	4:56:30	3.06				
	1/18/2018	4:57:30	3.54	1	5.76	0.4	17.5
29	1/18/2018	4:58:09	2.3				
	1/18/2018	4:59:09	3.1	1	9.6	0.3	19.9
30	1/18/2018	4:59:09	3.1				
	1/18/2018	5:00:09	3.57	1	5.64	0.5	17.7
31	1/18/2018	5:00:57	2.25				
	1/18/2018	5:01:57	3.1	1	10.2	0.3	20.9
32	1/18/2018	5:01:57	3.1				
	1/18/2018	5:02:57	3.56	1	5.52	0.5	17.2
33	1/18/2018	5:03:35	2.25				
	1/18/2018	5:04:35	3.07	1	9.84	0.3	20.0
34	1/18/2018	5:04:35	3.07				
	1/18/2018	5:05:35	3.55	1	5.76	0.4	17.7
35	1/18/2018	5:06:14	2.3				
	1/18/2018	5:07:14	3.1	1	9.6	0.3	19.9

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-8/P-2

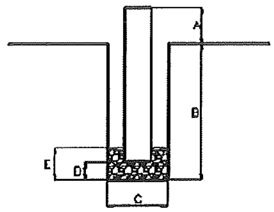
Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018

36	1/18/2018	5:07:14	3.1				
	1/18/2018	5:08:14	3.58	1	5.76	0.4	18.1
37	1/18/2018	5:08:55	2.25				
	1/18/2018	5:09:55	3.05	1	9.6	0.3	19.4
38	1/18/2018	5:09:55	3.05				
	1/18/2018	5:10:55	3.53	1	5.76	0.4	17.4
39	1/18/2018	5:11:37	2.25				
	1/18/2018	5:12:37	3.02	1	9.24	0.3	18.5
40	1/18/2018	5:12:37	3.02				
	1/18/2018	5:13:37	3.49	1	5.64	0.5	16.6
41	1/18/2018	5:14:10	2.2				
	1/18/2018	5:15:10	3	1	9.6	0.3	18.9
42	1/18/2018	5:15:10	3				
	1/18/2018	5:16:10	3.46	1	5.52	0.5	15.9
43	1/18/2018	5:16:41	2.2				
	1/18/2018	5:17:41	2.94	1	8.88	0.3	17.2
44	1/18/2018	5:17:41	2.94				
	1/18/2018	5:18:41	3.42	1	5.76	0.4	16.0
45	1/18/2018	5:19:14	2.3				
	1/18/2018	5:20:14	3.02	1	8.64	0.3	17.6
46	1/18/2018	5:20:14	3.02				
	1/18/2018	5:21:14	3.46	1	5.28	0.5	15.4

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-10/P-3

Project:	Proposed Multi-Family Residential Development, Redland	**Project No.**	G57101.01
Location:	SEC of East Lugonia Avenue and Occidental Drive, Redlands, California	**Test Date:**	1/18/2018



A. Top of Pipe Above Ground	24 Inches
B. Depth of Hole	52 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	4 Inches
E. Total Gravel Layer Depth	31 Inches
F. Pipe Length	72 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: 5 gallons to 21.4 inches from bottom
Checked Presoak Water Dropped 6 inches Twice in a Row in Less than 30 Minutes

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	1/18/2018	2:50:45	4.55				
Presoak	1/18/2018	2:51:50	5.35	1.1	9.6	0.3	22.3
2	1/18/2018	2:51:50	5.35				
Presoak	1/18/2018	2:52:20	5.55	0.5	2.4	0.5	17.9
3	1/18/2018	2:55:00	4.3				
Start Test	1/18/2018	2:56:00	5.14	1	10.08	0.3	22.1
4	1/18/2018	2:56:00	5.14				
	1/18/2018	2:57:00	5.51	1	4.44	0.6	14.8
5	1/18/2018	2:58:00	4.26				
	1/18/2018	2:59:00	5.09	1	9.96	0.3	21.3
6	1/18/2018	3:00:00	4.26				
	1/18/2018	3:01:00	5.07	1	9.72	0.3	20.7
7	1/18/2018	3:01:00	5.07				
	1/18/2018	3:02:00	5.48	1	4.92	0.5	15.7
8	1/18/2018	3:02:46	4.15				
	1/18/2018	3:03:46	5	1	10.2	0.3	20.7
9	1/18/2018	3:03:46	5				
	1/18/2018	3:04:43	5.43	1	5.16	0.5	16.5
10	1/18/2018	3:06:06	4.14				
	1/18/2018	3:07:06	4.98	1	10.08	0.3	20.3
11	1/18/2018	3:07:06	4.98				
	1/18/2018	3:08:06	5.4	1	5.04	0.5	15.0
12	1/18/2018	3:09:00	4.15				
	1/18/2018	3:10:00	4.98	1	9.96	0.3	20.1
13	1/18/2018	3:10:00	4.98				
	1/18/2018	3:11:00	5.41	1	5.16	0.5	15.4
14	1/18/2018	3:11:40	4.15				
	1/18/2018	3:12:40	4.97	1	9.84	0.3	19.8
15	1/18/2018	3:12:40	4.97				
	1/18/2018	3:13:40	5.4	1	5.16	0.5	15.3

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-10/P-3

Project:	Proposed Multi-Family Residential Development, Redland	**Project No.**	G57101.01
Location:	SEC of East Lugonia Avenue and Occidental Drive, Redlands, California	**Test Date:**	1/18/2018

16	1/18/2018	3:14:30	4.08				
	1/18/2018	3:15:30	4.9	1	9.84	0.3	19.1
17	1/18/2018	3:15:30	4.9				
	1/18/2018	3:16:30	5.38	1	5.76	0.4	16.5
18	1/18/2018	3:17:15	4.02				
	1/18/2018	3:18:15	4.96	1	11.28	0.2	21.9
19	1/18/2018	3:18:15	4.96				
	1/18/2018	3:19:15	5.39	1	5.16	0.5	15.2
20	1/18/2018	3:20:15	4.16				
	1/18/2018	3:21:15	5.09	1	11.16	0.2	23.3
21	1/18/2018	3:21:15	5.09				
	1/18/2018	3:22:15	5.47	1	4.56	0.6	14.6
22	1/18/2018	3:22:50	3.96				
	1/18/2018	3:23:50	4.88	1	11.04	0.2	20.7
23	1/18/2018	3:23:50	4.88				
	1/18/2018	3:24:50	5.33	1	5.4	0.5	15.1
24	1/18/2018	3:25:25	3.98				
	1/18/2018	3:26:25	4.87	1	10.68	0.2	20.1
25	1/18/2018	3:26:25	4.87				
	1/18/2018	3:27:25	5.32	1	5.4	0.5	15.0
26	1/18/2018	3:28:00	4.13				
	1/18/2018	3:29:00	4.91	1	9.36	0.3	18.5
27	1/18/2018	3:29:00	4.91				
	1/18/2018	3:30:00	5.36	1	5.4	0.5	15.5
28	1/18/2018	3:30:40	4.1				
	1/18/2018	3:31:40	4.91	1	9.72	0.3	19.0
29	1/18/2018	3:31:40	4.91				
	1/18/2018	3:32:40	5.35	1	5.28	0.5	15.1
30	1/18/2018	3:33:24	4.1				
	1/18/2018	3:34:24	4.9	1	9.6	0.3	18.8
31	1/18/2018	3:34:24	4.9				
	1/18/2018	3:35:24	5.35	1	5.4	0.5	15.3
32	1/18/2018	3:36:00	4.13				
	1/18/2018	3:37:00	4.92	1	9.48	0.3	18.8
33	1/18/2018	3:37:00	4.92				
	1/18/2018	3:38:00	5.36	1	5.28	0.5	15.2
34	1/18/2018	3:38:50	4.05				
	1/18/2018	3:39:50	4.85	1	9.6	0.3	18.3
35	1/18/2018	3:39:50	4.85				
	1/18/2018	3:40:50	5.33	1	5.76	0.4	16.0

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-10/P-3

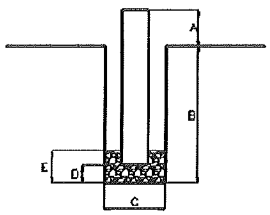
Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018

36	1/18/2018	3:41:25	4.17				
	1/18/2018	3:42:25	4.92	1	9	0.3	18.0
37	1/18/2018	3:42:25	4.92				
	1/18/2018	3:43:25	5.36	1	5.28	0.5	15.2
38	1/18/2018	3:44:05	3.95				
	1/18/2018	3:45:05	4.85	1	10.8	0.2	20.1
39	1/18/2018	3:45:05	4.85				
	1/18/2018	3:46:05	5.3	1	5.4	0.5	14.8
40	1/18/2018	3:46:45	4.13				
	1/18/2018	3:47:45	4.9	1	9.24	0.3	18.2
41	1/18/2018	3:47:45	4.9				
	1/18/2018	3:48:45	5.34	1	5.28	0.5	14.9
42	1/18/2018	3:49:30	4.08				
	1/18/2018	3:50:30	4.86	1	9.36	0.3	18.0
43	1/18/2018	3:50:30	4.86				
	1/18/2018	3:51:30	5.31	1	5.4	0.5	14.9
44	1/18/2018	3:52:15	4.05				
	1/18/2018	3:53:15	4.84	1	9.48	0.3	18.0
45	1/18/2018	3:53:15	4.84				
	1/18/2018	3:54:15	5.32	1	5.76	0.4	15.8

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-11/P-4

Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018



A. Top of Pipe Above Ground 32 Inches
B. Depth of Hole 45 Inches
C. Diameter of Hole 8 Inches
D. Depth of Gravel Below Pipe 5 Inches
E. Total Gravel Layer Depth 32 Inches
F. Pipe Length 72 Inches
G. Pipe Diameter 2 Inches

Pre-saturated: 5 gallons to 29 inches from bottom
Checked Presoak Water Dropped 6 inches Twice in a Row in Less than 30 Minutes

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Percolation Rate Corrected for Gravel (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	1/18/2018	1:34:00	4				
Presoak	1/18/2018	1:35:10	4.5	1	6	0.5	8.6
2	1/18/2018	1:35:10	4.5				
Presoak	1/18/2018	1:37:10	5	2	6	0.9	6.4
3	1/18/2018	1:39:30	3.82				
Start Test	1/18/2018	1:41:30	4.55	2	8.76	0.6	7.1
4	1/18/2018	1:41:30	4.55				
	1/18/2018	1:43:30	5.08	2	6.36	0.8	7.0
5	1/18/2018	1:44:30	3.75				
	1/18/2018	1:46:30	4.5	2	9	0.6	7.2
6	1/18/2018	1:46:30	4.5				
	1/18/2018	1:48:30	5.07	2	6.84	0.7	7.4
7	1/18/2018	1:49:30	3.75				
	1/18/2018	1:51:30	4.51	2	9.12	0.6	7.3
8	1/18/2018	1:51:30	4.51				
	1/18/2018	1:53:30	5.08	2	6.84	0.7	7.5
9	1/18/2018	1:54:30	3.8				
	1/18/2018	1:56:30	4.53	2	8.76	0.6	7.1
10	1/18/2018	1:56:30	4.53				
	1/18/2018	1:58:30	5.1	2	6.84	0.7	7.6
11	1/18/2018	1:59:40	3.82				
	1/18/2018	2:01:40	4.54	2	8.64	0.6	7.0
12	1/18/2018	2:01:40	4.54				
	1/18/2018	2:03:40	5.11	2	6.84	0.7	7.6
13	1/18/2018	2:04:45	3.85				
	1/18/2018	2:06:45	4.57	2	8.64	0.6	7.1
14	1/18/2018	2:06:45	4.57				
	1/18/2018	2:08:45	5.14	2	6.84	0.7	7.7
15	1/18/2018	2:09:45	3.79				
	1/18/2018	2:11:45	4.51	2	8.64	0.6	6.9

* Depth to water measured from top of pipe

PERCOLATION TEST FOR INFILTRATION ESTIMATE
No. B-11/P-4

Project: Proposed Multi-Family Residential Development, Redland **Project No.** G57101.01
Location: SEC of East Lugonia Avenue and Occidental Drive, Redlands, California **Test Date:** 1/18/2018

16	1/18/2018	2:11:45	4.51				
	1/18/2018	2:13:45	5.1	2	7.08	0.7	7.8
17	1/18/2018	2:14:45	3.8				
	1/18/2018	2:16:45	4.52	2	8.64	0.6	7.0
18	1/18/2018	2:16:45	4.52				
	1/18/2018	2:18:45	5.11	2	7.08	0.7	7.8
19	1/18/2018	2:19:40	3.83				
	1/18/2018	2:21:40	4.55	2	8.64	0.6	7.1
20	1/18/2018	2:21:40	4.55				
	1/18/2018	2:23:40	5.12	2	6.84	0.7	7.6
21	1/18/2018	2:24:30	3.84				
	1/18/2018	2:26:30	4.56	2	8.64	0.6	7.1
22	1/18/2018	2:26:30	4.56				
	1/18/2018	2:28:30	5.14	2	6.96	0.7	7.8
23	1/18/2018	2:29:30	3.87				
	1/18/2018	2:31:30	4.57	2	8.4	0.6	6.9
24	1/18/2018	2:31:30	4.57				
	1/18/2018	2:33:30	5.15	2	6.96	0.7	7.9
25	1/18/2018	2:34:30	3.78				
	1/18/2018	2:36:30	4.5	2	8.64	0.6	6.9
26	1/18/2018	2:36:30	4.5				
	1/18/2018	2:38:30	5.1	2	7.2	0.7	7.9

* Depth to water measured from top of pipe

APPENDIX E

PHOTOGRAPHS

Photograph No. 1: Viewing north of western portion of the vacant lot where stockpile of soil and cobbles exists, surrounded by cobbles, boulders and debris

Photograph No. 2: Typical conditions of vacant lot with gravel, cobbles, boulders, grasses and weeds at ground surface

Photograph No. 3: Viewing north at drainage ditch in western portion of vacant lot, south of Crystal Court

Photograph No. 4: Viewing east at Crystal Court

Photograph No. 5: Viewing north at asbestos pipe and nested cobbles and boulders in eastern portion of vacant lot

Photograph No. 6: Viewing north at screen wall exhibiting distress in the form of cracks. Screen wall divides the site from an existing apartment complex located north of the site.

Photograph No. 7: Viewing northwest at 1219 University Street located in eastern portion of the site

Photograph No. 8: Viewing west at 1215 University Street located in eastern portion of the site

Photograph No. 9: Viewing west at 1205 Unviersity Street located in eastern portion of the site

Photograph No. 10: Cracked patio slab on west side of residence at 1205 University Street