MOORE TWINING ASSOCIATES, INC.
Since 1898

DRAFT

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL DEVELOPMENT

NORTHEAST CORNER OF SYCAMORE LANE AND RUSSELL BOULEVARD

DAVIS, CALIFORNIA

Project Number: H12701.01

For:

California Property Owner I, LLC
c/o Brixmor Property Group
1525 Faraday Avenue, Suite 350
Carlsbad, CA 92008

December 2, 2022

December 2, 2022 H12701.01

Mr. Gregory Finley, PE
California Property Owner I, LLC
c/o Brixmor Property Group
1525 Faraday Avenue, Suite 350
Carlsbad, CA 92008

Subject: **Geotechnical Engineering Investigation**
Proposed Retail Development
Northeast Corner of Sycamore Lane and Russell Boulevard
Davis, California

Dar Mr. Finley:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed retail development to be located at the subject property.

The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations. It is recommended that those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) 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.

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

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

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

DRAFT

Allen H. Harker, PG
Professional Geologist

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for single-story retail structures to replace the existing shopping center structures at the northeast corner of Russell Boulevard and Sycamore Lane in Davis, California.

Based on the plans provided, it is our understanding the project will include removal of the existing shopping mall buildings and construction of seven (7) new single-story retail structures with a total footprint of about 100,000 square feet. Basements are not anticipated. Appurtenant construction is anticipated to include truck loading dock ramps, concrete flatwork, underground utilities, asphalt concrete paving and landscape areas. It is anticipated the majority of the existing parking lot will be reconstructed as part of the project. In addition, we understand the project may include onsite stormwater retention systems.

At the time of our field exploration, the site was occupied by multiple retail/commercial buildings, asphalt concrete pavements, concrete flatwork and isolated landscape planters with mature trees and landscaping. The site includes a long mostly single-story and partial two-story strip mall building and two separate buildings, a single-story Chinese restaurant in the northeast portion of the site and a single-story Trader Joe's grocery store in the southwest portion of the site that are planned to be demolished for new retail development. It is understood that the existing Trader Joe's grocery store located at the southwest corner of the overall property is to remain.

Between October 4 and 6, 2022, the investigation included eleven (11) borings denoted as B-1 through B-11, and four percolation test borings denoted as P-1 through P-4. Below the asphalt concrete pavements, the near surface soils encountered in the borings generally consisted of medium dense clayey sands or medium stiff to very stiff sandy lean clays, lean clays with sand and lean clays extending to depths of about 2½ feet to 45 feet BSG. These soils were underlain in some of the borings by loose to dense silty sands and/or additional medium stiff to very stiff sandy lean clays, loose clayey sands, and loose clayey gravel with sand.

Fill soils were difficult to distinguish between the native soils due to the absence of construction debris or multi-colored soils. However, based on the higher standard penetration test, N-values, encountered in the near surface soils of some of the borings, fill soils are suspected in borings B-1, B-2 and B-3. The fill soils suspected in borings B-1 through B-3 consisted of medium dense (nearly dense) clayey sands or stiff to very stiff sandy lean clays extending to depths of about 2½ to 3½ feet BSG.

Groundwater was generally not encountered in the borings during our October 4 through October 6, 2022 field exploration, except in boring B-5 where groundwater was encountered at a depth of 44 feet BSG.

Based on our field and laboratory investigation, the soils tested from the borings possess a low to medium expansion potential, moderate compressibility characteristics, moderate shear strength characteristics, and poor support characteristics for pavements when compacted as engineered fill.

EXECUTIVE SUMMARY (Continued)

In order to limit the differential static settlement of new foundations to ½ inch, over-excavation and compaction of the near surface soils is recommended to support the proposed foundations on a compacted engineered fill, including removal of all fill soils encountered and soils disturbed from demolition and removal of the existing improvements.

The soils encountered in the borings were generally clay soils which are not considered susceptible to liquefaction. However, some discontinuous zones of granular soils were encountered. Seismic settlements of about 1 inch total and ½ inch differential in 40 feet were estimated.

The results of percolation testing indicated negligible infiltration rates. Based on the test results and the nature of the fine-grained soils encountered during our investigation, infiltration of stormwater is not considered feasible.

Chemical testing of the near surface soil samples indicated the soils exhibit a “corrosive” to “highly corrosive” corrosion potential.

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

The potential for surface fault rupture at the site is considered low.

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

TABLE OF CONTENTS

TABLE OF CONTENTS

DRAFT

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED RETAIL DEVELOPMENT

NORTHEAST CORNER OF SYCAMORE LANE AND RUSSELL BOULEVARD

DAVIS, CALIFORNIA

Project Number: H12701.01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for a new retail development at the site of the existing Davis Collection shopping center located northeast corner of Sycamore Lane and Russell Boulevard in Davis, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by California Property Owner I, LLC to perform this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, site description, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices contain the drawings (Appendix A), the logs of borings (Appendix B), the results of laboratory tests (Appendix C), and the results of percolation tests (Appendix D).

The Geotechnical Engineering Division of Moore Twining, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The intent of this investigation is to satisfy the requirements of the 2019 California Building Code (CBC) as related to geotechnical investigations. The purpose of the investigation was to conduct an exploration program, evaluate the data collected during the field investigation and laboratory testing, and provide geotechnical engineering recommendations for project design.

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations with regard to the anticipated foundation loads;

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

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

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

2.1.5 Recommendations for temporary excavations, trench excavation, and trench backfill;

2.1.6 Evaluation for the potential for storm water infiltration based on the results of percolation tests;

2.1.7 Evaluation of the potential for liquefaction and seismic settlement;

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

2.1.9 Recommendations for asphalt concrete and Portland cement concrete pavements; and

2.1.10 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed improvements described in the Anticipated Construction section of this report. This investigation did not include a geologic/seismic hazards evaluation, flood plain investigation, compaction tests, percolation tests, environmental investigation, or environmental audit. In addition, since the existing structures were in use, this investigation did not include destructive testing or subsurface exploration below the existing structure(s).

2.2 **Scope:** Our proposal, reference MTP 22-0606, dated August 26, 2022, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A set of various plans, dated August 19, 2022, prepared by Architecture Design Collaborative for an entitlement submittal, was reviewed.

2.2.2 An ALTA Survey, dated April 29, 2011, was provided by the client and was reviewed for locations of existing utilities and storm drain lines. The survey sheet provided did not indicate who prepared the survey.

2.2.3 A report entitled, “Phase I Environmental Site Assessment, University Mall, 737-885 Russell Boulevard & Anderson Road, Davis, California 95616,” prepared by AEI Consultants, dated August 20, 2018, was reviewed.

2.2.4 Groundwater depth information from a report entitled, "Fourth Quarter 2010 Groundwater Monitoring Report, ARCO Station #5332, 705 Russell Boulevard, Davis, California," dated January 26, 2011, prepared by Arcadis, was reviewed. The report was obtained from the State Water Resources Control Board GeoTracker website.

2.2.5 Boring Permit #22-052H was obtained from the Yolo County Environmental Health Division.

2.2.6 A visual site reconnaissance and subsurface exploration were conducted.

2.2.7 Various satellite images of the site from 1993 to 2022 from online sources, were reviewed.

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

2.2.9 Mr. Gregory Finley (Brixmor Property Group) and Mr. Bill Brown (Brixmor Property Group), were consulted prior to the investigation.

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

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

3.1 Site Description: The subject site is addressed as 737-885 Russell Boulevard in Davis, California. It is our understanding the subject site comprises about 8¼ acres. The site is located northeast of the intersection of Sycamore Lane and Russell Boulevard and is currently occupied by the Davis Collection retail development. The site is bounded by existing apartments to the north, Sycamore Lane to the west, Russell Boulevard to the south, and Anderson Road to the east. An adjacent Arco gas station at the northwest corner of Russell Boulevard and Anderson Road is not considered a part of the site. The location of the subject site is shown on Drawing No. 1 in Appendix A.

At the time of our field exploration, the site was fully developed with multiple retail/commercial buildings, asphalt concrete pavements, concrete flatwork and isolated landscape planters with mature trees and landscaping. The site includes a long, mostly single-story and partial two-story strip mall building with numerous retail tenants and two stand alone retail buildings including a Chinese food restaurant in the northeast portion of the site and a Trader Joe's grocery store in the southwest portion of the site. It is understood that the existing Trader Joe's grocery store located at the southwest corner of the overall property is to remain. The existing strip mall building includes two depressed loading docks at the west end of the building. Numerous underground utilities and storm drains exist at the project site.

Site lights were noted throughout the parking lot areas. The parking lot included several landscaped islands with trees, and bushes and larger mature trees were also noted along portions of the perimeter of the shopping center.

The exterior of the existing buildings to be demolished were observed; however, no observations were made inside the existing buildings. No cracks were observed along the exterior building walls that were indicative of obvious differential settlement.

The existing asphalt concrete pavements directly surrounding the buildings and in the parking lot area exhibited various forms of cracking including longitudinal and transverse cracks and block cracking. In addition, many isolated areas of alligator cracking were observed in the drive lanes. Several areas of the asphalt concrete pavements had been previously patched to repair areas of pavement distress or where utilities were installed.

The grades at the site generally sloped away from the existing buildings and toward storm drain inlets. When the boring locations were marked for Underground Service Alert on September 21, 2022, the storm drain inlets were noted to be covered by leaves. According to a security guard at the shopping center, a large storm event had occurred in the two days prior to September 21st, and the pavements in the shopping center were flooded with a significant amount of standing water, especially at a low point on the northeast side of the existing Chinese restaurant in the eastern portion of the shopping center.

Based on our review of a satellite image of the site, site grades appear to range from about 46 to 50 feet above mean sea level (AMSL).

3.2 **<u>Previous Studies and Site History</u>:** A report entitled, "Phase I Environmental Site Assessment, University Mall, 737-885 Russell Boulevard & Anderson Road, Davis, California 95616," prepared by AEI Consultants, dated August 20, 2018, was reviewed for historical information regarding the property.

The Phase I Environmental Site Assessment (ESA) report included a review of historical information. The report indicated the following: "*Based upon a review of historical sources, the subject property was identified as agricultural land in 1937. By 1957, the subject property was vacant. By 1965, the subject property was graded for commercial development and the current, main commercial structure was developed by 1965 as a multi-tenant shopping mall. The eastern addition to the subject property was developed around 1981, as a partial two story structure connected to the main shopping mall, and an additional single-story structure. A commercial structure was developed on the southwestern side of the subject property in approximately 1989, which was later demolished in 2010 when the current commercial structure for a Trader Joe's was developed in the same year*."

The copy of the Phase I ESA report did not appear to include all of the historical aerial photographs reviewed. Based on our review of aerial images of the site from online sources dating back to 1993, the shopping center generally appears as it does in present day. The reconfiguration of the parking lot may have occurred around 1981 when the eastern addition to the subject property was developed. The Phase I ESA report also indicates that the current ATM kiosk was developed in the parking lot on the southern side of the subject property around 2001.

In regards to groundwater in the vicinity of the subject site, the report indicated, "*Based on groundwater monitoring data from an adjacent site at 705 Russell Boulevard (abutting the southeastern portion of the subject property) obtained from GeoTracker, groundwater is presumed to be present at an estimated depth of 19 to 42 feet bgs.*" Based on our review of two of the monitoring well installation logs included in the report for the ARCO station located adjacent to the southeast portion of the subject property, groundwater was encountered during drilling of monitoring wells MW-6 and MW-8 on June 3, 1993 and June 7, 1993, which stabilized at depths of 35.5 and 36.2 feet BSG, respectively.

A report entitled, "Fourth Quarter 2010 Groundwater Monitoring Report, ARCO Station #5332, 705 Russell Boulevard, Davis, California," dated January 26, 2011, prepared by Arcadis, was reviewed from the State Water Resources Control Board GeoTracker website. The report shows monitoring wells that were on the ARCO site, as well as in the southeastern portion of the subject site (in the parking lot area west of the ARCO site). Based on our review of the groundwater measurements made from the eight (8) monitoring wells between the years 1993 and 2010, groundwater depths ranged in depth from about 16½ feet BSG in 1998 to about 49 feet BSG in 1994.

No other previous geotechnical engineering, environmental, geological, or compaction test reports conducted for this site were provided for review. If these reports become available, the reports should be provided for review and consideration for this project.

3.3 <u>Anticipated Construction</u>: Based on the plans provided, it is our understanding the project will include removal of the existing shopping mall buildings and construction of seven (7) new single-story retail structures with a total footprint of about 100,000 square feet. Basements are not anticipated. Appurtenant construction is anticipated to include truck loading dock ramps, concrete flatwork, underground utilities, asphalt concrete paving and landscape areas. It is anticipated the majority of the existing parking lot will be reconstructed as part of the project. In addition, we understand the project may include onsite stormwater retention systems.

Structural loads for the proposed buildings were assumed to be up to 75 kips for interior column loads and 3 kips per lineal foot for load bearing walls.

Finished floor elevations of the proposed buildings are planned to range from about 48.9 feet to about 50.8 feet. Based on the relatively flat nature of the site, cuts and fills are anticipated to be limited to about 2 or 3 feet. As part of the site development, the existing building(s) will be removed, with the exception of the Trader Joe's building.

4.0 <u>INVESTIGATIVE PROCEDURES</u>

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

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

4.1.1 <u>Site Reconnaissance</u>: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Allen Harker of Moore Twining during marking out of the boring locations on September 21, 2022, and by Mr. Jordi Fragoza of Moore Twining during the field exploration between October 4 and 6, 2022. The features noted are described in the background information section of this report.

4.1.2 <u>Drilling Test Borings</u>: Prior to the investigation, Boring Permit #22-052H was obtained from the Yolo County Environmental Health Division. In addition, the borings were marked for Underground Service Alert, and a private utility locator was used to mark the locations of detected underground utilities in the vicinity of the boring locations.

Between October 4 and 6, 2022, the investigation included eleven (11) borings denoted as B-1 through B-11, and four (4) percolation test borings denoted as P-1 through P-4. It should be noted that boring B-7 was used to install percolation pipe and conduct percolation test P-2; thus, it is shown on the boring location map (Drawing No. 2 in Appendix A) as B-7/P-2. The borings were drilled to depths ranging from about 5 feet to 50 feet BSG. The depths of the borings considered the estimated depth of influence of the anticipated foundation loads, and the subsurface soil conditions encountered.

The test borings were drilled using a truck-mounted CME-75 drill rig equipped with 6⅝-inch or 8-inch outside diameter (O.D.) hollow-stem augers.

During the drilling of the test borings, bulk and relatively undisturbed samples of soil were obtained for laboratory testing. 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 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 up to an hour following completion of the borings.

Test boring locations were determined with reference to the existing site features shown on the site plan. The borings were generally backfilled with cuttings and topped with asphalt concrete cold patch except for borings deeper than 20 feet which were backfilled with neat cement in accordance with the requirements of Yolo County Environmental Health Division. The approximate locations of the borings are shown on Drawing No. 2 in Appendix A of this report.

4.1.3 Soil Sampling: 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 with the drill rig. The soil was retained in stainless steel rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.1.4 Percolation Tests: Percolation tests were conducted on October 5 and 6, 2022. Percolation test borings P-1 and P-4 were drilled to a depth of about 5 feet BSG on October 5 and October 4, 2022, respectively. Percolation test borings B-7/P-2 and P-3 were drilled to a depth of about 10 feet BSG on October 4 and October 5, 2022, respectively. As previously noted in this report, boring B-7 was used for percolation test P-2.

At the locations of P-1, P-2, B-7/P-3 and P-4, a percolation test was conducted within each borehole and an infiltration rate was estimated from the percolation test data.

The percolation tests were conducted by adding water to the percolation test holes and measuring the decline in the water level in the holes over time. The test holes were cylindrical with a diameter of about 8 inches. Gravel packing was used to protect the sidewalls of the hole from washout during refilling. A 2-inch diameter perforated PVC pipe was placed over a thin layer of gravel in the borehole and used to transmit poured water to the bottom of the hole.

The percolation test holes were presoaked with about 11.4 to 15 inches of water at least 24 hours prior to conducting the percolation tests. About 3.8 to 8.1 inches of presaturation water remained in the holes when checked about 21 to 25 hours after the presoak. On the day of the percolation tests, water was added to the percolation test holes prior to beginning the percolation tests. The percolation tests were conducted for about 2 to 3 hours by measuring the drop in water level over time. Measurements of water levels and the time of each reading were recorded on the field percolation test logs which are included in Appendix D of this report. The rate of water level decline near the end of the test period (generally stabilized) was used to estimate the average stabilized percolation rate of the soils tested. The head of the water in the test hole during the percolation test was generally about 11 to 14 inches when refilling the water level in the percolation test holes.

4.2 **Laboratory Testing:** The laboratory testing was programmed to determine selected physical and engineering properties of selected samples of the soils obtained during drilling. 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 **Surface Conditions:** At the time of our field exploration, the site was occupied by existing retail buildings, asphalt and concrete pavements, and landscaped areas. Various underground utilities are located throughout the site. Additional information regarding the existing site conditions is noted in the Background Information section of this report.

5.2 **Asphalt Concrete Pavements:** The existing asphalt concrete pavement sections and subgrade soil encountered in the borings drilled in asphalt concrete pavement areas are summarized in Table No. 1 below.

Table No. 1
Asphalt Concrete Pavement Section Thicknesses Encountered

Boring Number	Asphalt Concrete Thickness[1] (inches)	Aggregate Base Thickness[2] (inches)	Subgrade Soil
B-1	2	4	Clayey Sand Fill
B-2	2	4	Sandy Lean Clay Fill
B-3	2	4	Sandy Lean Clay Fill
B-4	3	4	Sandy Lean Clay
B-5	3	4	Lean Clay with Sand
B-6	2	4	Sandy Lean Clay
B-7/P-2	2	4	Silty Sand
B-8	3	4	Sandy Lean Clay
B-9	3	4	Sandy Lean Clay
B-10	5	4	Sandy Lean Clay
B-11	3	4	Lean Clay with Sand
P-1	3	4	Sandy Lean Clay
P-3	2	4	Sandy Lean Clay
P-4	2	3	Sandy Lean Clay

[1] - Asphalt concrete thickness averaged to the nearest inch.
[2] - Aggregate base thickness was measured to the nearest inch.

5.3 **<u>Soil Profile</u>:** Below the asphalt concrete pavements, the near surface soils encountered in the borings generally consisted of clayey sands, sandy lean clays, lean clays with sand and lean clays extending to depths of about 2½ feet to 45 feet BSG. These soils were underlain in some of the borings by silty sands and/or additional sandy lean clays, clayey sands, and clayey gravel with sand. The pavement section in percolation test boring B-7/P-2 was underlain by silty sands extending to a depth of 3½ feet BSG, which were underlain by sandy lean clays and clayey sands extending to the maximum depth explored, about 10 feet BSG.

Fill soils were difficult to distinguish between the native soils due to the absence of construction debris or multi-colored soils. However, based on the higher standard penetration test, N-values, encountered in the near surface soils of some of the borings, fill soils are suspected in borings B-1, B-2, B-3 and B-7/P-2. The fill soils suspected in borings B-1 through B-3 consisted of medium dense (nearly dense) clayey sands or stiff to very stiff sandy lean clays extending to depths of about 2½ to 3½ feet BSG. The fill soils suspected in percolation test boring B-7/P-2 consisted of medium dense silty sand that extended to a depth of 3½ feet BSG.

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 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.

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

Clayey Sands Fill Soils: The clayey sand fill soils encountered in boring B-1 were medium dense as indicated by an equivalent Standard Penetration Test (SPT), N-value, of 30 blows per foot, which was estimated by driving a California Modified split barrel sampler. The moisture content of a fill sample tested was about 7 percent. An Atterberg Limits test conducted on the same sample indicated a liquid limit of 25 and a plasticity index of 12.

Silty Sand Fill Soils: The silty sand fill soils encountered in percolation test boring B-7/P-2 were medium dense as indicated by a Standard Penetration Test (SPT), N-value, of 14 blows per foot. The moisture content of the silty sand fill sample tested was about 6 percent. An Atterberg Limits test conducted on the silty sand fill sample collected from depths of 1 to 2½ feet BSG from B-7/P-2 indicated the sample was non-viscous and non-plastic.

Sandy Lean Clay Fill Soils: The sandy lean clay fill soils encountered in borings B-2 and B-3 were medium stiff to stiff as indicated by a Standard Penetration Test (SPT), N-value, of 16 blows per foot and an equivalent Standard Penetration Test (SPT), N-value, ranging of 14 blows per foot, which was estimated by driving a California Modified split barrel sampler. The moisture content of the fill samples tested were about 12 and 15 percent. The results of testing of one (1) relatively undisturbed sample indicated a dry density of 110.2 pounds per cubic foot.

Native Sandy Lean Clays, Lean Clay with Sand and Lean Clays: The native sandy lean clays, lean clays with sand and lean clays encountered were described as medium stiff to very stiff, as determined by Standard Penetration Test (SPT), N-values, ranging from 4 to 16 blows per foot, and as indicated by equivalent Standard Penetration Test (SPT), N-values, ranging from 5 to 18 blows per foot, which were estimated by driving a California Modified split barrel sampler. The moisture content of the samples tested ranged from about 7 to 22 percent. The results of testing of sixteen (16) relatively undisturbed samples indicated dry densities ranging from 87.2 to 109.0 pounds per cubic foot. Atterberg Limits tests on various lean clay with sand and lean clay samples indicated liquid limits of 40, 33, 42, and corresponding plasticity indices of 21, 15, 27, respectively. A direct shear test conducted on a sandy lean clay sample collected at depths of 5 to 6½ feet BSG from boring B-10 indicated an internal angle of friction of 29 degrees and 80 pounds per square foot of cohesion. Three (3) consolidation tests conducted on sandy lean clay samples collected from borings B-3, B-8, and B-10 indicated about 6.5, 5, and 8.9 percent consolidation under a load of 16 kips per square foot). The consolidation test from B-3 at depths of 3½ to 5 feet BSG exhibited 0.6 percent swell when wetted under a load of 0.5 kips per square foot. The consolidation test from B-8 at depths of 1 to 2½ feet BSG exhibited 1.8 percent swell when wetted under a load of 0.25 kips per square foot. The consolidation test from B-10 at depths of 5 to 6½ feet BSG exhibited 0.2 percent swell when wetted under a load of 0.5 kips per square foot.

Native Silty Sands: The native silty sands encountered were described as loose to dense, as determined by Standard Penetration Test (SPT), N-values, ranging from 5 to 47 blows per foot. The moisture content of the silty sands ranged from about 4 to 7 percent for samples tested above groundwater and from about 23 to 24 percent for samples tested below groundwater. Atterberg Limits tests conducted on silty sand samples collected from depths of about 45 to 46 ½ feet BSG from boring B-5 and from depths of 18½ to 20 feet BSG from boring B-11 both indicated the samples were non-viscous and non-plastic.

Native Clayey Sands: The native clayey sands encountered were described as loose, as determined by a Standard Penetration Test (SPT), N-value, of 4 blows per foot. The moisture content of a sample tested was about 12 percent. An Atterberg Limits test conducted on a clayey sand sample collected from depths of about 8 ½ to 10 feet BSG from percolation test boring B-7/P-2 indicated a liquid limit of 33 and a plasticity index of 21.

Native Clayey Gravel with Sand: The native clayey gravel with sand encountered near the bottom of percolation test boring P-4 was described as loose, as determined by a Standard Penetration Test (SPT), N-value, of 4 blows per foot. The moisture content of a sample tested was about 8 percent. An Atterberg Limits test conducted on a clayey gravel with sand sample indicated a liquid limit of 28 and a plasticity index of 13.

Expansion Index Tests: Three (3) expansion index tests conducted on clayey sand and sandy lean clay samples collected from depths of about 1 to 3½ feet BSG from borings B-1, B-2 and B-8 indicated expansion index values of 33, 64 and 67, respectively.

Maximum Density-Optimum Moisture Determinations: A maximum density-optimum moisture determination conducted on a sample collected from depths of about 1 to 3½ feet BSG from boring B-5 indicated a maximum dry density of 118.1 pounds per cubic foot at an optimum moisture content of 12.5 percent. A maximum density-optimum moisture determination conducted on a sample collected from depths of about 1 to 3½ feet BSG from boring B-11 indicated a maximum dry density of 115.9 pounds per cubic foot at an optimum moisture content of 14.4 percent.

R-value: The result of two (2) R-value tests conducted on near surface samples of sandy lean clay obtained from borings B-6 and B-10 from depths of 1 to 3½ feet BSG indicated R-values of 16 and 12, respectively.

Chemical Tests: Chemical tests performed on three (3) near surface soil samples collected from depths of about 1 to 3½ feet BSG from borings B-1, B-5 and B-11 indicated pH values of 8.7, 8.8, and 8.5; minimum resistivity values of 3,400; 2,700; and 2,000 ohm-centimeters; 0.0035, 0.0027, and 0.016 percent by weight concentrations of sulfate; and 0.0012, 0.00067, and 0.006 percent by weight concentration of chloride, respectively.

5.5 Groundwater Conditions: Groundwater was generally not encountered in the borings during our October 4 through October 6, 2022 field exploration, except in boring B-5 where groundwater was encountered at a depth of 44 feet BSG. A stabilized groundwater level could not be obtained from this boring as it was required to be grouted immediately after drilling under the observation of a Yolo County Environmental Health Division inspector.

Based on our review of the groundwater measurements made from the eight (8) monitoring wells between the years 1993 and 2010 at the adjacent ARCO gas station, groundwater ranged in depth from about 16½ feet BSG in 1998 to about 49 feet BSG in 1994.

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.

5.6 Results of Percolation Testing: The infiltration rates estimated from the percolation test data are summarized in Table No. 1 below. The percolation test data are included in Appendix D.

Table No. 1
Results of Percolation Testing

Location and Depth	Field (Unfactored) Infiltration Rate (Inches per Hour)[1]	Subgrade Soil Type
P-1 at 5 feet BSG	0	Lean Clay
B-7/P-2 at 10 feet BSG	0.2	Clayey Sand

Location and Depth	Field (Unfactored) Infiltration Rate (Inches per Hour)[1]	Subgrade Soil Type
P-3 at 10 feet BSG	0	Lean Clay
P-4 at 5 feet BSG	0.1	Clayey Gravel with 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 encountered during 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 field exploration, the site was occupied by multiple retail/commercial buildings, asphalt concrete pavements, concrete flatwork and isolated landscape planters with mature trees and landscaping. The site includes a one and two-story strip mall building with various tenants, a stand alone Chinese food restaurant and a stand alone Trader Joe’s grocery store. It is understood that the existing Trader Joe’s grocery store located at the southwest corner of the overall property is to remain. The strip mall building includes two depressed loading docks at the west end of the building. Numerous underground utilities and storm drains exist throughout the project site.

As part of the site preparation, the existing surface and subsurface improvements (buildings, loading docks, foundations, pavements, underground utilities, storm drains pipelines, etc.) and associated fill soils will need to be removed. In addition, all soils disturbed from demolition and removal of the existing surface and subsurface improvements should be removed during site preparation. During our field exploration, undocumented fill soils were encountered in some of the borings to depths of about 2½ to 3½ feet BSG. As part of the site preparation, the existing fill soils will need to be removed and compacted as engineered fill in areas of planned improvements which are sensitive to settlement. Excavations resulting from removal of surface and subsurface improvements should be backfilled with engineered fill in accordance with the recommendations of this report.

Any debris generated during demolition at the site and any debris encountered in the fill soils should not be incorporated into the soils for use as fill below the proposed buildings.

The trees, root systems, and plants within the existing landscape planters will need to be removed where new improvements are planned. Stripping and removal of plants and trees should include removal of root balls and removal of all roots greater than ¼ inch in diameter and all soils with an organic content of at least 3 percent by weight. Stripped materials from landscaped areas should not be mixed with soils to be reused as engineered fill.

6.2 **Expansive Soils:** In evaluation of the potential for expansive soils, expansion index testing was performed on a representative sample of the near surface soils encountered. The expansion index testing was performed in accordance with ASTM D4829. The samples were tested and classified by expansion potential in accordance with Table 1 of ASTM D4829 and the results are summarized in Appendix C of this report. The results of the expansion index testing indicated the near surface lean clay soils are expansive with a medium expansion potential based on expansion index values of 64 and 67. Due to the expansive soils conditions, this report recommends that the interior slab-on-grade and all slabs attached to the building be underlain by at least 4 inches of aggregate base over a non-expansive granular engineered fill. As an alternative to importing non-expansive fill, it may be possible to chemically treat the onsite clay soils to reduce the plasticity of the soil for use as a non-expansive fill.

6.3 **Static Settlement and Bearing Capacity of Shallow Foundations:** The potential for excessive total and differential static settlement of foundations and slabs-on-grade is a geotechnical engineering 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 compressibility of the near surface soils, the presence of undocumented fill, and the potential for disturbance of the near surface soils from demolition and removal of the existing improvements, over-excavation and compaction of the near surface soils is recommended to support the new foundations on engineered fill in order to limit the static settlement to 1 inch total and ½ inch differential. Provided the site preparation recommendations of this report are followed, a net allowable soil bearing pressure of 2,500 pounds per square foot, for dead-plus-live loads, may be used for design. This is based on assumed structural loads for the proposed buildings of up to 75 kips for interior column loads and 3 kips per lineal foot for load bearing walls.

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.

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

6.4 Seismic Ground Rupture and Design Parameters: The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Based on our review, the an unnamed fault located about 19 ½ miles northwest of the site near the Dunnigan Hills area is the closest active fault to the site. The potential for surface fault rupture at the site is considered low.

It is assumed 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 coefficients 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 0.448g 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). A Maximum Considered Earthquake magnitude of 6.1 was determined for the site based on deaggregation analysis (United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/).

6.5 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 movement of the soil mass, combined with loss of bearing, can result. Saturated, loose, granular soils, 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).

Groundwater was encountered during the October 2022 field exploration in boring B-5 at a depth of 44 feet BSG. Based on our review of the groundwater measurements made from the eight (8) monitoring wells between the years 1993 and 2010 at the adjacent ARCO gas station, groundwater ranged in depth from about 16½ feet BSG in 1998 to about 49 feet BSG in 1994. Thus, a groundwater depth of 16½ feet BSG was used for the liquefaction analysis.

The analysis was conducted using the computer program LiquefyPro, developed by CivilTech Software. A horizontal ground acceleration of 0.448g, a maximum considered earthquake of 6.1 and a groundwater depth of 16½ feet were used in the analysis of the soils. Soil parameters, such as wet unit weight, N-values and fines content were input from the boring data for the soil layers encountered throughout the depths explored. The analysis was conducted based on the soil conditions encountered in boring B-5 that extended to a depth of 50 feet BSG and B-11 that extended to a depth of about 20 feet BSG. The soils encountered in the borings were generally clay soils which are not considered susceptible to liquefaction. The silty sand layer encountered from depths of 45 to 50 feet BSG in boring B-5 was medium dense to dense and was not indicated to be susceptible to liquefaction. However, the silty sand layer encountered from depths of 18½ to 20 feet BSG in boring B-11 was loose and was considered susceptible to liquefaction. Although the thickness of this loose silty sand encountered in Boring B-5 was not determined, since a granular soil was not encountered at this depth in any of the other borings, the granular soil encountered in boring B-11 are not a continuous layer throughout the site and it is anticipated that this layer is limited in lateral extent as well as thickness. The analysis indicated seismic settlements of about 1 inch total and ½ inch differential.

6.6 **<u>Asphalt Concrete (AC) Pavements</u>:** Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphalt 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 8.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

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

Based on the results of the laboratory testing, an R-value of 12 was used as a basis for the pavement section thickness recommendations.

6.7 **<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).

The recommendations provided in this report for PCC pavements are based on a trash truck loading at a frequency equivalent to a traffic index between 6.0 and 8.0 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 150 psi/in was used for the pavement design considering the results of the R-value testing and considering that the pavement will be underlain by 6 inches of aggregate base.

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

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 2, below.

Table No. 2
Association of Corrosion Engineers (NACE) Corrosion Severity 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” to “highly corrosive” corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the “highly corrosive” corrosion potential of the soils tested. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

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

Table No. 3
ACI Exposure Categories for Water Soluble Sulfate in Soils

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

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

7.0 CONCLUSIONS

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

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

7.2 Below the existing pavements, the near surface soils encountered in the borings generally consisted of medium dense clayey sands or medium stiff to very stiff sandy lean clays, lean clays with sand and lean clays extending to depths of about 2½ feet to 45 feet BSG. These soils were underlain in some of the borings by loose to dense silty sands and/or additional medium stiff to very stiff sandy lean clays, loose clayey sands, and loose clayey gravel with sand. Undocumented fill soils were encountered in some of the borings to depths of about 2½ to 3½ feet BSG.

7.3 Based on our field and laboratory investigation, the clay soils generally encountered in the borings exhibited a medium expansion potential, moderate compressibility characteristics, moderate shear strength characteristics, and poor support characteristics for pavements when compacted as engineered fill.

7.4 Groundwater was generally not encountered in the borings during our October 4 through October 6, 2022 field exploration, except in boring B-5 where groundwater was encountered at a depth of 44 feet BSG. Based on our review of the groundwater measurements made from the eight (8) monitoring wells between the years 1993 and 2010 at the adjacent ARCO gas station, groundwater ranged in depth from about 16½ feet BSG in 1998 to about 49 feet BSG in 1994.

7.5 In order to limit the differential static settlement of new foundations to ½ inch, over-excavation and compaction of the near surface soils is recommended to support the proposed foundations on engineered fill, including removal of all fill soils encountered and soils disturbed from demolition and removal of the existing improvements.

7.6 Due to the expansive soil conditions, this report recommends a non-expansive fill below concrete slabs on grade to reduce the potential for excessive heave. As an alternative to importing a granular fill for the non-expansive section, it may be possible to treat the onsite soils with a chemical additive (i.e., lime or cement) to reduce the plasticity of the onsite soils for use as a non-expansive fill below slabs on grade. However, chemical treatment compatibility testing would need to be conducted to determine the type and application rate for treatment of the onsite soils for this purpose.

7.7 The soils encountered in the borings were generally clay soils which are not considered susceptible to liquefaction. The analysis indicated seismic settlements of about 1 inch total and ½ inch differential in 40 feet.

7.8 The results of percolation testing indicated negligible infiltration rates of 0 to 0.2 inches per hour at depths of about 5 to 10 feet BSG in percolation tests P-1, B-7/P-2, P-3 and P-4. Based on the test results and the fine -grained nature of the soils encountered during our investigation, infiltration of stormwater is not considered feasible at the depths the percolation tests were conducted.

7.9 Chemical testing of the near surface soil samples indicated the soils exhibit a "corrosive" to "highly corrosive" corrosion potential.

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

7.11 The potential for surface fault rupture at the site is considered 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 Structural loads for the proposed retail buildings were assumed as described in the Anticipated Construction section of this report. 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 appropriate. In the event the foundation loads are different than assumed, the recommendations in this report may need to be revised.

8.1.2 A preconstruction meeting including, as a minimum, the owner, developer, 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 project requirements and scheduling.

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

8.1.4 As an alternative to importing non-expansive fill that is recommended below the aggregate base below concrete slabs-on-grade in this report, the expansive clay soils could potentially be lime treated to reduce their plasticity and expansion characteristics. However, samples would need to be tested for lime treatment suitability to confirm if the on-site clay soils are suitable for lime treatment and determine the appropriate percentage of lime (such as 5 percent high calcium quicklime) to use.

8.1.5 If wet, unstable soil conditions are experienced, methods such as aeration, mixing wet soils with drier soils, chemical (i.e., lime) treatment of the soil, or over-excavation and placement of a bridge lift of aggregate base and a geotextile stabilization fabric such as Mirafi 600X, may be required to achieve a stable soil condition. The actual method employed to stabilize the bottom of the excavation or pavement subgrade should be selected at the time of construction.

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

8.2 Site Grades 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 five feet away from the structures, in accordance with the applicable code requirements, 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 for a distance of at least five feet from the building 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 structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be 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 structure and should establish positive drainage of water away from the structure. 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 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.

8.2.7 Storm drain inlets for proposed storm drain systems should be monitored and cleaned periodically to keep inlets from getting clogged.

8.2.8 The results of the percolation testing indicate that infiltration of storm water will not be feasible at the depths of the percolation tests (5 and 10 feet BSG). In the event subsurface storm water collection systems which concentrate runoff and allow wetting of the soils are planned, the proposed locations and details of these features should be provided to Moore Twining for review and comment. In general, stormwater collection systems which allow wetting of the soils should not be located within 20 feet of building foundations. If these types of features are required, sufficient setbacks to existing improvements should be maintained, and/or specific measures such as deepened curbs, cutoffs, liners, etc. should be incorporated in the designs to reduce the potential for excessive settlement of improvements due to moisture and freewater migration from storm water disposal systems.

8.3 Site Preparation

8.3.1 As part of site preparation, all existing underground utilities, foundations, subsurface structures, and associated fills should be excavated and removed from the site and all soils disturbed from the demolition and removal of these improvements should be over-excavated to expose undisturbed soils. Trench backfill soils should be excavated from within a zone extending from 1 foot below the pipe at a 1H to 1V slope to the ground surface. Utilities and storm drain lines to be removed should be completely removed and disposed of off-site. Excavations to remove existing improvements should extend to at least 12 inches below the bottom of the improvements to be removed or to the depth required to remove all soils disturbed from demolition, whichever is greater. After over-excavation, prior to backfill, the bottom of the excavation should be scarified to a depth of 8 inches, moisture conditioned, and compacted as engineered fill.

8.3.2 As part of site preparation, all surface vegetation, landscape bark, topsoil, organics, etc., should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and surface topsoils with more than 3 percent organics by dry weight. For estimating purposes, a minimum stripping depth of 4 inches is anticipated. Stripping should extend laterally a minimum of 5 feet outside the limits of the new improvements (i.e., proposed buildings, slabs-on-grade, pavements, etc.). 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.

8.3.3 Existing trees in the areas of new improvements should be removed. The proper removal of existing trees and their associated root structures is an important aspect of this project that should be properly planned and monitored. Excavation of tree roots should be conducted to remove all root systems greater than about ¼ inch in diameter. These materials should be raked and hand-picked, as necessary, to remove tree roots larger than ¼ inch in diameter and concentrated root masses. All roots larger than ¼ inch in diameter or any accumulation of organic matter that will result in an organic content more than 3 percent should be removed and not used as engineered fill. Limbs, tree branches, roots, etc. should not be disced into the soils. After removal, the bottom of the excavation should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations. Areas of depressions should be excavated and backfilled with engineered fill under the observation of Moore Twining.

8.3.4 After stripping and removal of existing surface and subsurface improvements, the building areas and all new foundations should be over-excavated to at least 3½ feet below preconstruction site grades, to at least 12 inches below the bottom of the existing improvements to be removed, to the depth required to remove all fill soils (encountered to depths of 2½ to 3½ feet BSG in some of the borings), and to at least 12 inches below the bottom of the footings, whichever is greater. The over-excavation limits should include the entire building footprint, all foundations and adjacent walkways, a minimum of 5 feet horizontally beyond the foundations, and a minimum of 3 feet horizontally beyond walkways adjacent to the building, whichever is further. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the excavation should be scarified 8 inches in depth, moisture conditioned one (1) to four (4) percent above optimum moisture content and compacted as engineered fill.

8.3.5 The plans should depict the minimum limits of over-excavation for the building pads as described in section 8.3.4.

8.3.6 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for surveying and measuring to verify 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.

8.3.7 Following stripping and removal of existing surface and subsurface improvements, exterior slabs-on-grade which are not located adjacent to the building (i.e., exterior slabs which are outside the building pad preparation limits), pavements and areas to receive fill outside the building pad over-excavation limits should be prepared by over-excavation to a minimum of 12 inches below the resulting ground surface, to the depth required to remove existing fill soils (encountered to depths of 2½ to 3½ feet BSG in some of the borings), to the bottom of the aggregate base, and to at least 12 inches below the bottom of the existing improvements to be removed, whichever is greater. Over-excavation should extend horizontally a minimum of 3 feet beyond exterior slabs on grade and pavements, or up to the existing improvements to remain, whichever occurs first. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the over-excavation should be scarified to a minimum depth of 12 inches, moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted as engineered fill. As an alternative to removal of the existing fill soils, the existing fill soils may be left in place below exterior slab-on-grade which are located outside of the building pad preparation limits or pavement areas if the increased risk of settlement is considered acceptable by the Owner.

8.3.8 Structural loads for miscellaneous, lightly loaded foundations (such as retaining walls, sound walls, screen walls, monument signs, etc.) should be evaluated on a case by case basis to present supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, the areas of miscellaneous foundations should be over-excavated to at least 12 inches below preconstruction site grades, to at least 12 inches below subsurface structures to be removed, to the depth required to remove all existing fill soils (encountered to depths of 2½ to 3½ feet BSG in some of the borings), and to the bottom of foundations, whichever is greater. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the over-excavation should be scarified to a depth of 8 inches, moisture conditioned to one (1) to four (4) percent above optimum moisture content and compacted as engineered fill. The over-excavation should extend a minimum of 3 feet beyond the limits of the foundations on all sides, or to property lines, or to improvements to remain, whichever occurs first.

8.3.9 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.

8.3.10 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.

8.3.11 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.

8.3.12 Final grading shall produce a building pad 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.

8.3.13 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.

8.4 Engineered Fill

8.4.1 The near surface soils encountered with an expansion index of less than 75 are considered suitable for use as engineered fill below depths of 18 inches below interior concrete slabs on grade and below depths of 12 inches below exterior concrete slabs on grade and Portland cement concrete pavements, provided that the soils are free of debris, do not contain material greater than 6 inches in maximum dimension, and are moisture conditioned in accordance with the recommendations of this report. During site preparation, debris and unsuitable materials encountered should be removed from the soils to be used as engineered fill. Interior concrete slabs on grade and exterior concrete slabs on grade directly adjacent to the buildings should be supported on a minimum of 4 inches of non-recycled Class 2 aggregate base over 14 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. Exterior slabs-on-grade which are not located adjacent to the building should be underlain by 4 inches of Class 2 aggregate base over 8 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. As an alternative, Class 2 aggregate base may be substituted for the imported, non-expansive granular fill soils. Portland cement concrete pavements should be underlain by 6 inches of Class 2 aggregate base over the prepared subgrade soils. It also may be possible to lime treat the on site clay soils in lieu of importing non-expansive, granular fill soils. However, laboratory lime treatment suitability testing would need to be conducted in order to determine if the on-site soils are suitable for lime treatment and to evaluate the type and percentage of chemical additive.

8.4.2 If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.4.3 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.

8.4.4 Import fill soil used for the building pad preparation (if any) should be non-recycled, have a very low expansion potential and be 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
Plasticity Index (ASTM D4318)	Less than 12
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Resistivity	> 3,000 ohms-cm

Prior to importing fill, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. The Contractor shall pay for the environmental testing required to determine compliance with the requirements of this report. This certification shall consist of, as a minimum, recent analytical data specific to the source of the import material including proper chain-of-custody documentation. An acceptable guideline for analytical testing of import soil is included in the October 2001 Department of Toxic Substance Control document. Moore Twining will sample and test the material after the environmental certification submittal is approved to verify that the proposed material complies with the geotechnical engineering recommendations of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested for the geotechnical properties.

8.4.5 Onsite clayey soils used as engineered fill and final utility trench backfill (minimum of 12 inches above the pipe) should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to at least one (1) and four (4) percent above optimum moisture content, and compacted to a dry density of at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.6 Imported, granular engineered fill, bedding sand and initial utility trench backfill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to optimum to three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.7 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at the minimum frequency listed in Table No. 3, below.

Table No. 4
Minimum In-place Density Test Frequency

Area	Minimum Test Frequency
Mass Fills or Subgrade for Building Pad	1 test per 5,000 square feet per compacted lift, but not less than 3 tests per building pad per lift
Pavement Subgrade and Aggregate Base	1 test per 10,000 square feet per compacted lift
Utility Lines	1 test per 150 feet per compacted lift

8.4.8 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, 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. Materials such as crushed rock should be placed in thin (less than 8 inches) lifts and each lift should be compacted with a minimum of three (3) passes with a vibratory compactor.

8.4.9 Aggregate base below the building slab should comply with State of California Department of Transportation requirements for a Class 2 aggregate base, with exception that the aggregate base used below the building slab should not contain recycled materials. Aggregate base for exterior slabs on grade and pavements should comply with State of California Department of Transportation requirements for a Class 2 aggregate base and may include recycled materials. 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.

8.5 Conventional Shallow Spread Foundations and Concrete Slabs on Grade

8.5.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations and slabs on grade based on the estimated settlements. The following should be anticipated for design: 1) a total static settlement and heave of 1 inch; 2) a differential static settlement and heave 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.

8.5.2 Building foundations supported on engineered fill soils prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.5.3 All perimeter footings for the new building should have a minimum depth of 24 inches below the lowest adjacent grade. All interior foundations should have a minimum depth of 18 inches below the bottom of the floor slab. All footings for the new building should have a minimum width of 15 inches, regardless of load.

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

8.5.5 Structural loads for miscellaneous, lightly loaded non-building foundations (such as retaining walls, sound walls, screen walls, monument signs, etc.) should be supported on subgrade soils prepared as recommended in the Site Preparation section of this report. Spread and continuous footings for miscellaneous foundations extending a minimum depth of 18 inches below grade may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.

8.5.6 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 latitude of 38.547424 degrees and a longitude of -121.760327 degrees and a Site Class D. 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 Factors

Seismic Factor	2019 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.448
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration (PGA)	0.362
Spectral Response At Short Period (0.2 Second), Ss	0.861

Seismic Factor	2019 CBC Value
Spectral Response At 1-Second Period, S_1	0.329
Site Coefficient (based on Spectral Response At Short Period), Fa	1.156
Site Coefficient (based on spectral response at 1-second period) Fv	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	0.995
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}	0.663
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 the structural engineer determines that an Exception of Section 11.4.8 of ASCE 7-16 is applicable for the project design.

8.5.7 The prepared soils exposed in foundation excavations should be periodically moistened to maintain the moisture content in the onsite clayey soils at a minimum of one (1) percent above optimum until concrete is placed. It should be noted that the contractor should take precautions not to allow the exposed soils to dry, including weekends and holidays.

8.5.8 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.

8.5.9 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 0.30 can be used for design. In areas where slabs are underlain by a synthetic moisture vapor membrane, an allowable coefficient of friction of 0.10 can be used for design.

8.5.10 Site lighting and pylon signs may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot may be used to resist axial loads. Lateral load resistance may be estimated using the 2019 CBC non-constrained procedure. 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 250 pounds per cubic foot to a maximum of 2,500 pounds per square foot. The passive pressure may be assumed to act over twice the pier diameter. The upper 12 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance.

8.6 Frictional Coefficient and Earth Pressures

8.6.1 The bottom surface area of concrete footings in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.30, can be used for design.

8.6.2 The allowable passive resistance of the engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. The upper 12 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.6.3 The active and at-rest pressures of imported, non-expansive engineered fill meeting the requirements of Section 8.7.1 of this report may be assumed to be equal to the pressures developed by a fluid with a density of 45 and 68 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.6.4 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.

8.6.5 The wall designer should determine if seismic increments should be used or not. If seismic increments are required, contact Moore Twining for recommendations for seismic geotechnical design considerations for the retaining structures.

8.6.6 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a drain, a filter fabric encased gravel section and a geo-composite system, to prevent hydrostatic pressures from acting on the walls. Recommendations for drainage of walls are included in Section 8.7 of this report. Drainage should be directed to perforated pipes running parallel to the walls which can carry drainage from behind the walls to the on-site drainage system. Clean-outs should be incorporated into the design.

8.7 Retaining Walls

8.7.1 Retaining walls should be constructed with imported granular backfill placed within the zone extending from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	80 - 100
Percent Passing No. 200 Sieve	0 - 20
Plasticity Index	Less than 5
Internal Angle of Friction	$\geq$30 degrees

8.7.2 All retaining wall backfill should be compacted as engineered fill.

8.7.3 Retaining walls may be subject to lateral loading from pressures exerted from slabs-on-grade, and pavement 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.

8.7.4 Retaining walls should be constructed with a drain system including, as a minimum, drain pipes surrounded by at least 1 cubic foot of ¾-inch open graded rock fully encapsulated by geotextile filter fabric (140N or equivalent). Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system does not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The drainage system should be detailed on the plans.

8.7.5 Segmented wall design (mechanically stabilized 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. However, 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.

8.7.6 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.

8.7.7 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., waterproofing measures should be applied to walls. Waterproofing systems should be designed by a qualified professional.

8.8 Interior Concrete Slabs on Grade and Moisture Vapor Retarder

The recommendations provided herein are intended only for design of interior concrete slabs-on-grade, and their proposed uses, which do not include construction loading. The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

8.8.1 The concrete slabs on grade should be reinforced for the anticipated temperature and shrinkage stresses, settlement and swell. A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade as well as any reinforcement for temperature and shrinkage stresses based on the settlements noted in this report.

8.8.2 The subgrade soils 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 concrete slabs on grade should be supported on 4 inches of non-recycled aggregate base over 14 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils.

8.8.3 The moisture content of the clay subgrade soils below the non-expansive fill should be verified to be at least one (1) percent above optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.8.4 A vapor retarder should be placed below interior building slabs where moisture could permeate into the interior and create problems. Refer to the American Concrete Institute's Guide to Concrete Floor and Slab Construction (ACI 302.1R) for selection and installation of moisture vapor retarders. It is recommended that a Stegowrap 15 vapor retarder be used where moisture could permeate into the interior and create problems, such as where flooring or floor slab applications will contain moisture sensitive materials (or other slab applications or uses). The vapor retarder should overlay the compacted 4 inch layer of aggregate base and underlying non-expansive soils recommended in this report. It should be noted that placing the PCC slab directly on the vapor retarder may increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab unless a watertight roofing system is in place prior to slab construction to reduce the amount vapor emission through the slab-on-grade. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking.

The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 1745 Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs and ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of floor covering, floor covering adhesive or other slab material applications be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

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

8.8.6 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.

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

8.8.8 The moisture retarding membrane is not required beneath exposed concrete floors, such as exposed warehouses floors, provided that moisture intrusion into the structure is permissible for the design life of the structure.

8.8.9 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a low water-cement ratio of 0.52 lb./lb. 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 structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

8.8.10 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 perimeter foundations; 2) provide an aggregate base layer below the slabs; and 3) the suitability of the loads from construction equipment which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.8.11 If construction traffic will be traveling over the aggregate base material, or the aggregate base will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The proposed compacted subgrade can experience instability under construction traffic resulting in heaving and depressions in the subgrade. Often the aggregate base can reduce the potential for instability under the construction traffic.

8.8.12 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.

8.8.13 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.

8.9 Exterior Slabs-On-Grade

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

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

8.9.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Exterior slabs on grade directly adjacent to the building should be supported on 4 inches of aggregate base over 14 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. The exterior slabs on grade which are not located adjacent to the building should be supported on 4 inches of aggregate base over 8 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. As an alternative, Class 2 aggregate base may be substituted for the imported, non-expansive granular fill soils.

8.9.3 The moisture content of the clay subgrade soils below the non-expansive fill should be verified to be at least one (1) percent above optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.9.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend below the non-expansive fill section underlying the slabs on grade.

8.9.5 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 (minimum of two percent over optimum for the clay subgrade soils) should be in the general contractor's possession prior to placing concrete for exterior flatwork.

8.10 Asphaltic Concrete (AC) Pavements

Recommendations are provided below for new asphaltic concrete pavements planned as part of the new construction and are not for intended for pervious pavements.

8.10.1 The subgrade soils for asphaltic concrete pavements should be prepared as recommended in the "Site Preparation" section of the recommendations in this report.

8.10.2 The following pavement sections are based on an R-value of 12, a minimum asphaltic concrete thickness of 3 inches, and traffic index values ranging from 5.0 to 8.0. 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. It should be noted that if pavements are constructed prior to construction of the buildings, the traffic index value should account for construction traffic.

Table No. 6
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	8.5	12
5.5	3.0	10.5	12
6.0	3.0	12.5	12
6.5	3.5	13.0	12

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
7.0	4.0	14.0	12
7.5	4.0	15.5	12
8.0	4.5	16.5	12

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

8.10.3 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.

8.10.4 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.

8.10.5 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.

8.10.6 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.

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

8.10.8 It is recommended that the base 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.

8.10.9 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.

8.10.10 The asphalt concrete should comply with the requirements for a Type A asphalt concrete as described in Section 39 of the latest California Department of Transportation (Caltrans) Standard Specification, or the requirements of the governing agency, whichever is more stringent.

8.11 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.

8.11.1 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.

8.11.2 The following preliminary Portland cement concrete pavement sections have been prepared for average daily truck traffic ranging from 1.9 to 21 trucks per day which corresponds to Traffic Indices ranging from 6 to 8. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic. The design thicknesses were 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, 6) truck loading consisting of 1 single axle load of 18 kips, and 5) a maximum single axle weight of 12,000 pounds, and a maximum tandem axle weight of 36,000 pounds.

Table No. 7
Two-Layer Portland Cement Concrete Pavements

Traffic Index	ADTT (Trucks/day)	PCC thickness (inches)	Aggregate Base[1] (inches)	Compacted Subgrade[2] (inches)
6.0	1.9	6.0	6.0	12.0
7.0	7	6.5	6.0	12.0
8.0	21	6.5	6.0	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)

8.11.3 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.

8.11.4 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.

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

8.11.6 Specifications for the concrete mixtures used in the PCC pavement to reduce the effects of excessive shrinkage (such as curling and excessive shrinkage at joints), including maximum water requirements for the concrete mix, allowable shrinkage limits, curing methods, etc. should be provided by the designer/detailer of the PCC slabs. In addition, as noted in Section 8.11.5, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement to maintain stability. The minimum PCC thickness noted in this report assumes aggregate interlock occurs at contraction joints. However, curling and excessive shrinkage can disengage aggregate interlock and allow greater pavement deflection at free edges. The design engineer should decide if aggregate interlock is appropriate or specify joint reinforcement.

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

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

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

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

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

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

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

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

8.12 Temporary Excavations

8.12.1 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, and 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.

8.12.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 1.5 to 1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be supported by engineered shoring systems.

8.12.3 In no case should non-shored excavations extend below a 1.5H to 1V zone below existing utilities, top of foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 1.5 H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.12.4 Shoring systems should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Moore Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.12.5 Surface sheet flow drainage shall be directed away from the tops of all excavations. Positive drainage shall be established and maintained throughout the construction process.

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

8.13 Utility Trenches

8.13.1 This report recommends an imported, non-expansive soil section below concrete slabs-on-grade within the building pad preparation limits and below exterior concrete slabs. After trenching of utilities in areas of non-expansive fill, the non-expansive sections should be re-established below the slabs and PCC pavements to match the adjacent sections. In the event the excavated non-expansive soils are mixed with onsite clayey soils, these materials will not be allowed as trench backfill within the non-expansive zone and may be replaced with aggregate base. The contractor is responsible to conduct the excavation and trench backfilling to meet the requirements of this report, including the upper portions of the trenches to reestablish the non-expansive sections to match the adjacent sections.

8.13.2 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 90 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.

8.13.3 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 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, or on-site materials moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted to a minimum of 90 percent relative compaction. The upper 12 inches of trench backfill below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.13.4 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
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80

8.13.5 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.

8.13.6 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 for granular soils, or moisture conditioned to between one (1) and four (4) percent above the optimum moisture content and compacted to at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557 for on-site clayey soils. The upper 12 inches of trench backfill below the aggregate base for pavements should be compacted to at least 95 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.

8.13.7 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.

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

8.13.9 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.

8.13.10 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.

8.13.11 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.

8.14 <u>Corrosion Protection</u>

8.14.1 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 analytical results of sample analyses indicate the samples had a resistivity value of 3,600 and 2,700 and 2,000 ohms-centimeter, with pH values of 8.7, 8.8 and 8.5, respectively. Based on the resistivity value, the soils exhibit a "corrosive" to "highly corrosive"corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "highly 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.

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

8.14.3 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. In the event Moore Twining does not conduct the observations and testing of the building pad preparation, reports signed by a registered geotechnical engineer documenting the earthwork inspections, in-place density testing and certification of the pad as meeting the project requirements should be provided to our firm for review.

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

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

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

10.5 If Moore Twining is not retained to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of the earthwork preparation 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 satisfactory to the owner and review this report. After their review, the firm should, in writing, state that they 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 the Anticipated Construction section of this report. 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. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

DRAFT

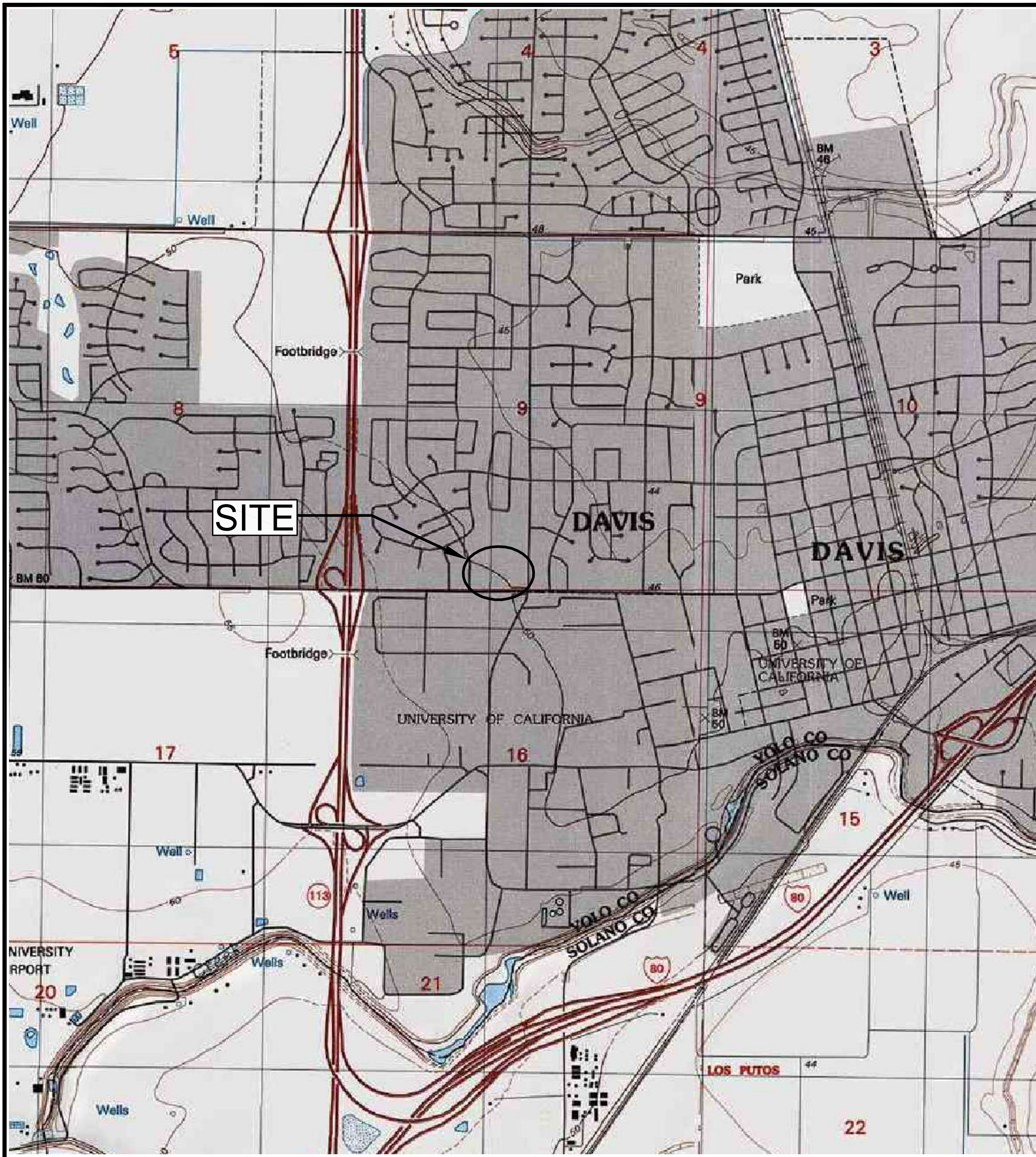
Allen H. Harker, PG
Professional Geologist

DRAFT

Read L. Andersen, RGE
Manager

APPENDIX A

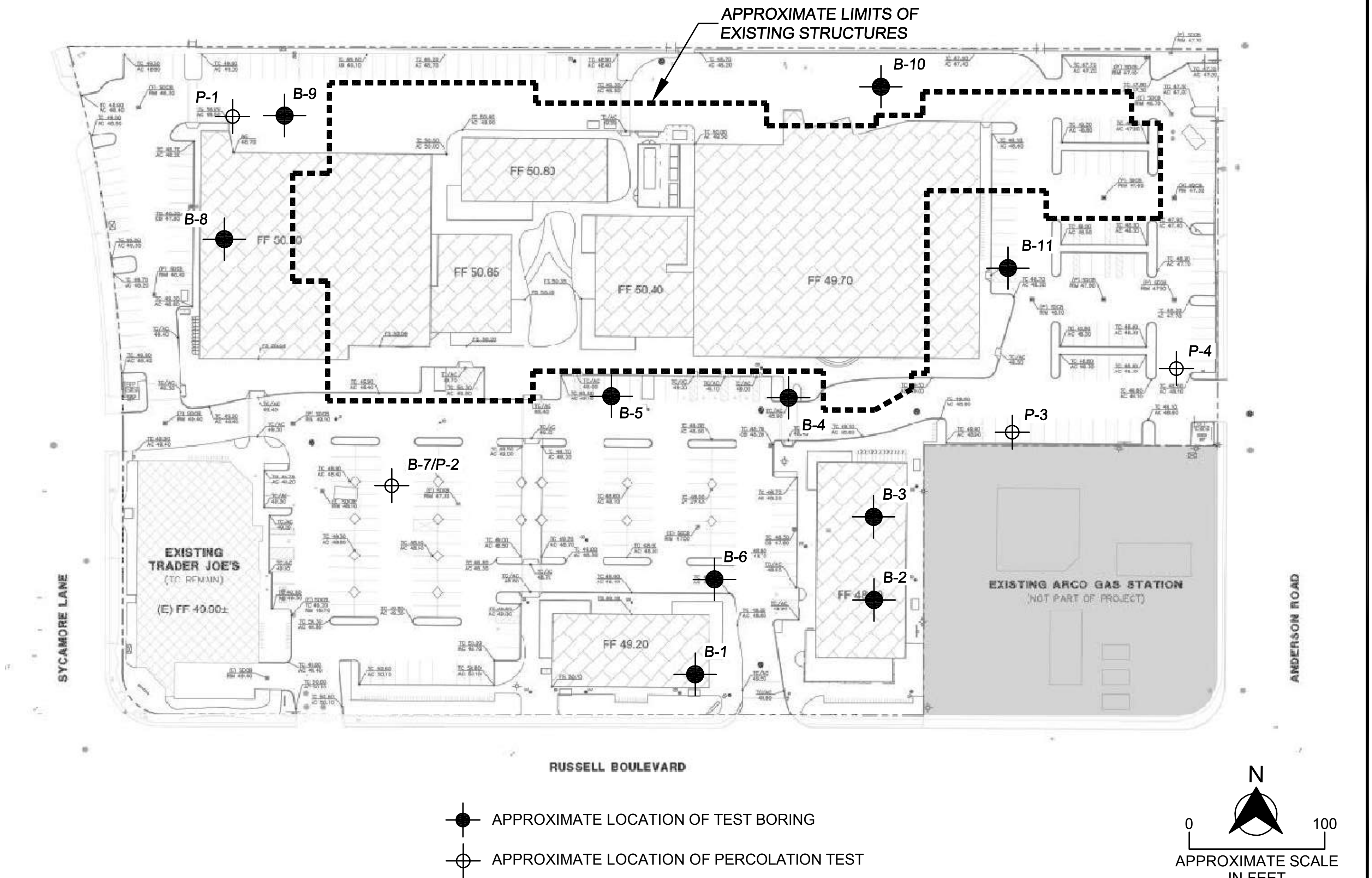
DRAWINGS



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
MERRITT, CALIFORNIA QUADRANGLE 1992

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED RETAIL DEVELOPMENT NORTHEAST CORNER OF SYCAMORE LANE AND RUSSELL BOULEVARD DAVIS, CALIFORNIA	FILE NO.: 12701-01-01	DATE: 11/16/2022	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H12701.01	DRAWING NO. 1	



TEST BOING AND PERCOLATION TEST BORING LOCATION MAP
PROPOSED RETAIL DEVELOPMENT
NORTHEAST CORNER OF SYCAMORE LANE AND RUSSELL BOULEVARD
DAVIS, CALIFORNIA

FILE NO. 12701-01-01	DATE DRAWN: 11/16/2022
DRAWN BY: RM	APPROVED BY:
PROJECT NO. H12701.01	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.

Test Boring: B-1

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			
	5/6 22/6 24/6	AB	Aggregate Base = 4 inches	From 1-2.5': Ring Sample Disturbed Gravel = 8.4% Sand = 44.6% -200 = 47.0% LL = 25 PI = 12	46	6.6
	10/6 5/6 3/6 3/6	FILL SM	FILL - CLAYEY SAND; medium dense, damp, fine to medium grained, light brown, with a little fine gravel		8	3.5
5	6/6 9/6 8/6	CL	SILTY SAND; loose, damp, fine grained, brown, trace gravel		17	7.0
			SANDY LEAN CLAY; very stiff, damp, low plasticity, brown	From 1-3.5': EI = 33 pH = 8.7 SR = 3,600 ohm-cm Cl = 0.0012% SS = 0.0035%		
10	5/6 7/6 9/6		Increase in plasticity, decrease in sand	From 5-6.5': DD = 87.2 pcf	16	
15	4/6 6/6 8/6		Stiff		14	
20	3/6 4/6 4/6		Medium stiff, increase in plasticity		8	
			Bottom of Boring B-1 at 20 feet BSG			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

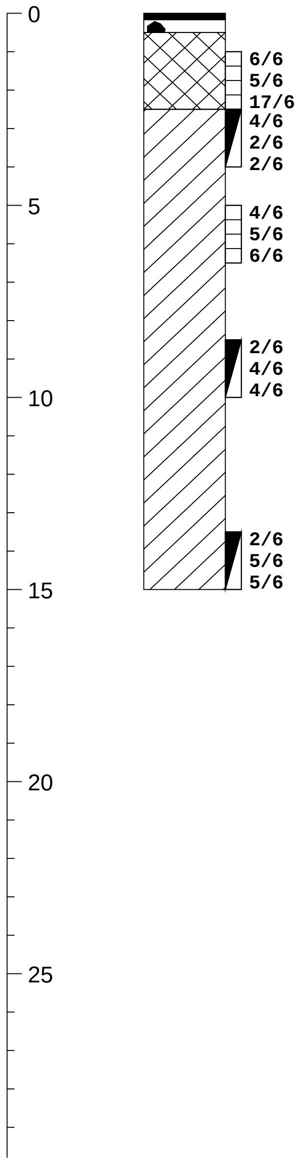
Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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 %
		AC	Asphalt Concrete = 2 inches			
		AB	Aggregate Base = 4 inches	DD = 110.2 pcf	22	14.5
		FILL	FILL - SANDY LEAN CLAY; very stiff, moist, low plasticity, dark brown	From 1-3.5': EI = 64	4	17.4
		CL	SANDY LEAN CLAY; soft, moist, low plasticity, dark brown			
			LEAN CLAY; medium stiff, moist, low to medium plasticity, dark brown	DD = 101.4 pcf	11	14.6
					8	
			Increase in moisture content		10	
			Bottom of Boring B-2 at 15 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-3

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			
		AB	Aggregate Base = 4 inches			
	3/6 8/6 8/6	FILL	FILL - SANDY LEAN CLAY; very stiff, moist, low medium plasticity, dark brown, trace gravel		16	12.0
	2/6 5/6 6/6	CL	SANDY LEAN CLAY; stiff, moist, medium plasticity, dark brown	DD = 101.9 pcf	11	15.9
5						
	4/6 8/6 12/6		LEAN CLAY	DD = 109.0 pcf	20	18.6
10	4/6 3/6 3/6				6	
15	2/6 3/6 5/6		Medium stiff		8	
	3/6 2/6 4/6		Medium stiff		6	
20			Bottom of Boring B-3 at 20 feet			
25						

Notes:

Figure Number

Test Boring: B-4

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 6, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 3 inches			
	4/6 4/6 4/6	AB	Aggregate Base = 4 inches	DD = 103.4 pcf	8	11.7
	2/6 3/6 2/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, brown		5	21.9
			Decrease in sand content, sharp increase in moisture content			
5	4/6 6/6 9/6		LEAN CLAY WITH SAND; stiff, trace gravel	DD = 101.2 pcf	15	16.7
10	2/6 4/6 3/6		LEAN CLAY; medium stiff		7	
15	3/6 5/6 4/6		Stiff		9	
20	2/6 3/6 1/6		Medium stiff		4	
			Bottom of Boring B-4 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-5

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: 44 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 3 inches			
		AB	Aggregate Base = 4 inches			
	3/6 3/6 3/6	CL	LEAN CLAY WITH SAND; medium stiff, moist, high plasticity, dark brown, trace fine gravel	From 1-3.5': Gravel = 0.7% Sand = 24.0% -200 = 75.3% LL = 40 PI = 21 pH = 8.8 SR = 2,700 ohm-cm Cl = 0.00067% SS = 0.0027%	6	12.5
5	3/6 3/6 5/6		LEAN CLAY; medium stiff, moist, medium plasticity, dark brown		8	12.5
	7/6 11/6 15/6		Very stiff	From 3.5-5': DD = 98.3 pcf Sand = 8.0% -200 = 92.0% LL = 33 PI = 15	26	16.0
10	3/6 5/6 4/6		Stiff, increase in moisture and plasticity	From 7-8.5': DD = 105.3 pcf	9	
15	3/6 3/6 5/6		LEAN CLAY; medium stiff		8	19.8
20	2/6 3/6 6/6		Stiff		9	
25	4/6 4/6 6/6		Decrease in moisture content		10	15.2

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: 44 feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	4/6 6/6 7/6		LEAN CLAY WITH SAND		13	
35	4/6 6/6 7/6		LEAN CLAY		13	19.4
40	3/6 6/6 8/6		Increase in moisture content		14	19.8
45	5/6 8/6 10/6	SM	SILTY SAND; medium dense, wet, fine grained, brown, trace fine gravel	Gravel = 4.8% Sand = 51.8% -200 = 43.4% LL = Non-viscous PI = Non-plastic	18	24.3
	7/6 21/6 26/6		Dense		47	22.6
50			Bottom of Boring B-5 at 50 feet			
55						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-6

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 6, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			
	5/6 3/6 3/6	AB	Aggregate Base = 4 inches	From 1-3.5': R-value = 16	6	8.6
		CL	SANDY LEAN CLAY; medium stiff, damp, low plasticity, brown			
5	2/6 3/6 4/6		LEAN CLAY WITH SAND; increase in plasticity		7	
	3/6 5/6 6/6		Stiff		11	
10			Bottom of Boring B-6 at 10 feet			
15						
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-7/P-2

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			5.5
	12/6 9/6 5/6	AB	Aggregate Base = 4 inches	From 1-2.5': Sand = 59.1% -200 = 40.9% LL = Non-viscous PI = Non-plastic	14	
		FILL	FILL - SILTY SAND; medium dense, moist, fine grained, brown			
5	3/6 6/6 7/6	CL	NATIVE - SANDY LEAN CLAY; stiff, moist, low to medium plasticity, dark brown, trace gravel		13	
10	3/6 2/6 2/6	SC	CLAYEY SAND; loose, moist, fine to coarse grained, dark brown, with some fine gravel	From 8.5-10': Gravel = 10.4% Sand = 40.8% -200 = 48.8% LL = 33 PI = 21	4	12.2
			Bottom of Boring/Percolation Test Boring B-7/P-2 at 10 feet			
15						
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-8

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 6, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 3 inches			
	5/6 7/6 6/6	AB	Aggregate Base = 4 inches	DD = 99.9 pcf	13	15.2
	2/6 2/6 2/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, dark brown LEAN CLAY; soft, low to medium plasticity	From 1-3.5': EI = 67	4	17.6
5	5/6 9/6 12/6		LEAN CLAY WITH SAND; stiff	DD = 101.8 pcf	21	14.8
	3/6 2/6 2/6		Soft, trace gravel		4	
10						
	2/6 4/6 6/6		LEAN CLAY; stiff, no gravel		10	
15						
	2/6 4/6 5/6		Increase in plasticity		9	
20			Bottom of Boring B-8 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-9

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 3 inches			
		AB	Aggregate Base = 4 inches			
	2/6 3/6 4/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, brown		7	18.4
	3/6 6/6 5/6		Increase in sand content	DD = 100.9 pcf	11	16.4
5						
	7/6 11/6 12/6		LEAN CLAY; stiff	DD = 109.7 pcf	23	17.1
10	4/6 3/6 2/6		Medium stiff		5	
15	3/6 3/6 5/6		Increase in moisture content and plasticity		8	
	2/6 5/6 5/6		Stiff		10	
20			Bottom of Boring B-9 at 20 feet			
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-10

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 5 inches			
		AB	Aggregate Base = 4 inches	From 1-3.5': R-value = 12		
	9/6 4/6 4/6	CL	SANDY LEAN CLAY; medium stiff, moist, low plasticity, dark brown, trace gravel		8	17.3
	2/6 1/6 2/6		Soft		3	21.0
5	4/6 5/6 9/6		Stiff	DD = 98.7 pcf ø = 29° c = 80 psf	14	17.0
10	5/6 7/6 9/6		Increase in plasticity	DD = 106.3 pcf	16	14.7
15	2/6 3/6 4/6		LEAN CLAY; medium stiff		7	
20	4/6 4/6 5/6		Stiff, increase in moisture content		9	
25	4/6 8/6 8/6		Very stiff		16	
			Bottom of Boring B-10 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-11

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

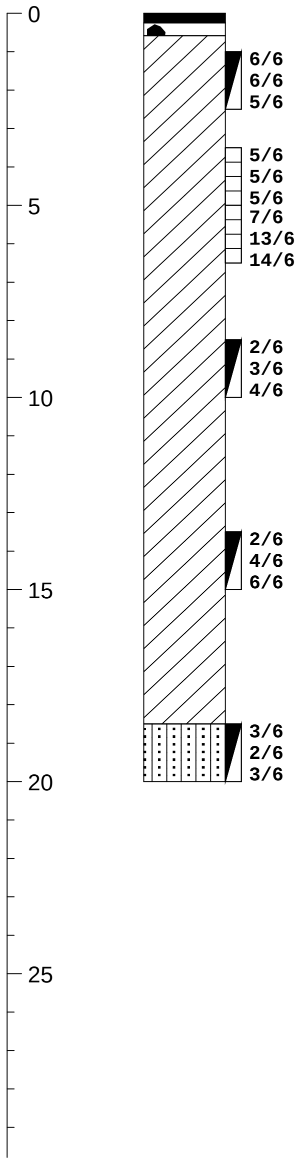
Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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 %
		AC	Asphalt Concrete = 3 inches			
	6/6 6/6 5/6	AB	Aggregate Base = 4 inches	From 1-3.5': Gravel = 1.4% Sand = 16.2% -200 = 82.4% LL = 42 PI = 27 pH = 8.5 SR = 2,000 ohm-cm Cl = 0.006% SS = 0.016%	11	9.5
		CL	LEAN CLAY WITH SAND; stiff, moist, high plasticity, brown, wiht trace fine gravel			
	5/6 5/6 5/6		Decrease in plasticity		15	10.1
	7/6 13/6 14/6		Very stiff, sharp increase in moisture content and plasticity, dark brown		27	20.5
	2/6 3/6 4/6		Medium stiff	From 3.5-5': DD = 88.6 pcf From 5-6.5: DD = 92.2 pcf	7	
	2/6 4/6 6/6		Stiff, increase in moisture and plasticity		10	
	3/6 2/6 3/6	SM	SILTY SAND; loose, moist, fine to coarse grained, dark brown, with a little fine gravel	From 18.5-20': Gravel = 5.2% Sand = 77.8% -200 = 17.0% LL = Non-viscous PI = Non-plastic	5	7.1
			Bottom of Boring B-11 at 20 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-1

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

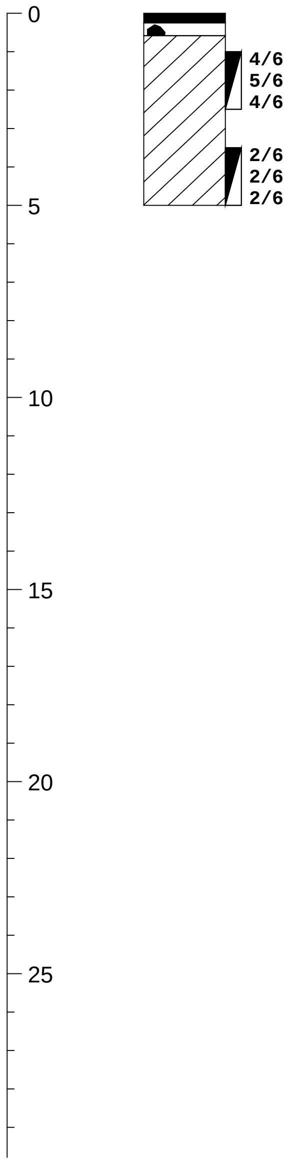
Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

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 %
		AC	Asphalt Concrete = 3 inches			
		AB	Aggregate Base = 4 inches		9	11.7
		CL	SANDY LEAN CLAY; stiff, moist, low to medium plasticity, brown			
			LEAN CLAY; soft		4	16.8
			Bottom of Percolation Test Boring P-1 at 5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-3

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 5, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			
	4/6 6/6 6/6	AB	Aggregate Base = 4 inches		12	6.9
		CL	SANDY LEAN CLAY; stiff, moist, low plasticity, brown			
	3/6 4/6 6/6		LEAN CLAY		10	
5						
	3/6 4/6 6/6		Increase in moisture content, decrease in plasticity		10	15.1
10			Bottom of Percolation Test Boring P-3 at 10 feet			
15						
20						
25						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-4

Project: Proposed Retail Development in Davis

Project Number: H12701.01

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: J.F.

Date: October 4, 2022

Elevation: N/A

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		AC	Asphalt Concrete = 2 inches			
	4/6 4/6 7/6	AB	Aggregate Base = 3 inches		11	8.3
		CL	SANDY LEAN CLAY; stiff, moist, low plasticity, dark brown			
	4/6 3/6 3/6	GC	CLAYEY GRAVEL WITH SAND; medium stiff, moist, fine gravel, brown	From 3.5-5': Gravel = 45.3% Sand = 41.4% -200 = 13.3% LL = 28 PI = 13	6	8.3
5			Bottom of Percolation Test Boring P-4 at 5 feet			
10						
15						
20						
25						

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Asphalt concrete
	Aggregate base
	Fill
	Silty Sand
	Lean Clay
	Clayey Sand
	Clayey gravel

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

Notes:

1. Exploratory borings were drilled between 10/4/22 and 10/6/22 using a CME 75 drill rig equipped with 6-5/8" outside diameter hollow stem augers.

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

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

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

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

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

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

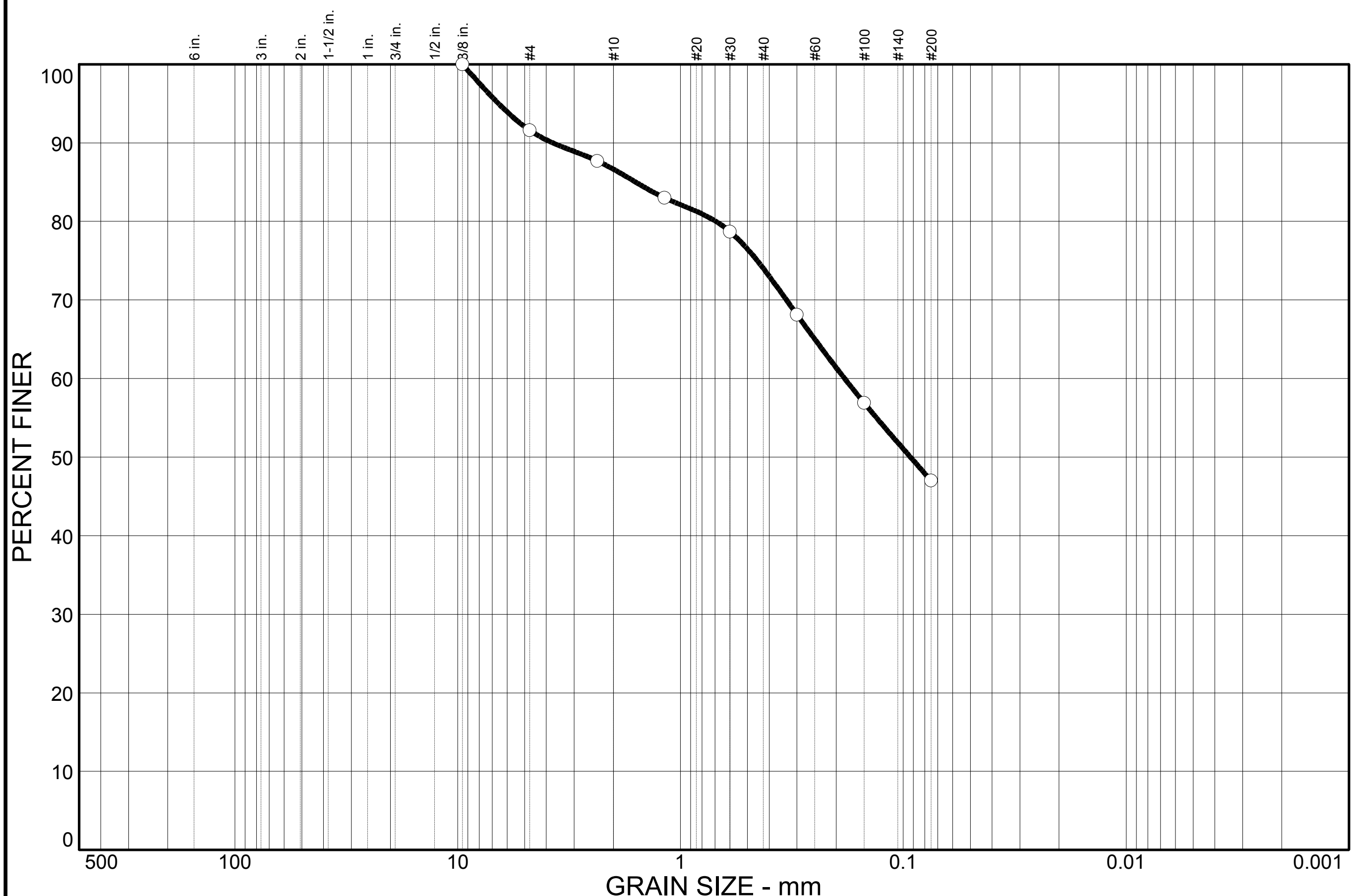
APPENDIX C

RESULTS OF LABORATORY TESTS

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

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

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	8.4	5.0	12.6	27.0	47.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	91.6		
#8	87.7		
#16	83.0		
#30	78.7		
#50	68.1		
#100	56.9		
#200	47.0		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 13 LL= 25 PI= 12

Coefficients

D_{85}= 1.59 D_{60}= 0.183 D_{50}= 0.0930
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 1-3.5'

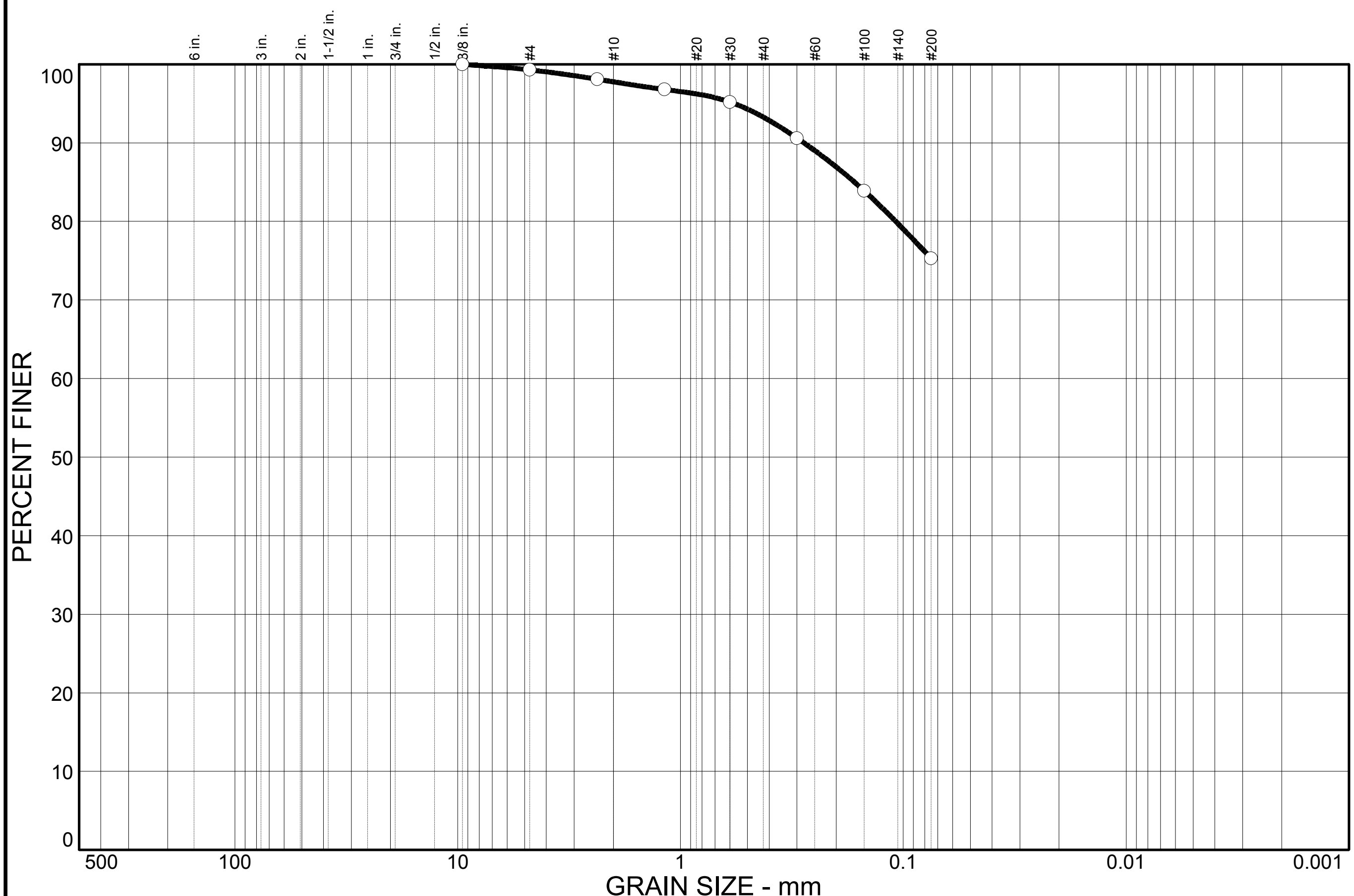
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.7	1.5	4.5	18.0	75.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.3		
#8	98.1		
#16	96.8		
#30	95.2		
#50	90.6		
#100	83.9		
#200	75.3		

* (no specification provided)

Material Description

Lean clay with sand

Atterberg Limits

PL= 19 LL= 40 PI= 21

Coefficients

D_{85}= 0.166 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample No.: B-5 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 1-3.5'

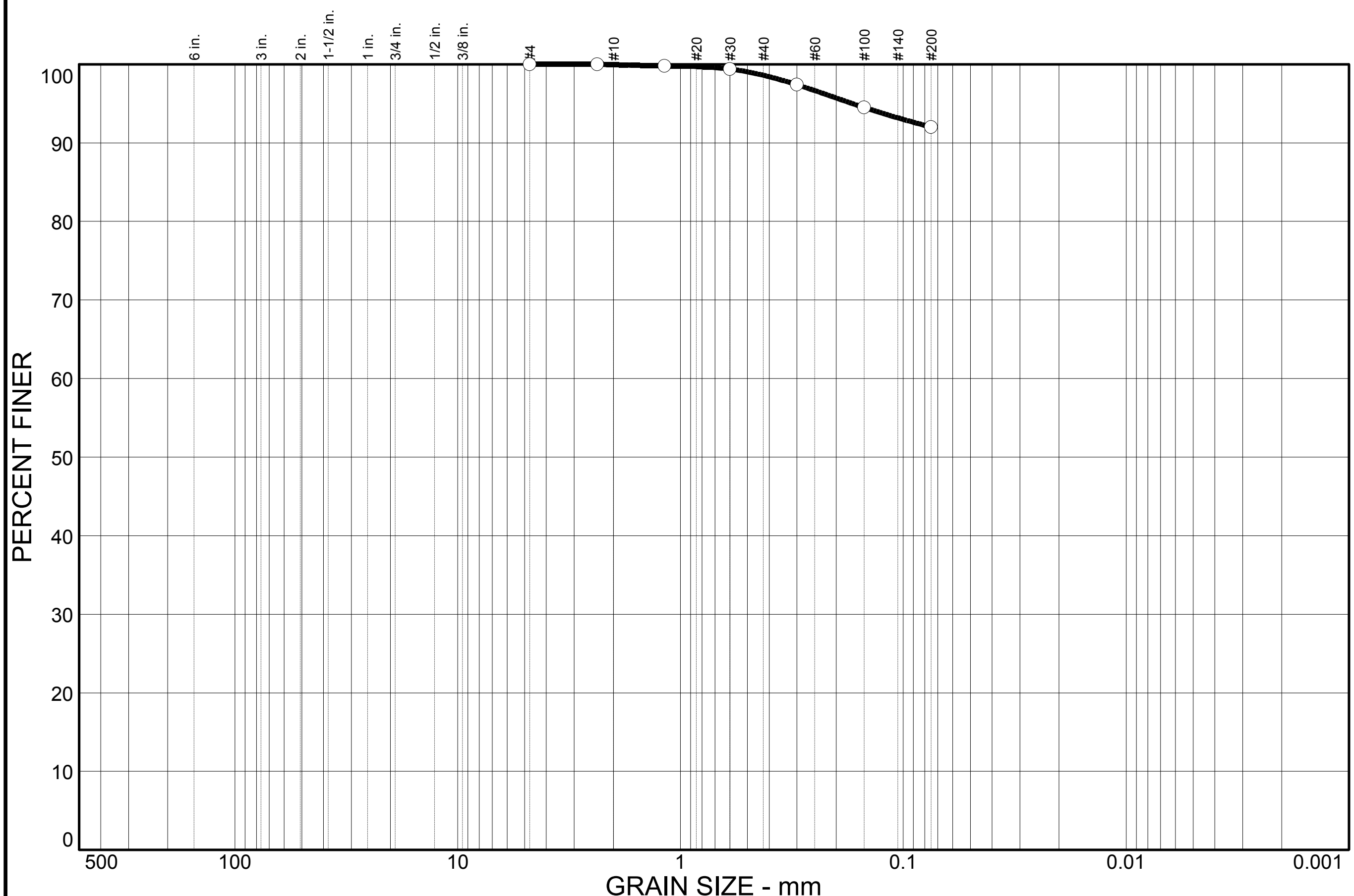
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.1	1.3	6.6	92.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	99.8		
#30	99.4		
#50	97.4		
#100	94.5		
#200	92.0		

* (no specification provided)

Material Description

Lean clay

Atterberg Limits

PL= 18 LL= 33 PI= 15

Coefficients

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

Classification

USCS= CL AASHTO=

Remarks

Sample No.: B-5 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 3.5-5'

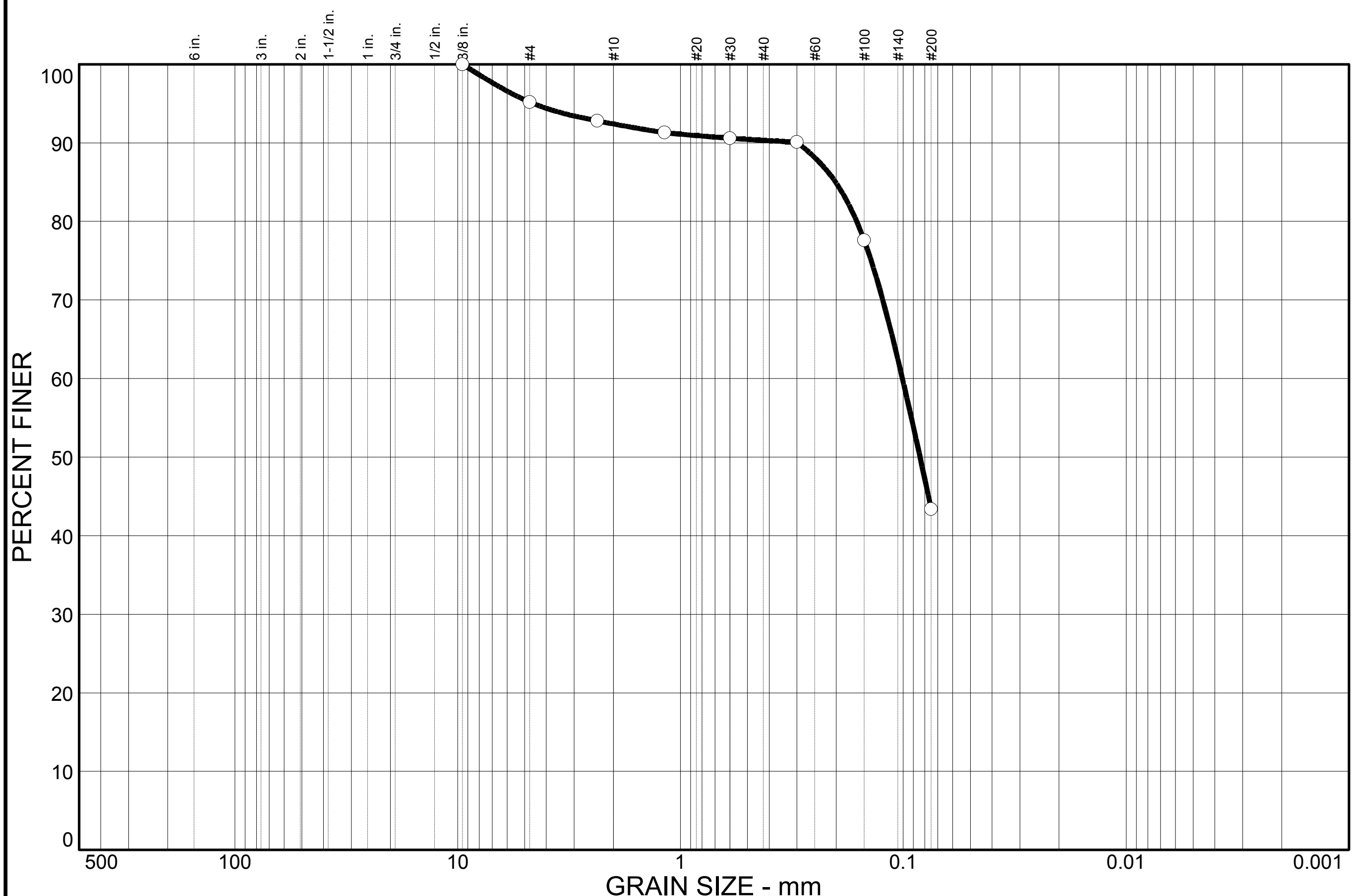
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	4.8	2.8	2.1	46.9	43.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	95.2		
#8	92.8		
#16	91.3		
#30	90.6		
#50	90.1		
#100	77.6		
#200	43.4		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.201 D_{60}= 0.101 D_{50}= 0.0842
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-5 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 45-46.5'

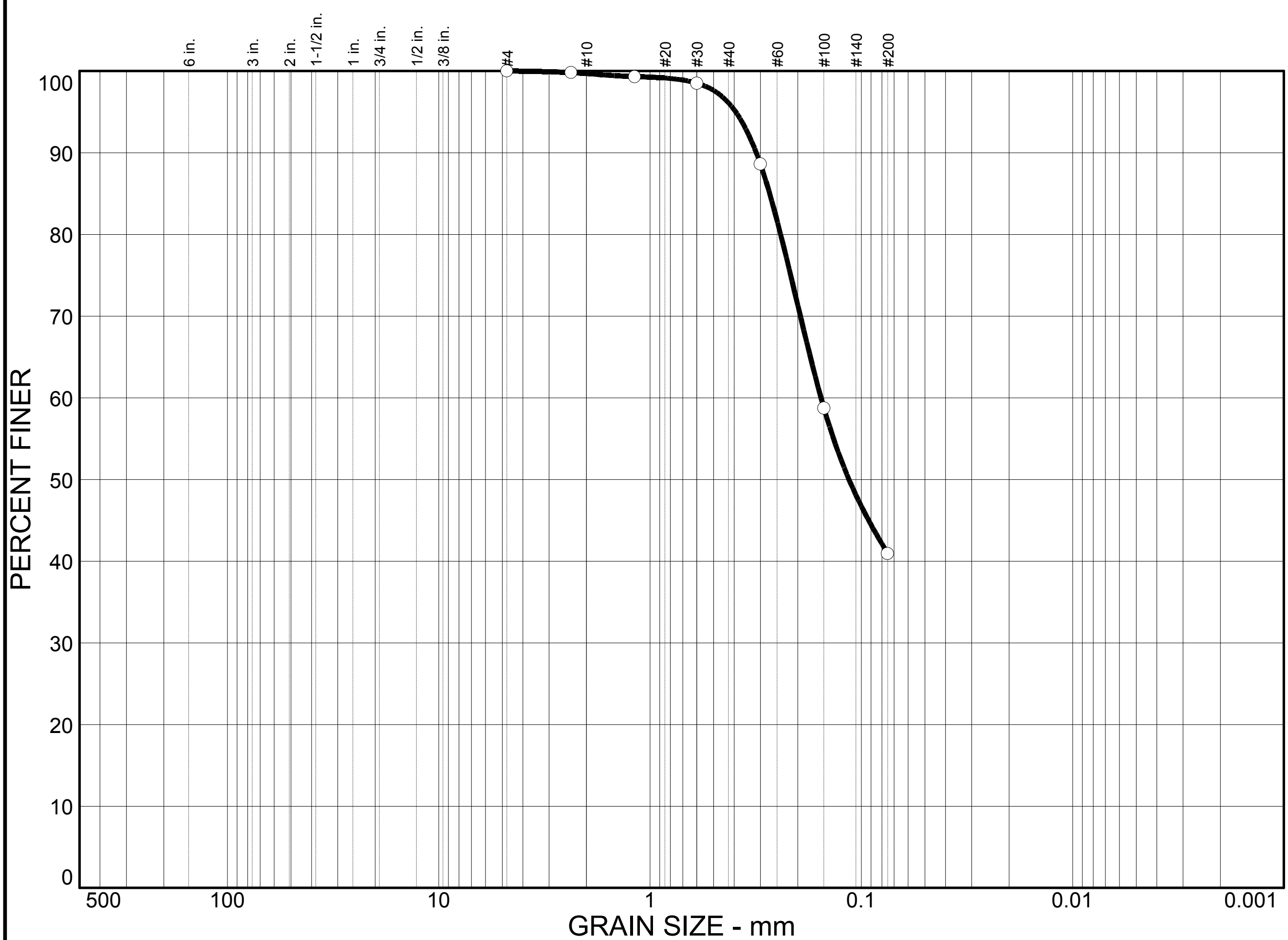
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.3	3.6	55.2	40.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#16	99.3		
#30	98.5		
#50	88.6		
#100	58.7		
#200	40.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.271 D_{60}= 0.155 D_{50}= 0.114
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

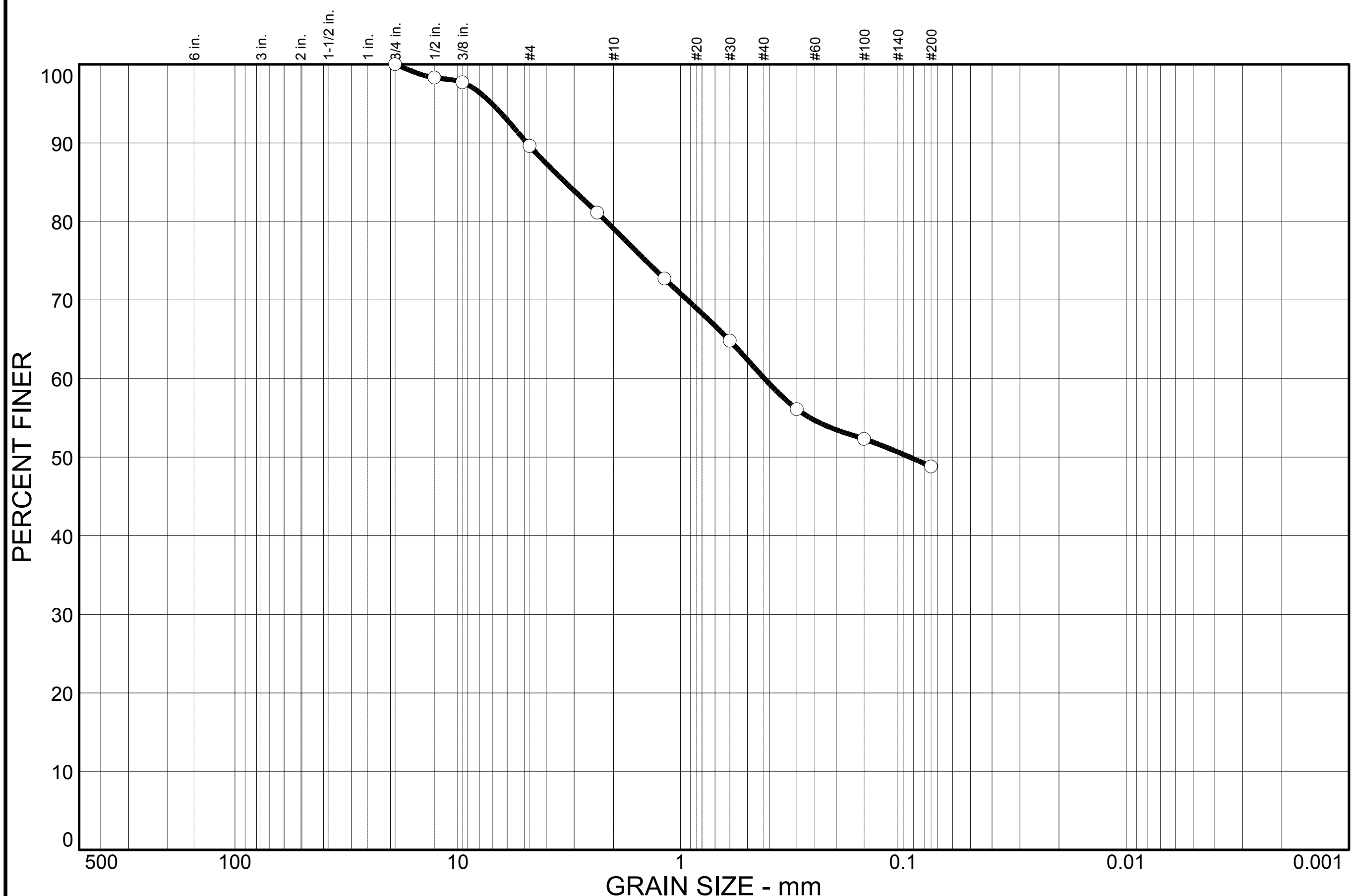
USCS= SM AASHTO=

Remarks

Sample No.: B-7/P2 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 1-2.5'

Moore Twining Associates, Inc. Fresno, CA	**Client:** California Property Owner I, LLC **Project:** Proposed Retail Development in Davis **Project No:** H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	10.4	10.5	19.0	11.3	48.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.3		
3/8 in.	97.7		
#4	89.6		
#8	81.1		
#16	72.7		
#30	64.8		
#50	56.1		
#100	52.3		
#200	48.8		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 12 LL= 33 PI= 21

Coefficients

D_{85}= 3.30 D_{60}= 0.420 D_{50}= 0.0936
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample No.: B-7/P-2 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 8.5-10'

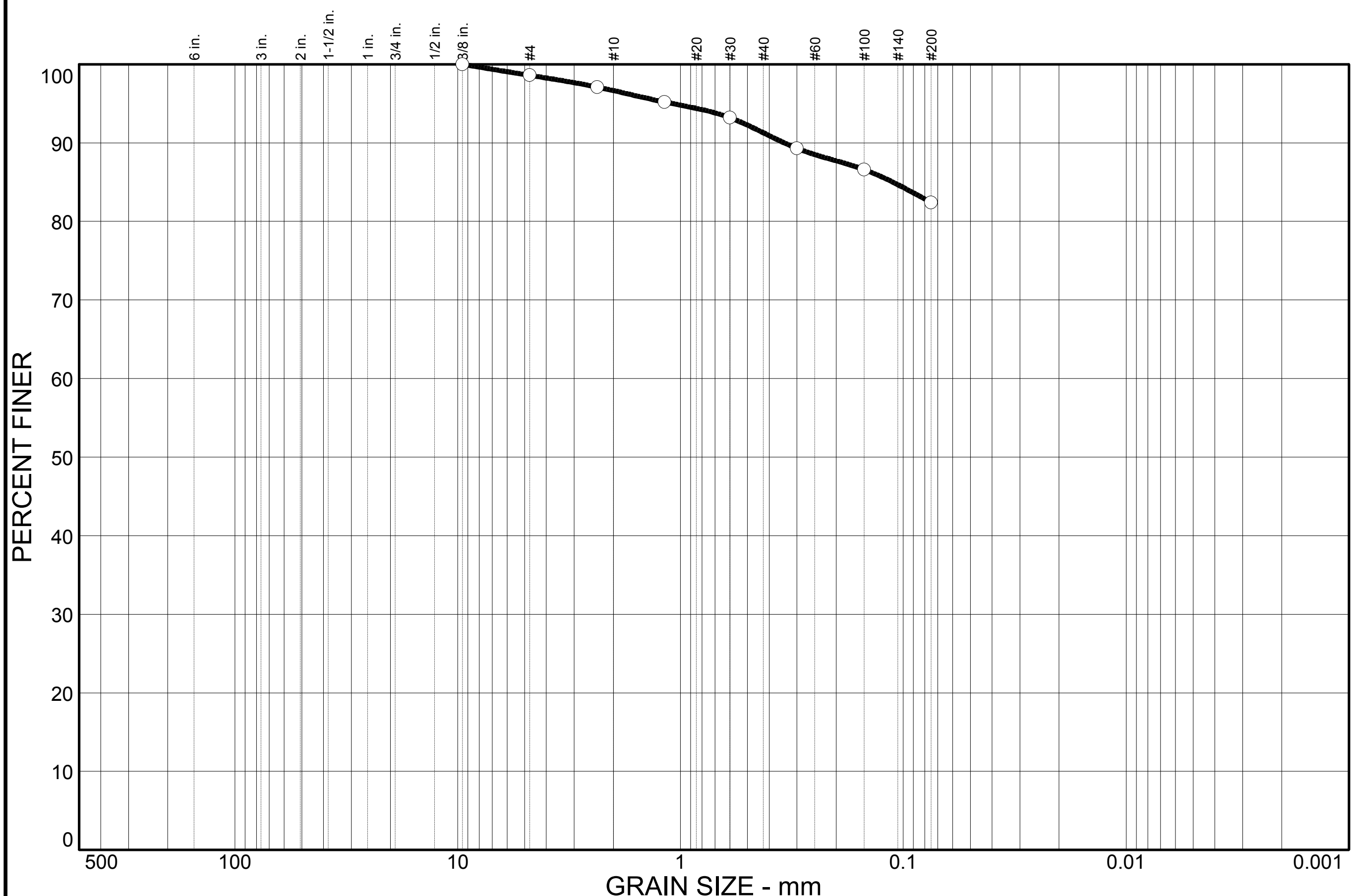
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.4	1.9	5.4	8.9	82.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.6		
#8	97.1		
#16	95.2		
#30	93.2		
#50	89.3		
#100	86.6		
#200	82.4		

* (no specification provided)

Material Description

Lean clay with sand

Atterberg Limits

PL= 15 LL= 42 PI= 27

Coefficients

D_{85}= 0.111 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

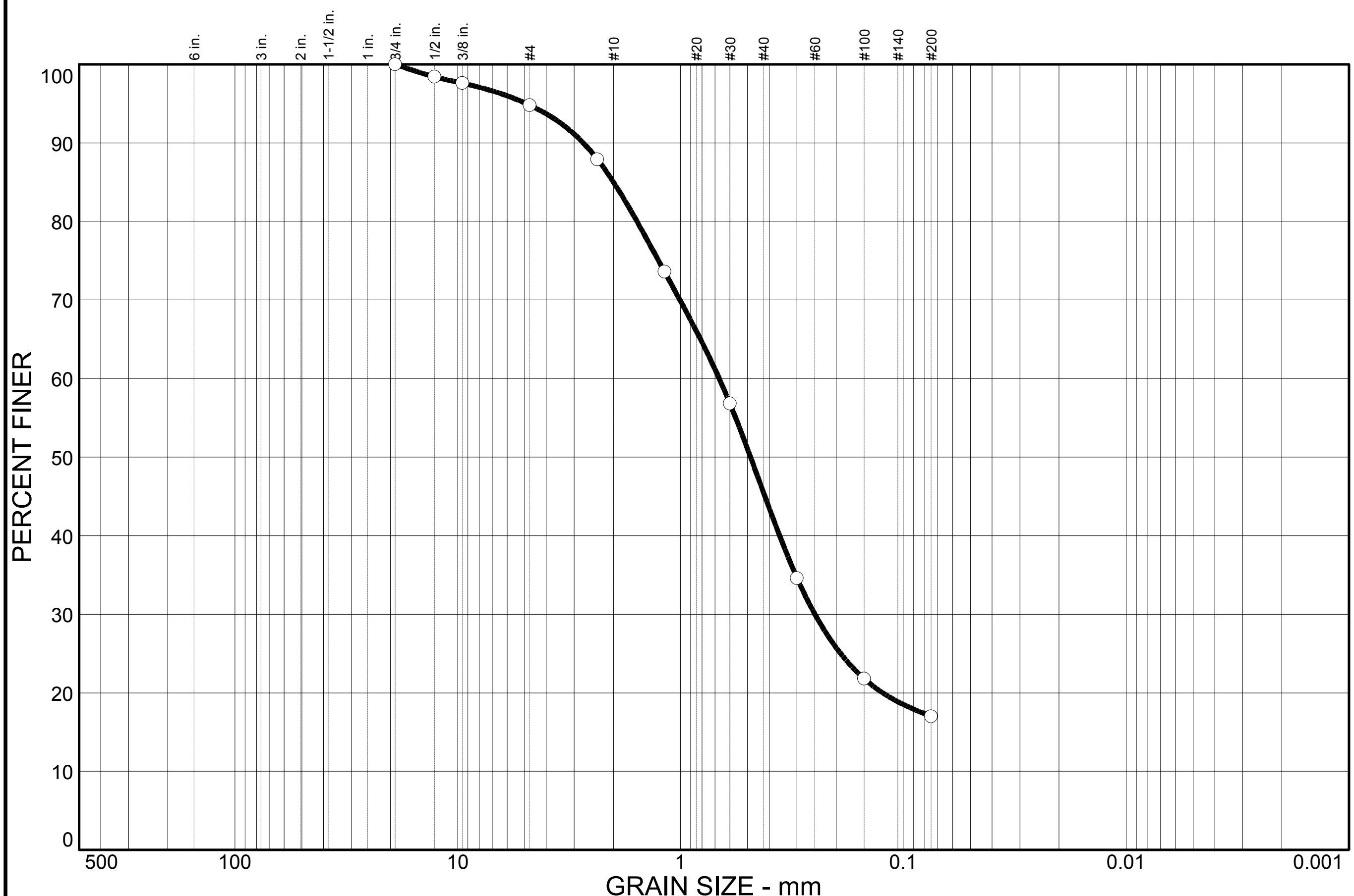
Remarks

Sample No.: B-11 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 1-3.5'

Moore Twining Associates, Inc.
Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis
Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.2	9.8	39.5	28.5	17.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.4		
3/8 in.	97.6		
#4	94.8		
#8	87.9		
#16	73.6		
#30	56.8		
#50	34.6		
#100	21.8		
#200	17.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 2.00 D_{60}= 0.671 D_{50}= 0.485
D_{30}= 0.249 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

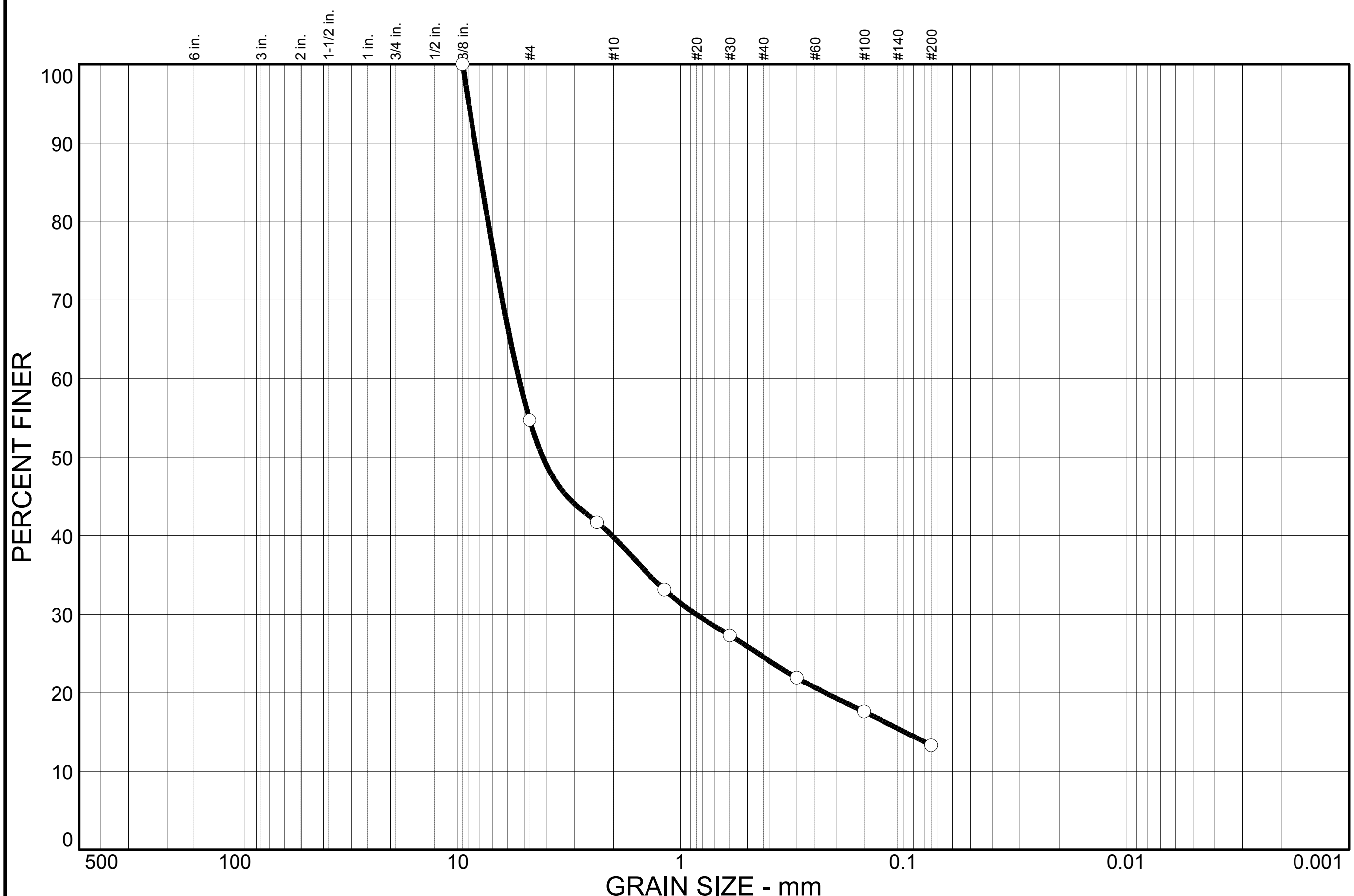
Sample No.: B-11 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 18.5-20'

Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis
Project No: H12701.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	45.3	14.9	15.3	11.2	13.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	54.7		
#8	41.7		
#16	33.1		
#30	27.3		
#50	21.9		
#100	17.6		
#200	13.3		

* (no specification provided)

Material Description

Clayey gravel with sand

Atterberg Limits

PL= 15 LL= 28 PI= 13

Coefficients

D_{85}= 7.82 D_{60}= 5.32 D_{50}= 4.14
D_{30}= 0.852 D_{15}= 0.0982 D_{10}=
C_u= C_c=

Classification

USCS= GC AASHTO=

Remarks

Sample No.: P-4 **Source of Sample:** **Date:** 10/4/22
Location: **Elev./Depth:** 3.5-5'

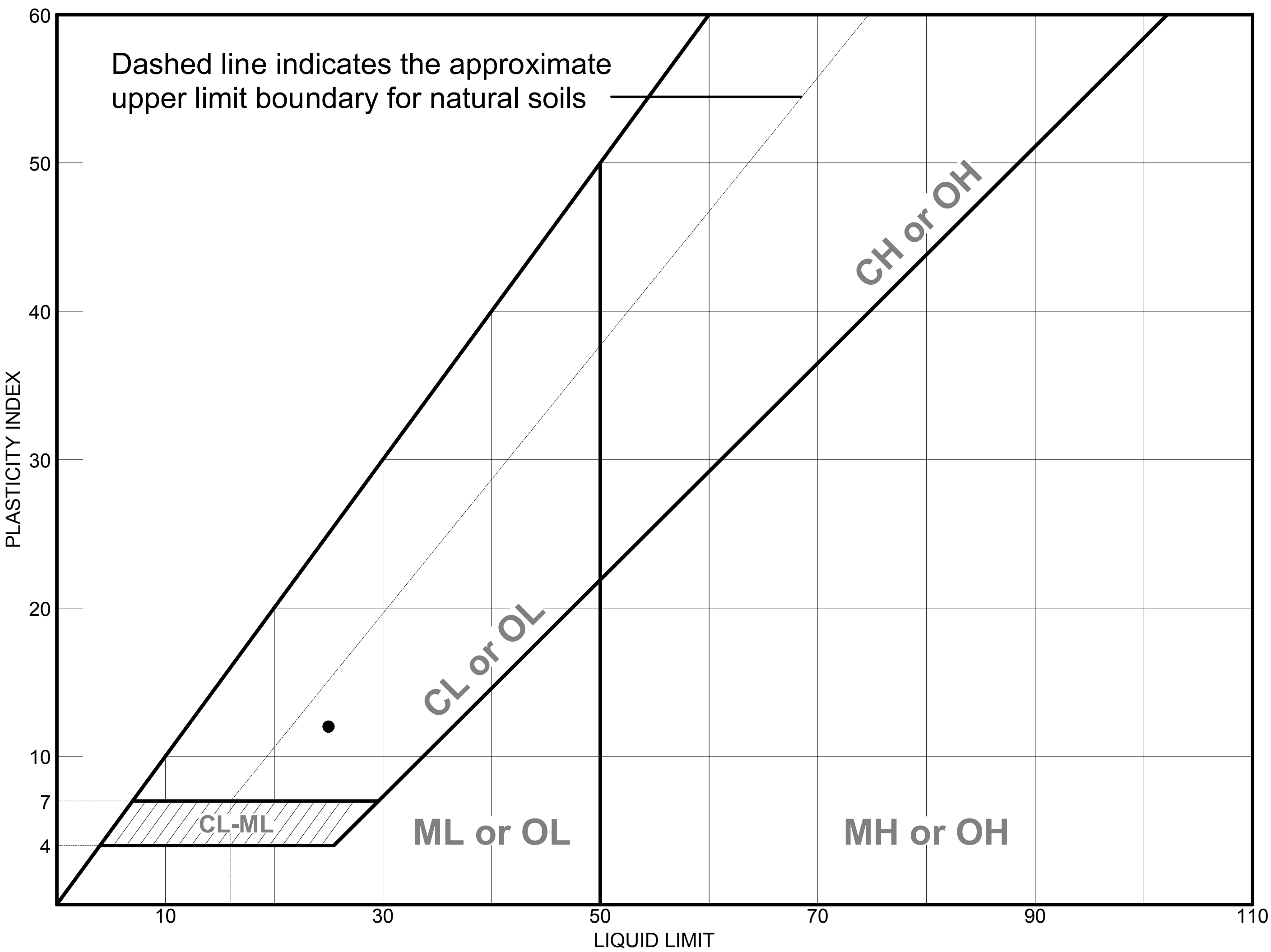
Moore Twining Associates, Inc.

Fresno, CA

Client: California Property Owner I, LLC
Project: Proposed Retail Development in Davis

Project No: H12701.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	25	13	12	74.0	47.0	SC

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

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

Remarks:

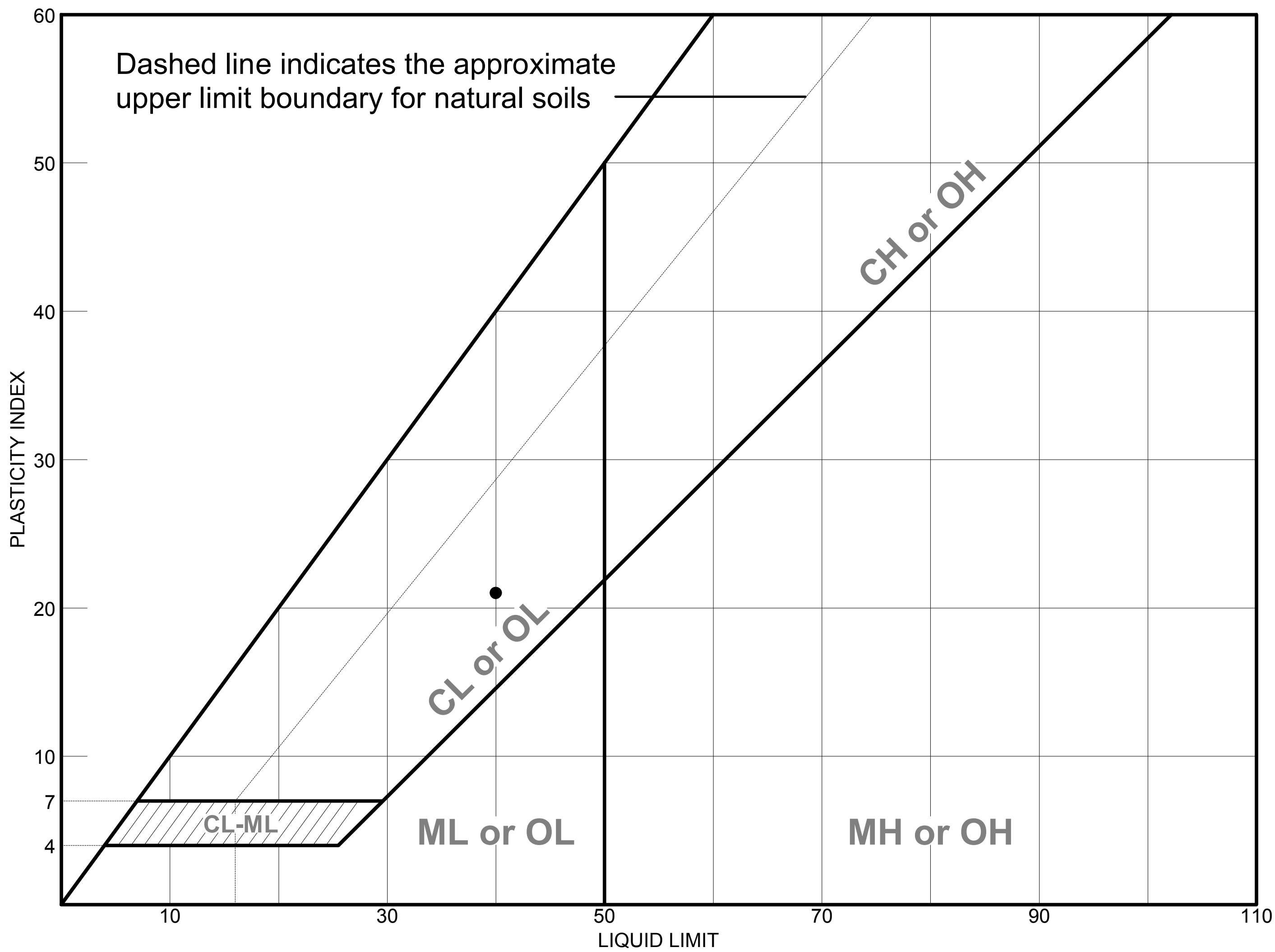
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay with sand	40	19	21	93.3	75.3	CL

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

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

Remarks:

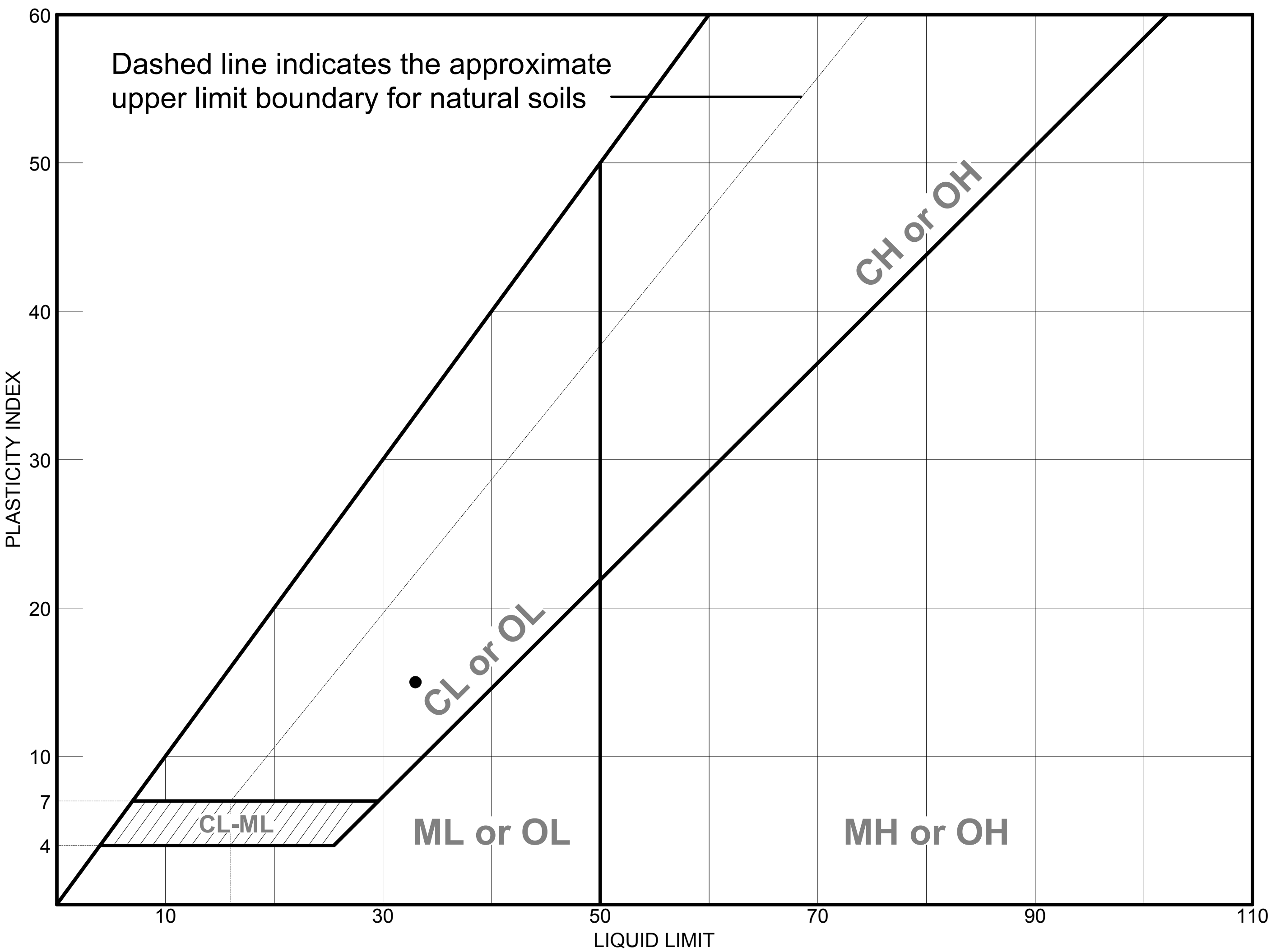
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay	33	18	15	98.6	92.0	CL

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

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

Remarks:

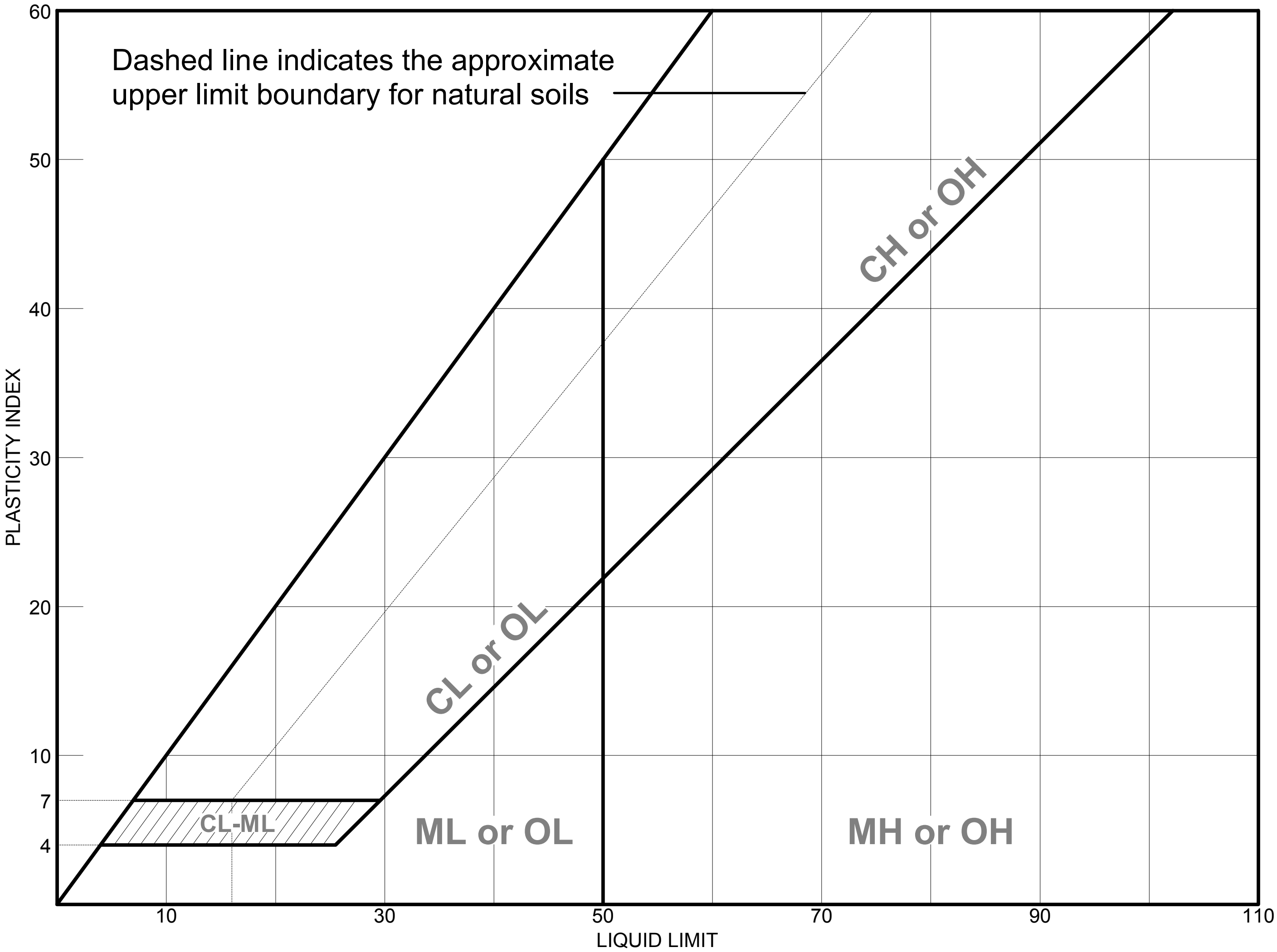
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	90.3	43.4	SM

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-5 **Elev./Depth:** 45-46.5'

Remarks:

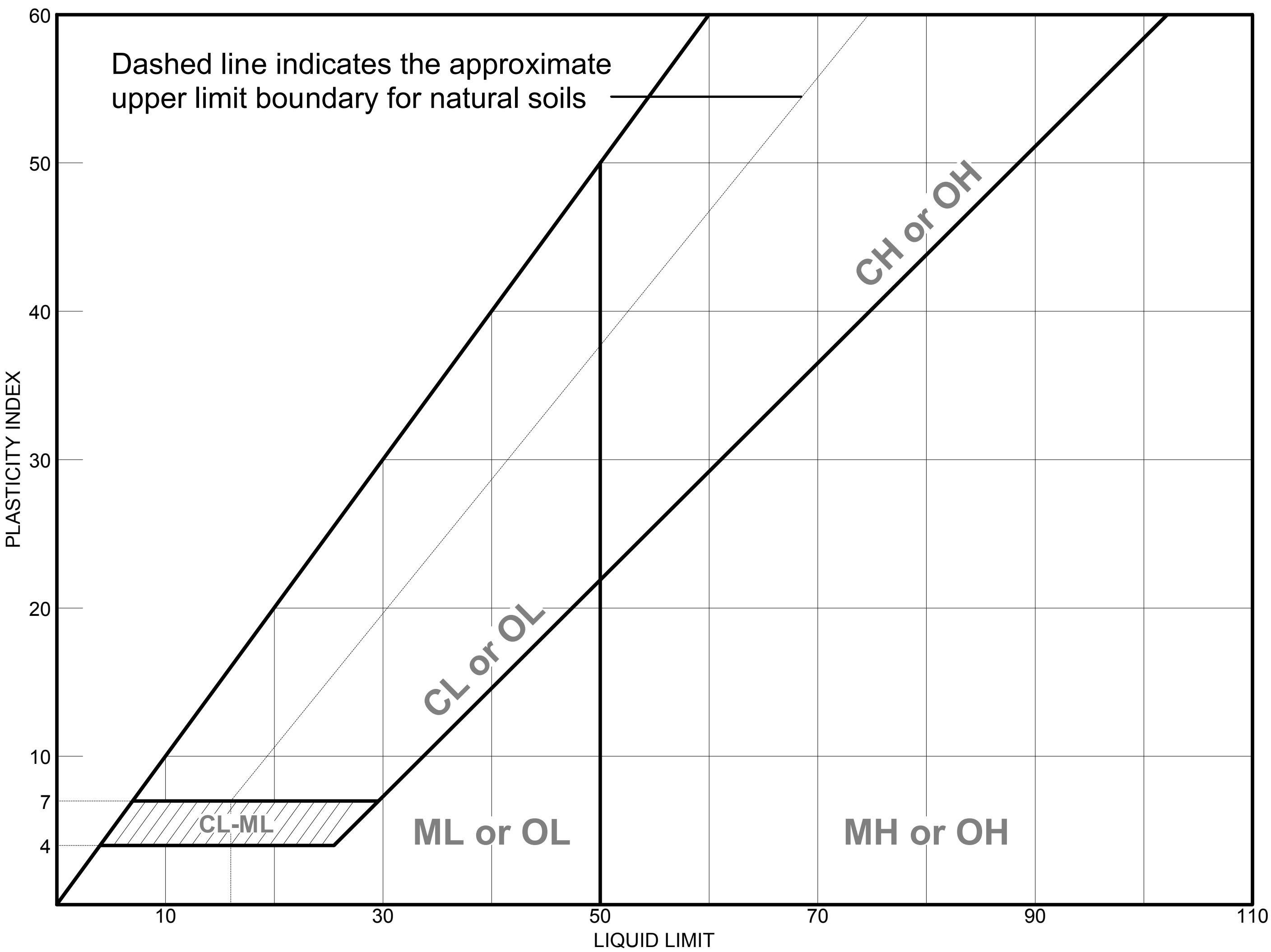
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	96.1	40.9	SM

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-7/P2 **Elev./Depth:** 1-2.5'

Remarks:

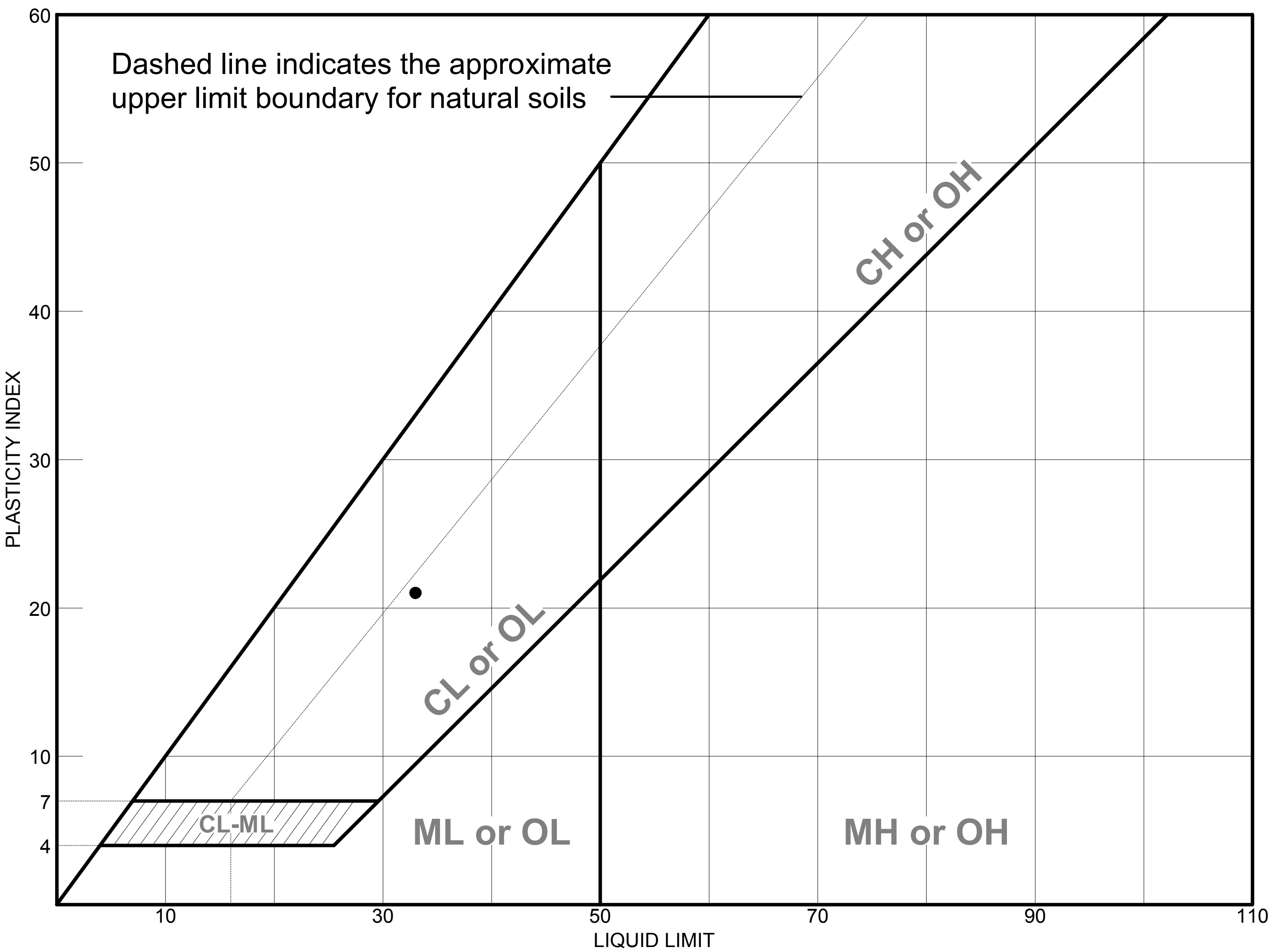
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	33	12	21	60.2	48.8	SC

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-7/P-2 **Elev./Depth:** 8.5-10'

Remarks:

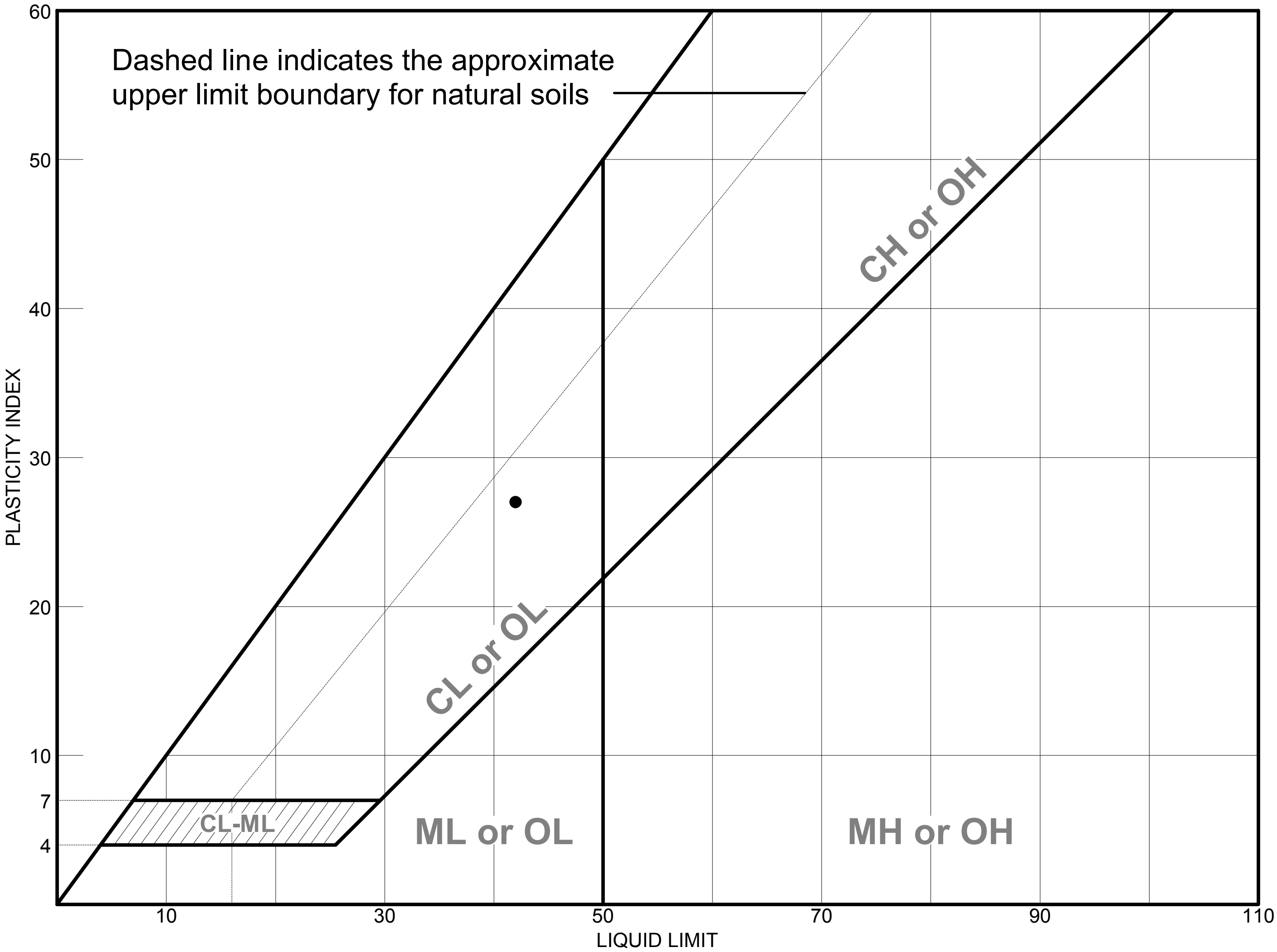
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay with sand	42	15	27	91.3	82.4	CL

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-11 **Elev./Depth:** 1-3.5'

Remarks:

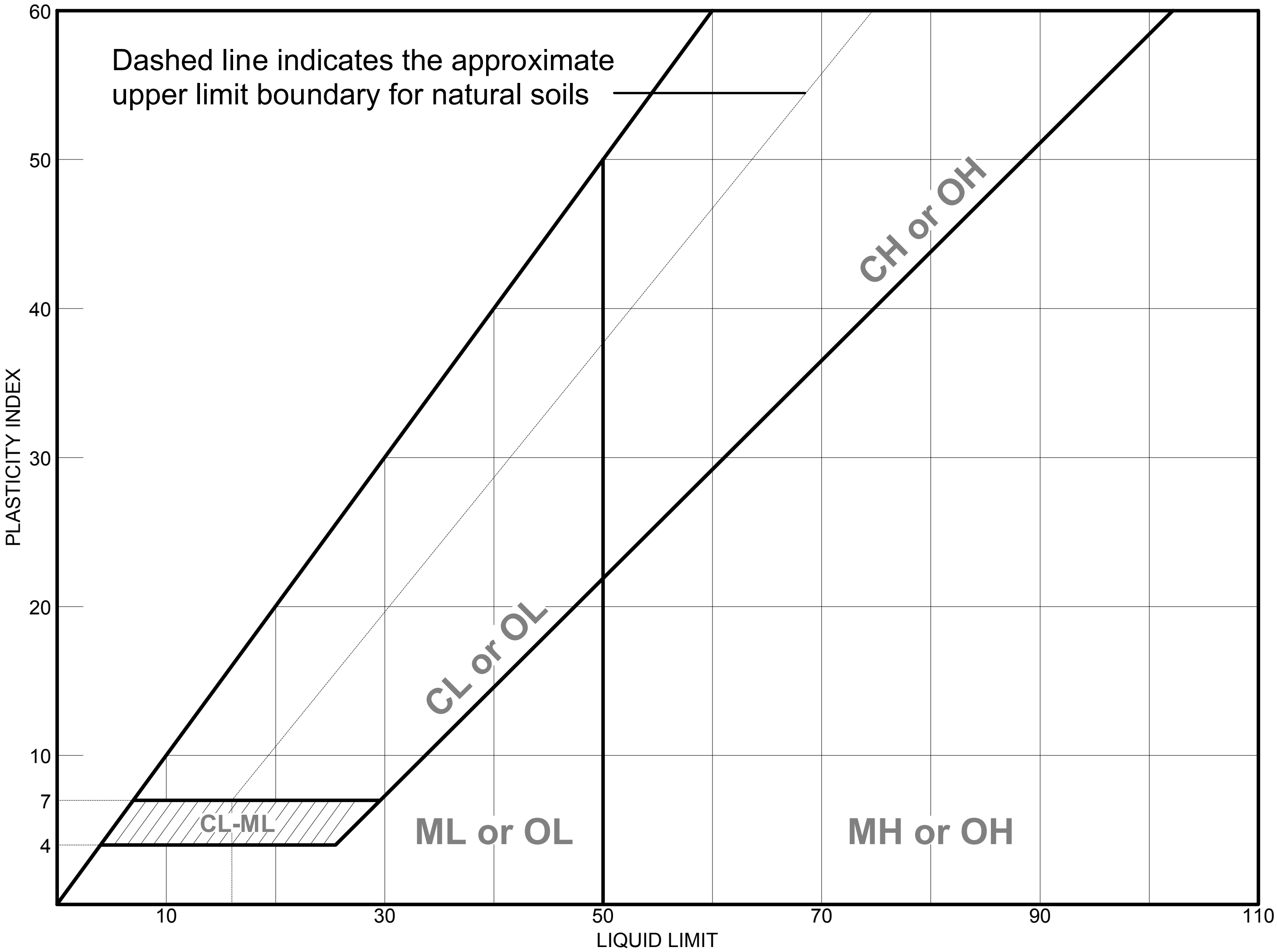
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	45.6	17.0	SM

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-11 **Elev./Depth:** 18.5-20'

Remarks:

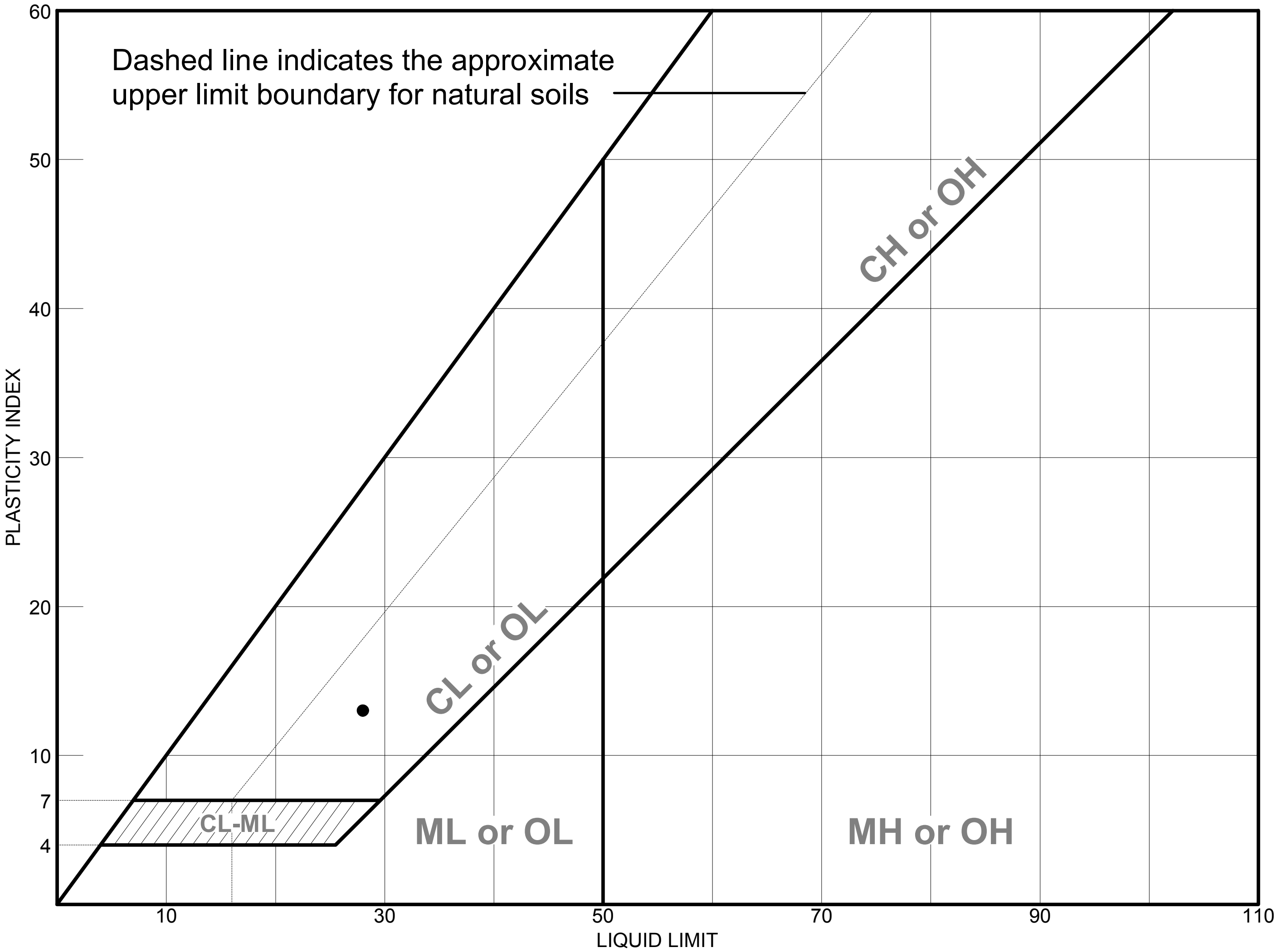
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey gravel with sand	28	15	13	24.6	13.3	GC

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** P-4 **Elev./Depth:** 3.5-5'

Remarks:

●

Moore Twining Associates, Inc.

Fresno, CA

Figure

MOORE TWINING ASSOCIATES, INC.

Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Retail Development in Davis	REPORT DATE:	11/1/2022
		TEST DATE:	10/13/2022
MTA PROJECT NO.:	H12701.01		
SAMPLE I.D.:	B-1 @ 1-3.5'		
SAMPLED BY:	JF		
SAMPLE DATE:	10/4/2022	TESTED BY:	BP

MATERIALS DESCRIPTION: Clayey sand

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	1.0176
Pan + Dry Soil Wt., gm	231.4	Dry Soil Wt., lbs	0.8568
Pan Wt., gm	0.0		
Initial % Moisture Content	8.0	Final % Moisture Content	18.8

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9257	Ring + Sample Wt., lbs	1.0176
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9257	Remolded Wt., lbs	1.0176
Remolded Wet Density, pcf	127.3	Remolded Wet Density, pcf	135.5
Remolded Dry Density, pcf	117.8	Remolded Dry Density, pcf	114.1

Expansion Data:

Initial Volume	Final Volume
0.00727222	0.007512

Initial Gage Reading, in:	0.2303
Final Gage Reading, in:	0.2633
Expansion, in:	0.0330
Expansion Index	33

Comments: **Low Expansion Potential**

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

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

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

MOORE TWINING ASSOCIATES, INC.

Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Retail Development in Davis	REPORT DATE:	11/1/2022
		TEST DATE:	10/13/2022
MTA PROJECT NO.:	H12701.01		
SAMPLE I.D.:	B-2 @ 1-3.5'		
SAMPLED BY:	JF		
SAMPLE DATE:	10/4/2022	TESTED BY:	BP
MATERIALS DESCRIPTION:	Sandy lean clay		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9831
Pan + Dry Soil Wt., gm	227.2	Dry Soil Wt., lbs	0.7980
Pan Wt., gm	0.0		
Initial % Moisture Content	10.0	Final % Moisture Content	23.2

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8781	Ring + Sample Wt., lbs	0.9831
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8781	Remolded Wt., lbs	0.9831
Remolded Wet Density, pcf	120.7	Remolded Wet Density, pcf	127.1
Remolded Dry Density, pcf	109.7	Remolded Dry Density, pcf	103.2

Initial Volume	Final Volume
0.00727222	0.007735

Expansion Data:	
Initial Gage Reading, in:	0.1056
Final Gage Reading, in:	0.1693
Expansion, in:	0.0637
Expansion Index	64

Comments: **Medium Expansion Potential**

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

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

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

MOORE TWINING
ASSOCIATES, INC.
Since 1898

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Retail Development in Davis	REPORT DATE:	11/1/2022
		TEST DATE:	10/19/2022
MTA PROJECT NO.:	H12701.01		
SAMPLE I.D.:	B-8 @ 1-3.5'		
SAMPLED BY:	JF		
SAMPLE DATE:	10/4/2022	TESTED BY:	BP
MATERIALS DESCRIPTION:	Sandy lean clay		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9515
Pan + Dry Soil Wt., gm	223.2	Dry Soil Wt., lbs	0.7598
Pan Wt., gm	0.0		
Initial % Moisture Content	12.0	Final % Moisture Content	25.2

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8510	Ring + Sample Wt., lbs	0.9515
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8510	Remolded Wt., lbs	0.9515
Remolded Wet Density, pcf	117.0	Remolded Wet Density, pcf	122.6
Remolded Dry Density, pcf	104.5	Remolded Dry Density, pcf	97.9

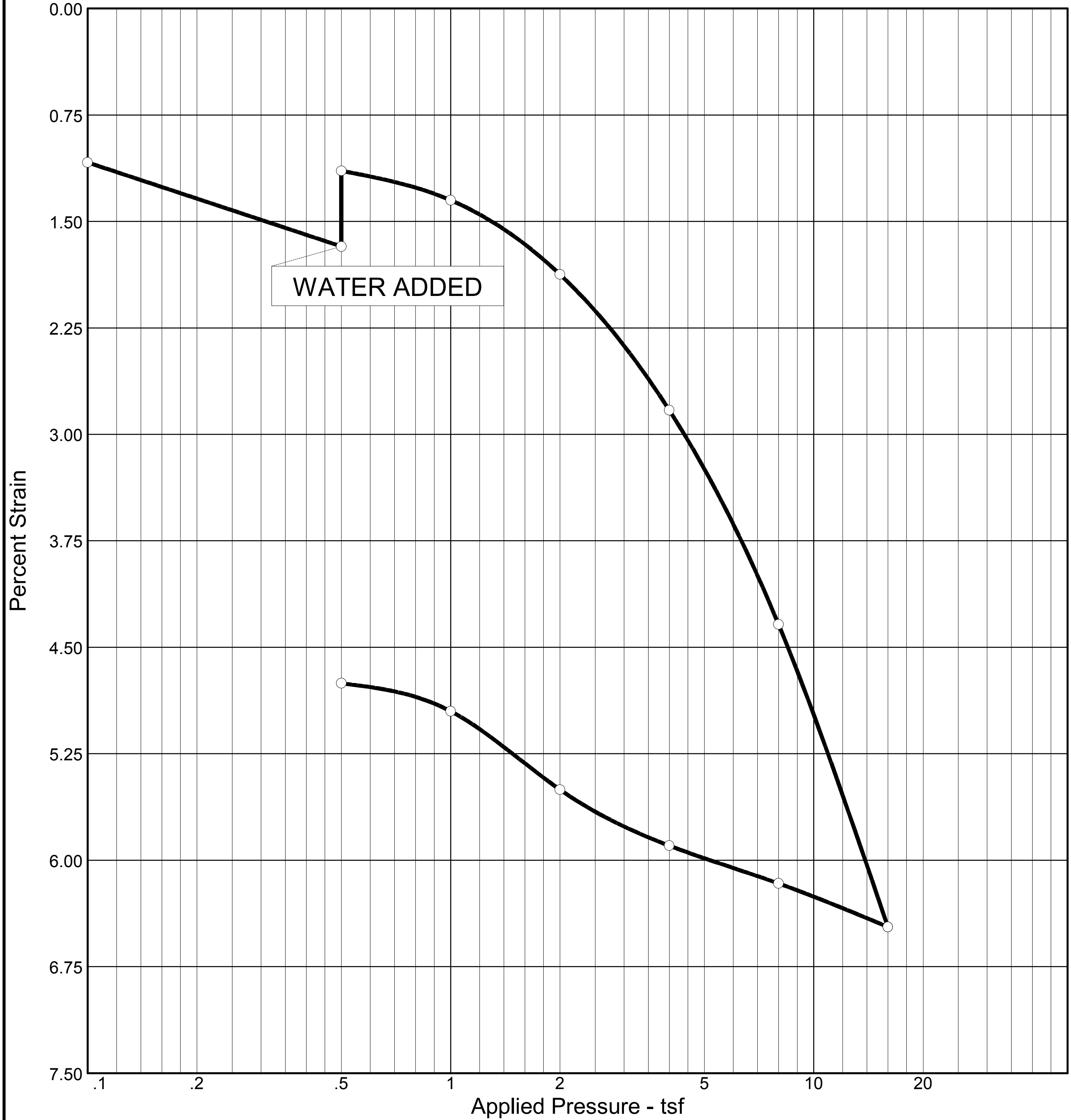
Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007758
Initial Gage Reading, in:	0.0411		
Final Gage Reading, in:	0.1079		
Expansion, in:	0.0668		
Expansion Index	67	**Comments:**	**Medium Expansion Potential**

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

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

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

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
91.0 %	14.6 %	116.0			2.65		3.81	0.10	0.02	1.63	0.6	0.426

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

Source: **Sample No.:** B-3 **Elev./Depth:** 3.5-5'

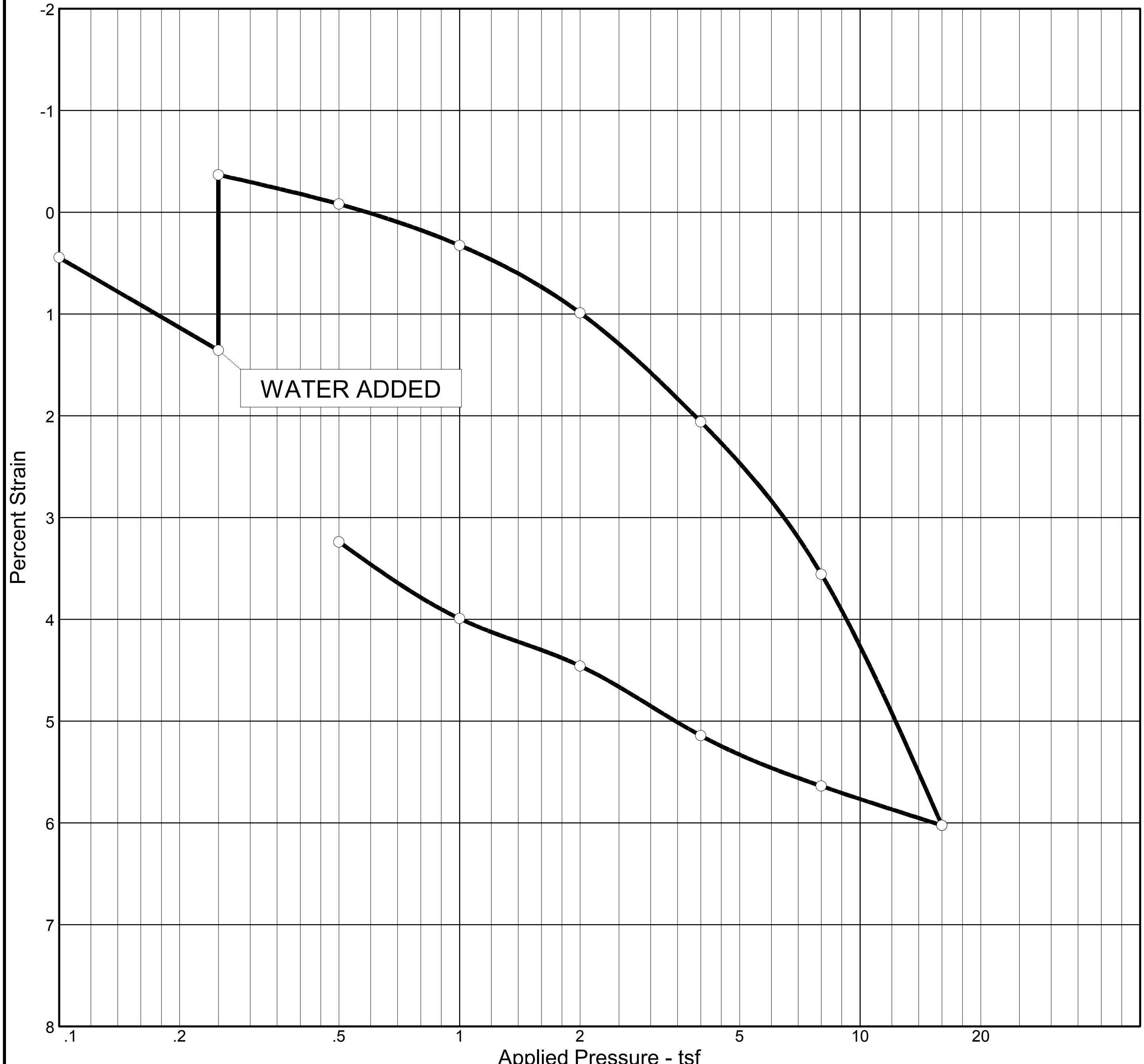
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Swell %	e_o
96.6 %	21.4 %	104.2			2.65		3.95	0.13	0.03	1.30	1.8	0.588

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay	CL	

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

Source: **Sample No.:** B-8 **Elev./Depth:** 1-2.5'

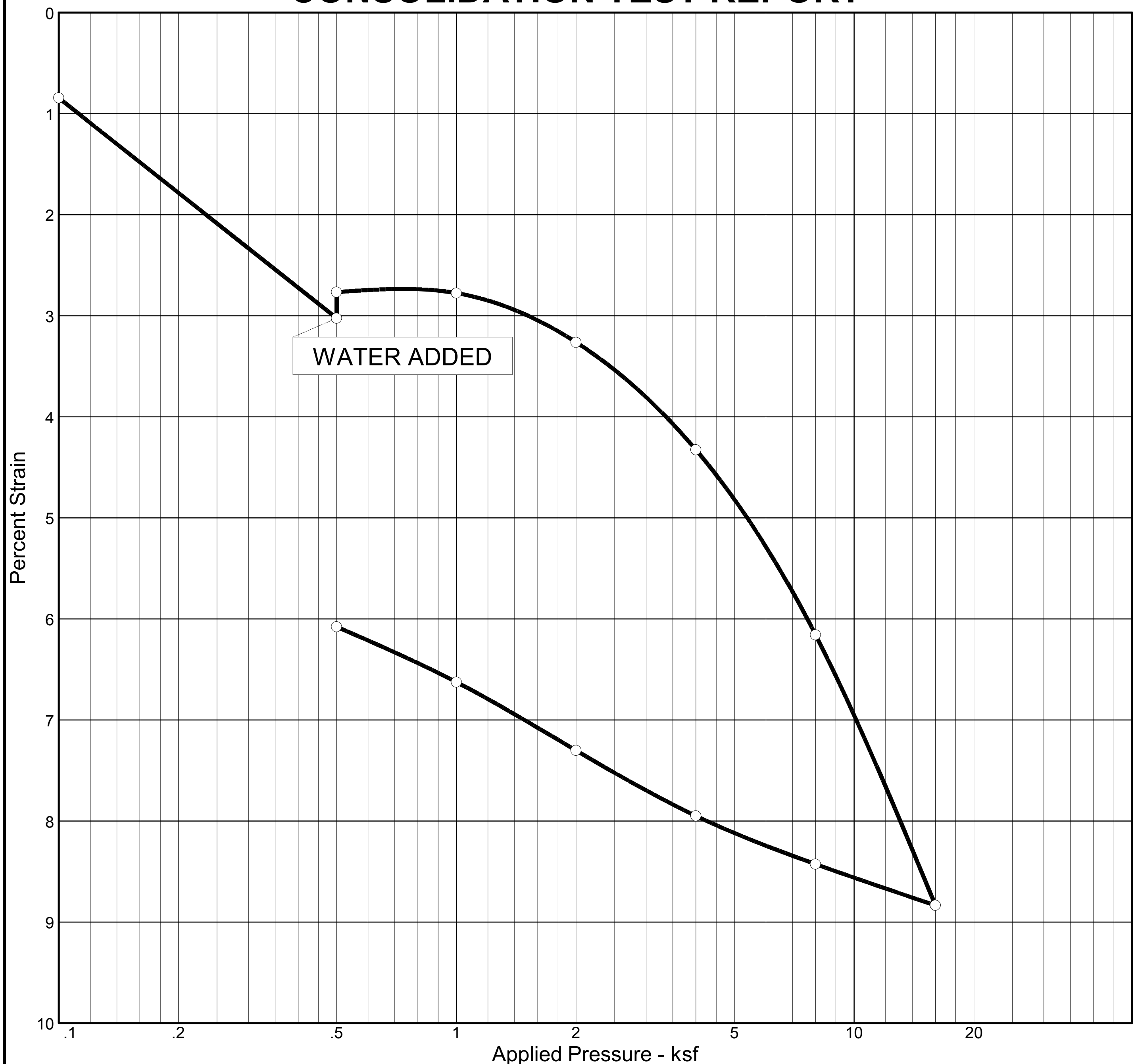
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
99.8 %	25.6 %	98.5			2.65		3.63	0.15	0.03	1.56	0.2	0.680

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy lean clay		

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

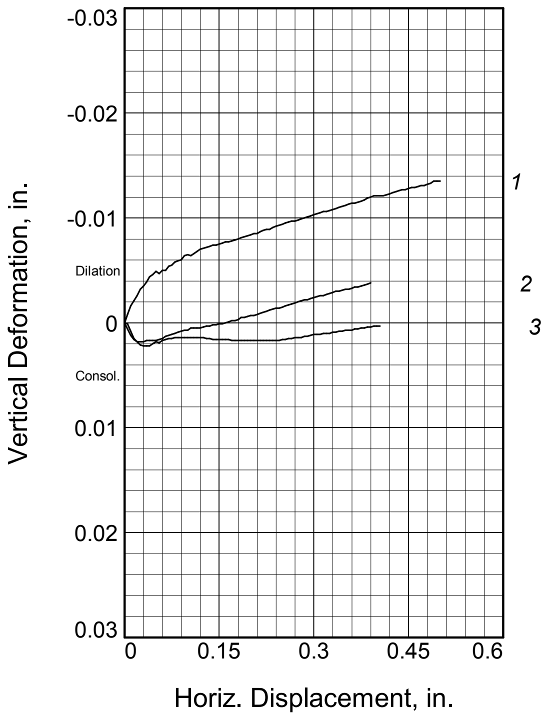
Source: **Sample No.:** B-10 **Elev./Depth:** 5-6.5'

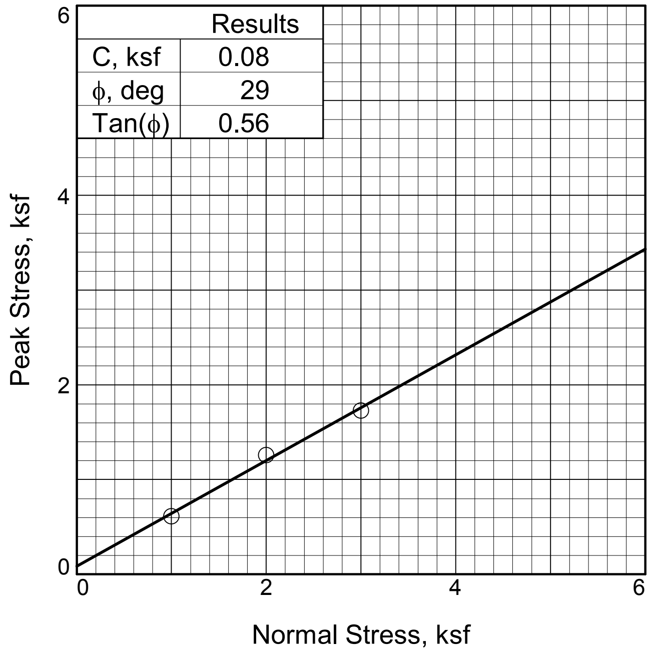
Moore Twining Associates, Inc.

Fresno, CA

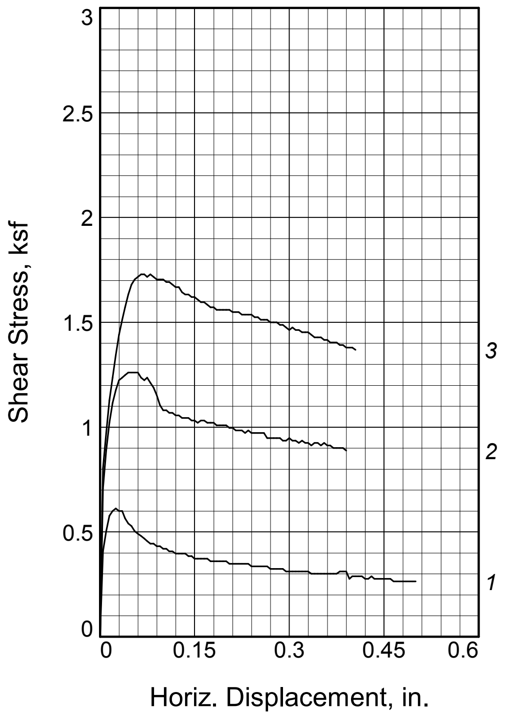
Remarks:

Figure





Results	
C, ksf	0.08
ϕ, deg	29
Tan(ϕ)	0.56



	Sample No.	1	2	3
Initial	Water Content, %	17.4	17.3	16.0
	Dry Density, pcf	98.5	98.7	95.7
	Saturation, %	67.7	67.9	58.1
	Void Ratio	0.6796	0.6768	0.7291
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.02	1.02	1.02
At Test	Water Content, %	22.2	23.9	21.2
	Dry Density, pcf	99.6	99.7	96.8
	Saturation, %	89.1	96.2	79.3
	Void Ratio	0.6616	0.6587	0.7088
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.01	1.01	1.01
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.61	1.26	1.73
	Displacement, in.	0.03	0.05	0.07
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	1.0	1.9	2.7

Sample Type:
Description: Sandy lean clay

Specific Gravity= 2.65
Remarks:

Figure _______

Client: California Property Owner I, LLC

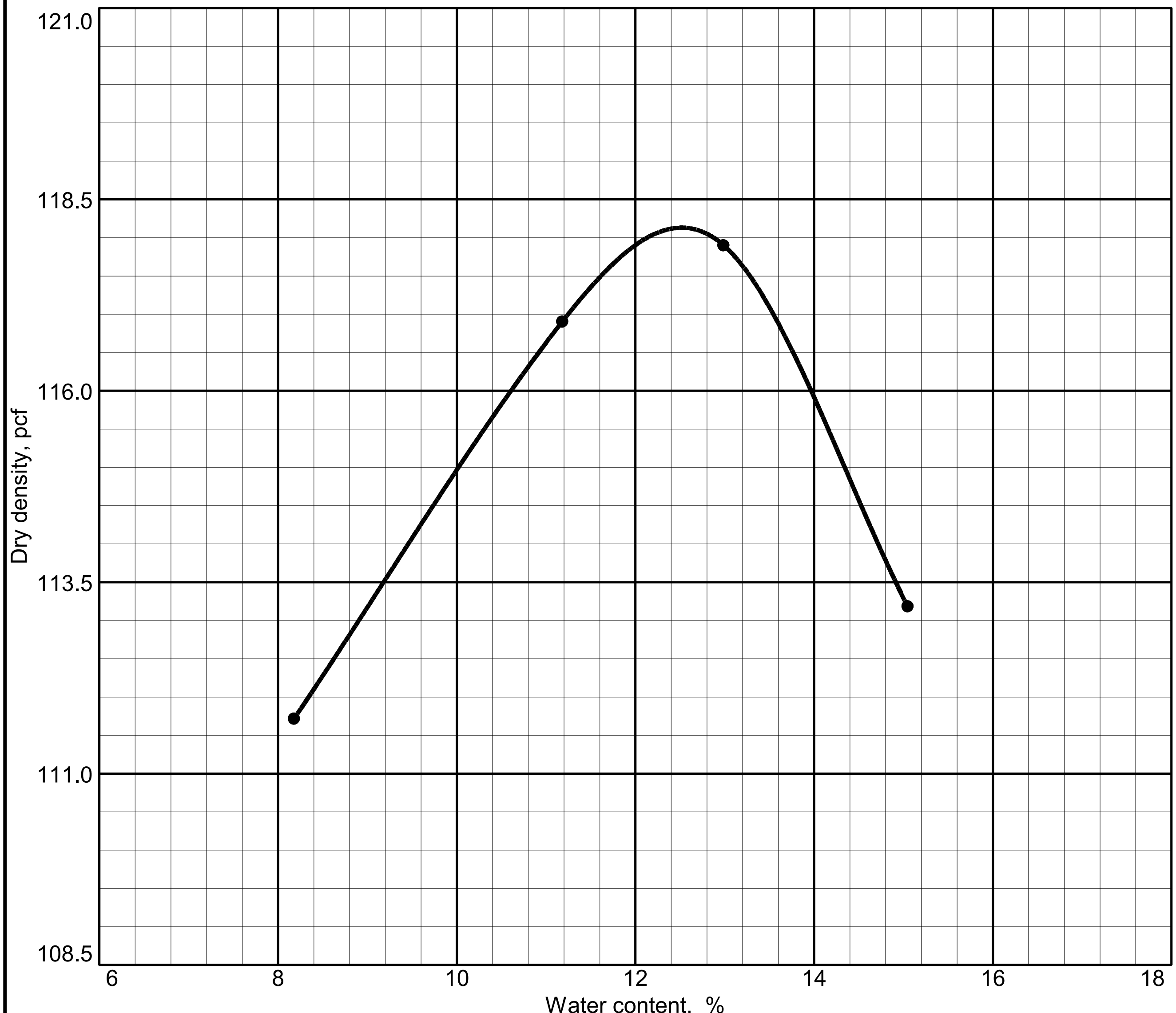
Project: Proposed Retail Development in Davis

Sample Number: B-10 **Depth:** 5-6.5'

Proj. No.: H12701.01 **Date Sampled:** 10/4/22

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

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						
1-3.5'	CL				40	21	0.0	75.3

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 118.1 pcf Optimum moisture = 12.5 %	Lean clay with sand

Project No. H12701.01 **Client:** California Property Owner I, LLC
Project: Proposed Retail Development in Davis

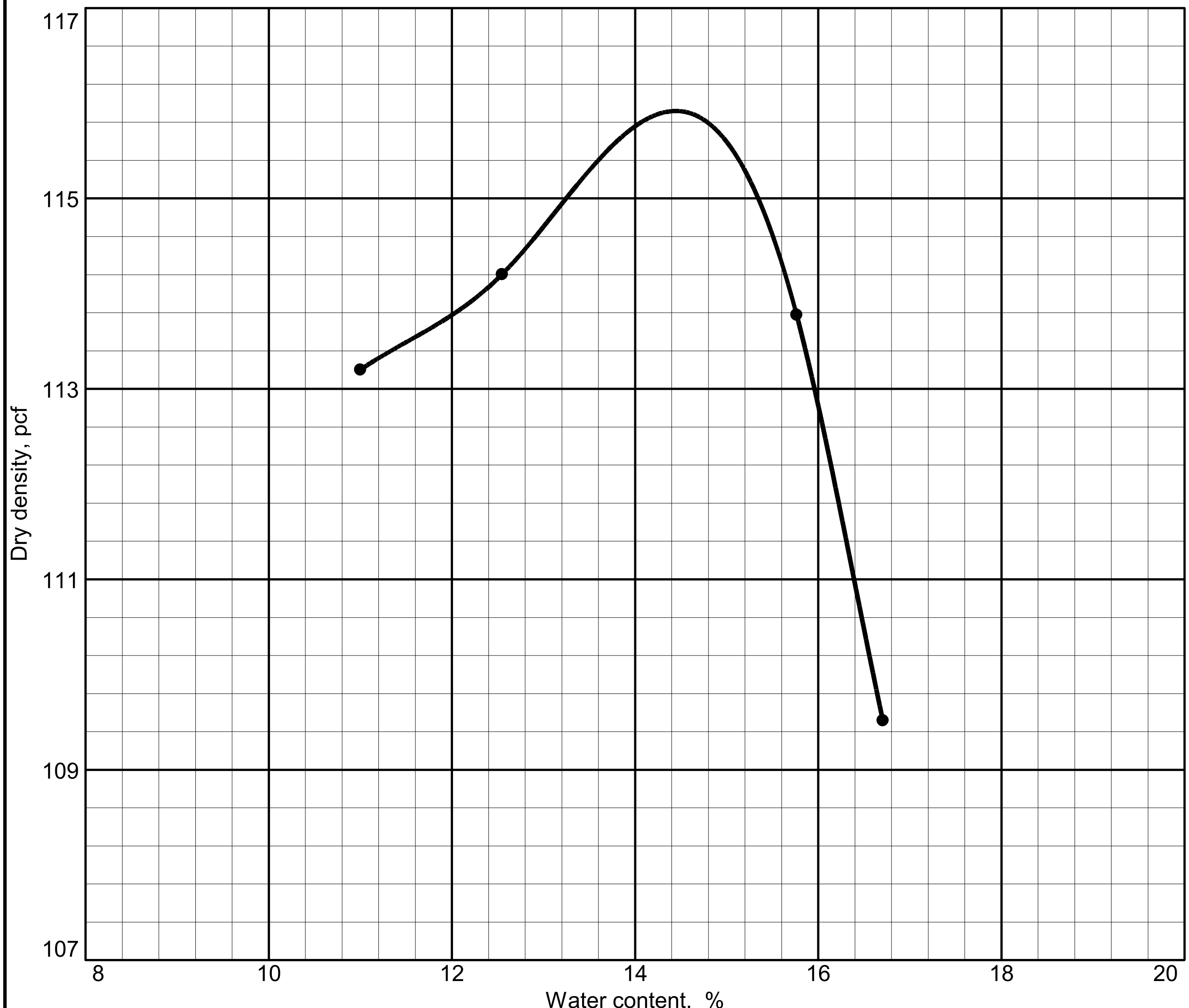
● **Source:** **Sample No.:** B-5 **Elev./Depth:** 1-3.5'

Moore Twining Associates, Inc.
Fresno, CA

Remarks:

Figure

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						
1-3.5'	CL				42	27	0.0	82.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 115.9 pcf Optimum moisture = 14.4 %	Lean Clay with Sand

Project No. H12701.01 **Client:** California Property Owner I, LLC

Project: Proposed Retail Development in Davis

● **Source:** **Sample No.:** B-11 **Elev./Depth:** 1-3.5'

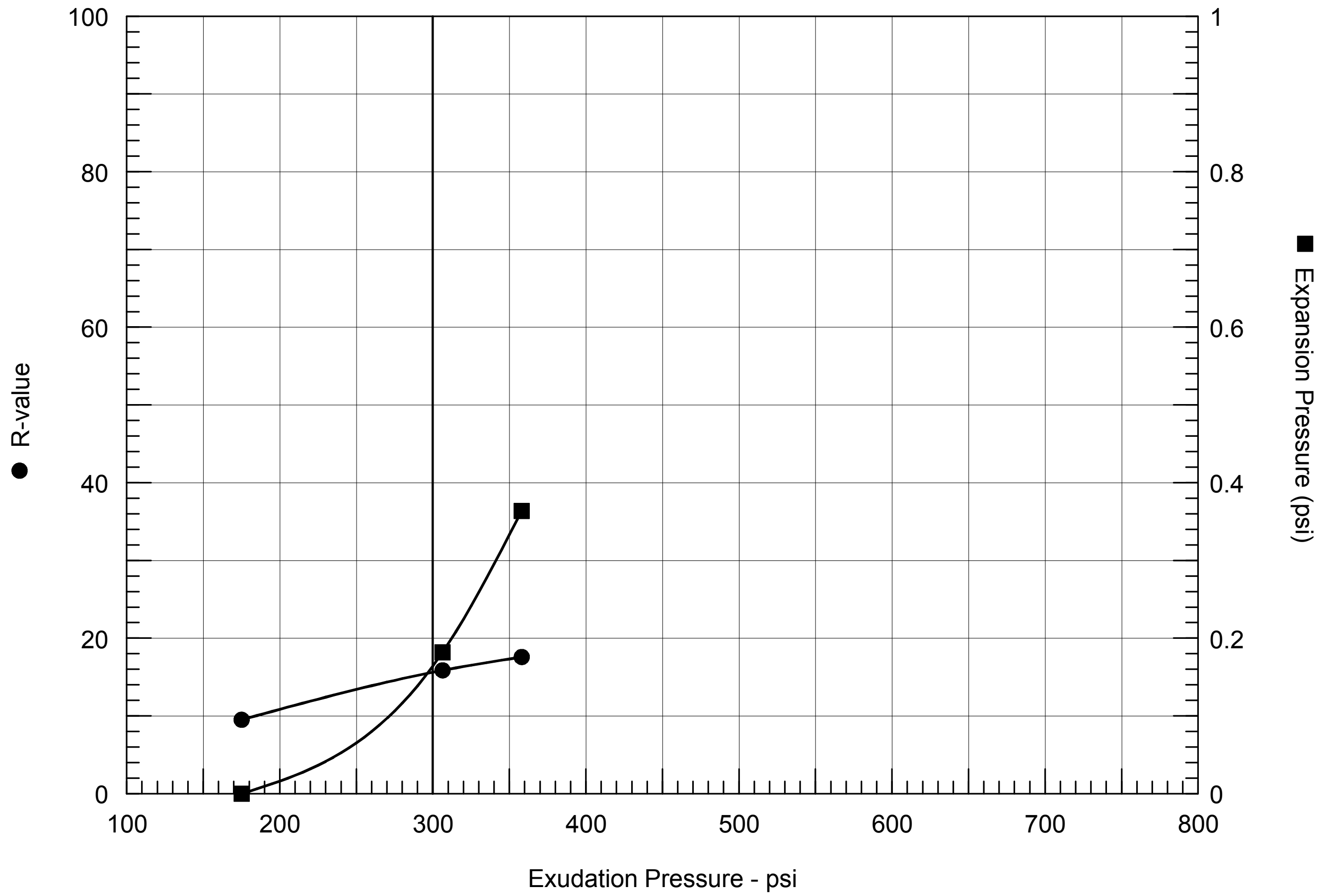
Moore Twining Associates, Inc.

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	50	107.1	19.5	0.18	124	2.45	306	16	16
2	100	105.9	18.3	0.36	118	2.42	358	19	18
3	25	101.3	20.6	0.00	136	2.48	175	10	10

Test Results	Material Description
R-value at 300 psi exudation pressure = 16 **Exp. pressure at 300 psi exudation pressure =** 0.16 psi	Sandy Lean Clay

Project No.: H12701.01

Project: Proposed Retail Development in Davis

Sample Number: B-6 **Depth:** 1-3.5'

Date: 11/10/2022

Tested by: MS

Checked by: MS

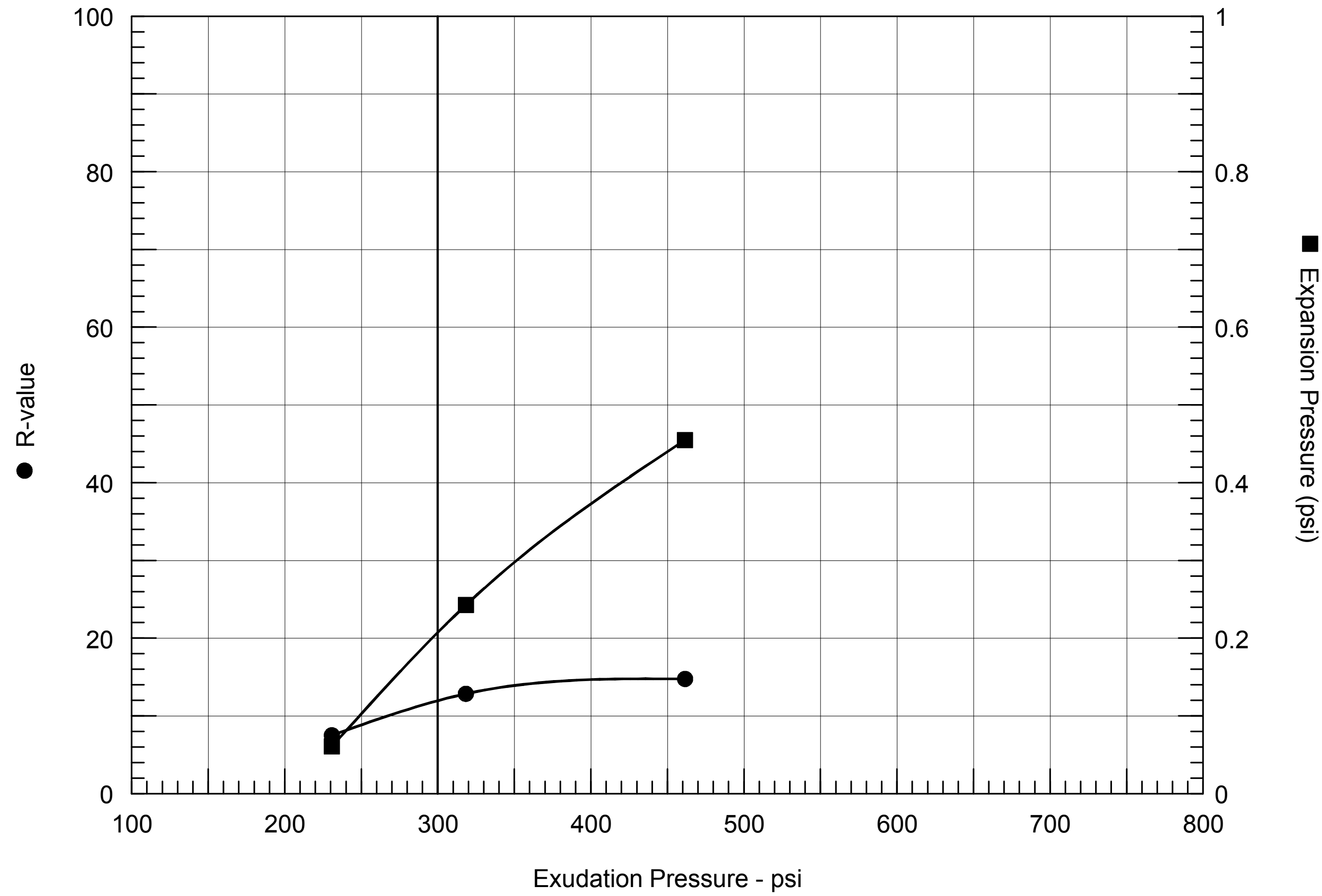
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure N/A

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	50	113.3	17.5	0.24	124	2.33	318	14	13
2	40	114.9	16.3	0.45	119	2.30	462	17	15
3	60	108.5	18.6	0.06	138	2.37	231	8	8

Test Results	Material Description
R-value at 300 psi exudation pressure = 12 **Exp. pressure at 300 psi exudation pressure =** 0.21 psi	Sandy lean clay

Project No.: H12701.01

Project: Proposed Retail Development in Davis

Sample Number: B-10 **Depth:** 1-3.5'

Date: 11/10/2022

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Tested by: MS

Checked by: MS

Remarks:

Figure N/A

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

October 20, 2022

Work Order #: **IJ11005**

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

RE: Proposed Retail Developement in Davis

Enclosed are the analytical results for samples received by our laboratory on **10/11/22** . For your reference, these analyses have been assigned laboratory work order number **IJ11005**.

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

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

Sincerely,

Moore Twining Associates, Inc.

Susan Federico

Susan Federico
Client Services Representative

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division	**Project:**	Proposed Retail Developement in Davis	**Reported:**
2527 Fresno Street	**Project Number:**	H12701.01	10/20/2022
Fresno CA, 93721	**Project Manager:**	Allen Harker	

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B1 @ 1-3.5'		IJ11005-01	Soil	10/04/22 00:00	10/11/22 09:37
B5 @ 1-3.5'		IJ11005-02	Soil	10/04/22 00:00	10/11/22 09:37
B11 @ 1-3.5'		IJ11005-03	Soil	10/04/22 00:00	10/11/22 09:37

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Proposed Retail Developement in Davis **Project Number:** H12701.01 **Project Manager:** Allen Harker	**Reported:** 10/20/2022

B1 @ 1-3.5'

IJ11005-01 (Soil) Sampled: 10/04/22 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**12**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 422
Chloride		**0.0012**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
Sulfate as SO4		**0.0035**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
pH		**8.7**	0.10	pH Units	1	B2J1309	10/13/22	10/14/22	Cal Test 643
Sulfate as SO4		**35**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 417

B5 @ 1-3.5'

IJ11005-02 (Soil) Sampled: 10/04/22 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**6.7**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 422
Chloride		**0.00067**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
Sulfate as SO4		**0.0027**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
pH		**8.8**	0.10	pH Units	1	B2J1309	10/13/22	10/14/22	Cal Test 643
Sulfate as SO4		**27**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 417

B11 @ 1-3.5'

IJ11005-03 (Soil) Sampled: 10/04/22 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**60**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 422
Chloride		**0.006**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
Sulfate as SO4		**0.016**	0.00060	% by Weight	3	[CALC]	10/14/22	10/14/22	[CALC]
pH		**8.5**	0.10	pH Units	1	B2J1309	10/13/22	10/14/22	Cal Test 643
Sulfate as SO4		**160**	6.0	mg/kg	3	B2J1309	10/13/22	10/14/22	Cal Test 417

Notes and Definitions

DUP1	A high RPD was observed between a sample and this sample's duplicate.
µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference
	Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field. If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

Project Name:	Proposed Retail Development in Davis	Report Date:	11/1/2022
		Sample Date:	10/4/2022
Project Number:	H12701.01		
		Sampled By:	JF
Subject:	Minimum Resistivity, ASTM G187	Tested By:	BP
Material Description:	Clayey sand	Test Date:	10/22/2022
Location:	B-1 @ 1-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
50 mls	8,300
100 mls	3,600
150 mls	3,400
200 mls	3,600

Remarks: Min. Resistivity is 3,400 Ohm-cm

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

MOORE TWINING ASSOCIATES, INC.

Project Name:	Proposed Retail Development in Davis	Report Date:	11/1/2022
		Sample Date:	10/4/2022
Project Number:	H12701.01		
		Sampled By:	JF
Subject:	Minimum Resistivity, ASTM G187	Tested By:	BP
Material Description:	Lean clay with sand	Test Date:	10/22/2022
Location:	B-5 @ 1-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	6,800
150 mls	3,100
200 mls	2,700
250 mls	2,700
300 mls	2,900

Remarks: Min. Resistivity is 2,700 Ohm-cm

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

MOORE TWINING ASSOCIATES, INC.
Since 1898

Project Name:	Proposed Retail Development in Davis	Report Date:	11/1/2022
		Sample Date:	10/4/2022
Project Number:	H12701.01		
		Sampled By:	JF
Subject:	Minimum Resistivity, ASTM G187	Tested By:	BP
Material Description:	Lean clay with sand	Test Date:	10/22/2022
Location:	B-11 @ 1-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	6,800
150 mls	3,400
200 mls	2,000
250 mls	2,000
300 mls	2,100

Remarks: Min. Resistivity is 2,000 Ohm-cm

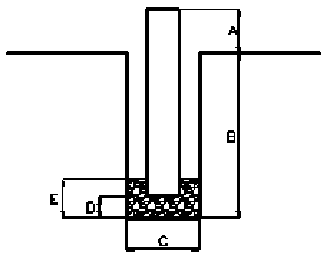
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

APPENDIX D

RESULTS OF PERCOLATION TESTING

PERCOLATION TEST
No. P-1

Project:	Proposed Retail Development in Davis	**Project No.**	H12701.01
Location:	NEC of Sycamore Lane and Russell Boulevard, Davis, CA	**Test Date:**	10/6/2022
Coordinates:	38.547694, -121.761409		



A. Top of Pipe Above Ground	0 Inches
B. Depth of Hole	63 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	21 Inches
F. Pipe Length	61 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: to 11.4 inches from bottom on 10/5/22 at 2:10 p.m.
Checked 4.4 inches from bottom on 10/6/22 at 1:45 p.m.

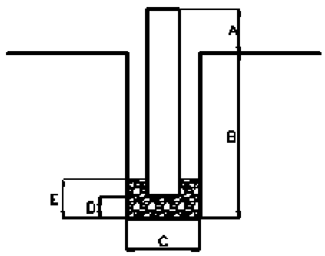
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	10/6/2022	13:50:00	4.3				
	10/6/2022	14:20:00	4.3	30	0.0	N/A	0.0
2	10/6/2022	14:20:00	4.3				
	10/6/2022	14:50:00	4.3	30	0.0	N/A	0.0
3	10/6/2022	14:50:00	4.3				
	10/6/2022	15:20:00	4.31	30	0.1	639.9	0.0
4	10/6/2022	15:20:00	4.31				
	10/6/2022	15:50:00	4.32	30	0.1	639.9	0.0

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project: Proposed Retail Development in Davis **Project No.** H12701.01
Location: NEC of Sycamore Lane and Russell Boulevard, Davis, CA **Test Date:** 10/5/2022
Coordinates: 38.546942, -121.760994



A. Top of Pipe Above Ground 8 Inches
B. Depth of Hole 114 Inches
C. Diameter of Hole 8 Inches
D. Depth of Gravel Below Pipe 2 Inches
E. Total Gravel Layer Depth 21 Inches
F. Pipe Length 120 Inches
G. Pipe Diameter 2 Inches

Pre-saturated: to 12.8 inches from bottom on 10/4/22 at 9:20 a.m.
Checked 3.8 inches from bottom on 10/5/22 at 10:00 a.m.

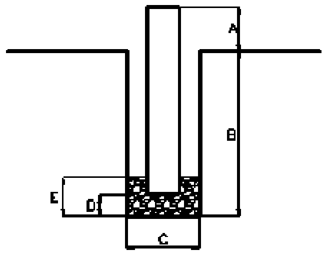
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	10/5/2022	14:50:00	9.18				
	10/5/2022	15:10:00	9.2	20	0.2	213.3	0.0
2	10/5/2022	15:10:00	9.2				
	10/5/2022	15:30:00	9.27	20	0.8	60.9	0.1
3	10/5/2022	15:30:00	9.2				
	10/5/2022	15:50:00	9.3	20	1.2	42.7	0.2
4	10/5/2022	15:50:00	9.2				
	10/5/2022	16:10:00	9.29	20	1.1	47.4	0.2
5	10/5/2022	16:10:00	9.2				
	10/5/2022	16:30:00	9.28	20	1.0	53.3	0.2
6	10/5/2022	16:30:00	9.1				
	10/5/2022	16:50:00	9.17	20	0.8	60.9	0.1
7	10/5/2022	16:50:00	9.17				
	10/5/2022	17:10:00	9.25	20	1.0	53.3	0.2
8	10/5/2022	17:10:00	9.2				
	10/5/2022	17:30:00	9.28	20	1.0	53.3	0.2
9	10/5/2022	17:30:00	9.2				
	10/5/2022	17:50:00	9.28	20	1.0	53.3	0.2

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-3

Project: Proposed Retail Development in Davis **Project No.** H12701.01
Location: NEC of Sycamore Lane and Russell Boulevard, Davis, CA **Test Date:** 10/6/2022
Coordinates: 38.547032, -121.759470



A. Top of Pipe Above Ground	6	Inches
B. Depth of Hole	118	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	2	Inches
E. Total Gravel Layer Depth	22	Inches
F. Pipe Length	122	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: to 14.2 inches from bottom on 10/4/22 at 10:14 a.m.
Checked 7 inches from bottom on 10/5/22 at 11:03 a.m.

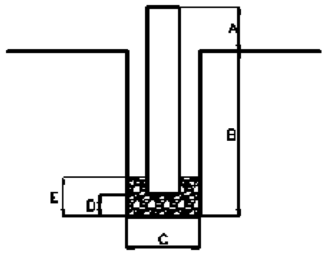
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	10/6/2022	10:25:00	9.15				
	10/6/2022	10:45:00	9.15	20	0.0	N/A	0.0
2	10/6/2022	10:45:00	9.15				
	10/6/2022	11:05:00	9.16	20	0.1	426.6	0.0
3	10/6/2022	11:05:00	9.16				
	10/6/2022	11:25:00	9.16	20	0.0	N/A	0.0
4	10/6/2022	11:25:00	9.16				
	10/6/2022	11:45:00	9.17	20	0.1	426.6	0.0
5	10/6/2022	11:45:00	9.17				
	10/6/2022	12:05:00	9.17	20	0.0	N/A	0.0
6	10/6/2022	12:05:00	9.17				
	10/6/2022	12:25:00	9.18	20	0.1	426.6	0.0
7	10/6/2022	12:25:00	9.18				
	10/6/2022	12:45:00	9.2	20	0.2	213.3	0.0
8	10/6/2022	12:45:00	9.15				
	10/6/2022	13:05:00	9.16	20	0.1	426.6	0.0
9	10/6/2022	13:05:00	9.16				
	10/6/2022	13:25:00	9.16	20	0.0	N/A	0.0

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-4

Project: Proposed Retail Development in Davis **Project No.** H12701.01
Location: NEC of Sycamore Lane and Russell Boulevard, Davis, CA **Test Date:** 10/6/2022
Coordinates: 38.547155, -121.758940



A. Top of Pipe Above Ground 20 Inches
B. Depth of Hole 59 Inches
C. Diameter of Hole 8 Inches
D. Depth of Gravel Below Pipe 2 Inches
E. Total Gravel Layer Depth 21 Inches
F. Pipe Length 77 Inches
G. Pipe Diameter 2 Inches

Pre-saturated: to 15.0 inches from bottom on 10/4/22 at 2:10 p.m.
Checked 8.1 inches from bottom on 10/5/22 at 11:07 a.m.

Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Uncorrected, Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	10/6/2022	10:30:00	5.33				
	10/6/2022	10:50:00	5.91	20	7.0	7.4	1.2
2	10/6/2022	10:50:00	5.35				
	10/6/2022	11:10:00	5.45	20	1.2	42.7	0.2
3	10/6/2022	11:10:00	5.35				
	10/6/2022	11:30:00	5.4	20	0.6	85.3	0.1
4	10/6/2022	11:30:00	5.35				
	10/6/2022	11:50:00	5.4	20	0.6	85.3	0.1
5	10/6/2022	11:50:00	5.34				
	10/6/2022	12:10:00	5.39	20	0.6	85.3	0.1
6	10/6/2022	12:10:00	5.35				
	10/6/2022	12:30:00	5.4	20	0.6	85.3	0.1
7	10/6/2022	12:30:00	5.32				
	10/6/2022	12:50:00	5.38	20	0.7	71.1	0.1
8	10/6/2022	12:50:00	5.34				
	10/6/2022	13:10:00	5.4	20	0.7	71.1	0.1
9	10/6/2022	13:10:00	5.35				
	10/6/2022	13:30:00	5.4	20	0.6	85.3	0.1

* Depth to water measured from top of pipe