DRAFT

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED HOME DEPOT STORE

261 NORTH McCDOWELL BOULEVARD

PETALUMA, CALIFORNIA

Project Number: D050S1.01

For:

Home Depot U.S.A., Inc.
2455 Paces Ferry Road C19
Atlanta, GA 30339

March 29, 2021

MOORE TWINING
ASSOCIATES, INC.
Since 1898

March 29, 2021 D050S1.01

Home Depot U.S.A., Inc.
2455 Paces Ferry Road C19
Atlanta, GA 30339

Attention: Mr. John Foy

Subject: **Draft: Preliminary Geotechnical Engineering Investigation Report**
Proposed Home Depot Store
261 North McDowell Boulevard
Petaluma, California

Dear Mr. Foy:

We are pleased to submit this preliminary geotechnical engineering investigation report prepared for the proposed Home Depot store 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 Moore Twining Associates, Inc. (Moore Twining) be retained to conduct the design level geotechnical investigation and to review those portions of the final plans and specifications that pertain to earthwork, pavements, and foundations to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement, however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

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

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

Sincerely,

MOORE TWINING ASSOCIATES, INC.

DRAFT

Read L. Andersen, RGE
Manager
Geotechnical Engineering Division

EXECUTIVE SUMMARY

This report presents the results of a preliminary geotechnical engineering investigation for the proposed Home Depot store to be located at 261 N. McDowell Boulevard in Petaluma, California. The site is considered geotechnically suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report are followed.

The subject site is occupied by a former K-mart building, retail shops, and includes a depressed loading dock ramp, a canopy structure, exterior concrete flatwork and asphalt concrete pavements. Existing underground utilities are located throughout the site.

The near surface soils encountered generally consisted of fat clay soils extending to depths ranging from approximately 5 feet to 15 feet BSG. The fat clays were generally underlain by interbedded silty sands, clayey sands, fat clays and lean clays to the maximum depth explored of 50 feet BSG. Fat clay fill soils were encountered in boring B-5 below the pavement section to a depth of 2½ feet below site grade.

Groundwater was encountered in soil borings at depths ranging from about 7.7 feet to 19½ feet BSG. Shallower groundwater conditions are anticipated since the borings were not left open for an extended period of time to determine stabilized groundwater depths, as restricted by the well (soil boring) permit. It is our understanding the project environmental consultant may be installing monitoring wells for future groundwater depth monitoring.

Our analysis indicate that static settlement of foundations placed directly on the native soils at the proposed foundation depth would exceed the Home Depot design criteria for differential settlement of ½ inch in 50 feet. In order to meet the total combined (static plus seismic) settlement criteria of 1 inch total and ½ inch differential in 50 feet, the structure loads would need to be supported on a minimum of 6 feet of engineered fill below foundations established by over-excavation and compaction of the onsite soils. However, due to the shallow groundwater conditions, it is recommended specialty deep ground modification approaches also be considered for support of the foundations as an alternative. This alternative would have the benefit of reduced dewatering, wet soils and soil stabilization issues.

In the event Home Depot would allow a higher settlement criteria for this project: 1 inch of total static settlement, ½ inch differential static settlement, ½ inch total seismic settlement, and ¼ inch differential seismic settlement, the structure loads could be supported on a minimum of 3 feet of engineered fill below foundations. Refer to the discussion in Section 6.7 of this report. Note that regardless of the approach for foundation support and settlement criteria, non-expansive fill materials will need to be placed for support of all concrete slabs on grade and to reduce the potential for expansive soils movement.

Based on the results of testing conducted for this investigation, the near surface clay soils encountered exhibit a high plasticity and medium to high expansion (nearly very high) potential. In order to reduce the potential for excessive heave of slabs on grade, this report recommends non-expansive fill below concrete slabs on grade. As an alternative to importing granular, non-expansive fill materials, it may be possible to lime treat the onsite soils for use as a non-expansive fill. However, laboratory suitability testing would be required to evaluate this approach.

EXECUTIVE SUMMARY, CONTINUED

As part of the site preparation, all surface and subsurface structures and improvements removed, such as foundation elements, subsurface structures, utilities, etc. should be completely removed and disposed of off-site in a legal manner, and the excavations properly backfilled. The site preparation should include removal of soils which are disturbed from demolition and all undocumented fill soils, such as the fill reportedly placed as part of closure of the auto service center.

Based on our analysis of liquefaction, preliminary estimates of total seismic settlement are about ½ inch total and ¼ inch differential seismic settlement in 50 feet.

Chemical testing of soil samples indicated the soils exhibit a "extremely corrosive" potential for metallic corrosion and a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

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

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED HOME DEPOT STORE

261 NORTH McCDOWELL BOULEVARD

PETALUMA, CALIFORNIA

Project Number: D050S1.01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering investigation for the proposed Home Depot store to be located at 261 N. McDowell Boulevard in Petaluma, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Home Depot U.S.A., Inc. 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, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, 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 and CPT soundings (Appendix B), and the results of laboratory tests (Appendix C).

2.0 PURPOSE AND SCOPE OF INVESTIGATION

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

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations, exterior slabs-on-grade, and pavements with regard to the Home Depot design criteria;

2.1.2 Conclusions regarding the potential for liquefaction, magnitude of seismic settlement, and recommendations for CBC seismic near source factors and coefficients;

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

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

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

2.1.6 Recommendations for temporary excavations and trench backfill; and

2.1.7 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed project referenced in the Anticipated Construction section of this report. The intent of the investigation was to provide a preliminary geotechnical investigation report. It is not the intent of the investigation to comply with the Home Depot Design Criteria Manual requirements for geotechnical investigations. Supplemental geotechnical investigations will be required to develop final design level recommendations and to comply with the Home Depot requirements.

2.2 **<u>Scope</u>:** Our proposal, dated October 9, 2020, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 The geotechnical investigation requirements of the Home Depot Design Criteria Manual (dated October 17, 2016) was reviewed.

2.2.2 A Overlay Drawing for the proposed Home Depot Store, prepared by Lars Andersen & Associates, dated May 21, 2020, was reviewed. The plan is referred to as the site plan.

2.2.3 A Closure Letter for the K-mart Auto Center including the Site Closure Summary, dated January 3, 2014 prepared by San Francisco Bay Regional Water Quality Control Board, was reviewed.

2.2.4 Soil Boring and CPT permits were obtained from Sonoma County Environmental Health Department prior to start of the investigation.

2.2.5 A visual site reconnaissance and subsurface exploration were conducted.

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

2.2.7 Mr. Dan Zoldak (Lars Andersen and Associates, Inc.) and Mr. Cordon T. Baesel (Troutman Pepper) were consulted during the investigation.

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

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

3.0 BACKGROUND INFORMATION

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

3.1 Site Description: The subject site is located at 261 N. McDowell Boulevard in the City of Petaluma (see Drawing No. 1 and 2 in Appendix A). The site is bordered to the north by existing site improvements and retail shops associated with the existing shopping center, to the south by Highway 101, to the west by a drive area and multi-residential building beyond and to the east by a retail development. The Home Depot site area comprises about 8.63 acres.

The subject site consisting of an existing/former K-mart and retail shop buildings. The K-mart building is about 89, 808 square feet in plan area. The existing K-mart building consists of exterior CMU block wall construction with interior steel columns and a slab on grade floor. A depressed loading dock with a canopy area is located on the southwest side of the building. A metal fence enclosure is located on the east side of the building. The building interior includes floor coverings in the former sales areas and an exposed concrete floor in the former shop and stock room areas. A mezzanine is located in the south side of the building. A row of retail shop buildings is located to the northeast of the Kmart and are separated by a canopy covered concrete walkway.

The area around the existing K-mart building is generally covered concrete walkways and asphalt concrete. The asphaltic concretre pavements noted to have varying degree of alligator and block type cracking.

Existing underground utilities are located throughout the site area.

3.2 Site History: Based on our review of satellite images from 1993 to 2020, the site has been developed with the existing improvements since at least 1993.

Based on our review of the as-built plans for the existing K-mart store, the K-mart building was constructed sometime in 1979.

3.3 Previous Studies: Based on our review of the Closure Letter for the K-mart Auto Center, including the Site Closure Summary, dated January 3, 2014 prepared by San Francisco Bay Regional Water Quality Control Board, twelve (12) underground hydraulic lifts and associated piping were removed from the K-mart auto area in 2011. The removal also included excavation of 111 tons of soil. The investigation also included installation of five (5) groundwater monitoring wells, water sampling and monitoring. The Site Closure Summary indicated a groundwater depth of 7.87 feet below site grade. The Closure Letter for the K-mart Auto Center indicates "..no further action related to the hydraulic fluid release(s) at the site is required."

No other geotechnical or environmental assessment reports had been provided to Moore Twining at the time of this investigation. If available, these reports should be provided to Moore Twining for review.

3.4 Anticipated Construction: Based on the site plan, it is our understanding that the existing Kmart Store and retail buildings located at the northwest corner of the site will be demolished and a new 107,891 square foot Home Depot Store will be constructed. The Home Depot Store will include a 28, 216 square foot Garden Center, a 2,465 square foot Tool Rental Center, lumber canopy, loading pad and a truck dock. New pavements and site improvements are also anticipated.

It is anticipated foundation loads will be typical for a prototype Home Depot store. For the purpose of this report, maximum wall and column loads of about 4.6 kips per lineal foot for walls and 75.6 kips for columns are anticipated. When the actual loads are known, this information should be provided to Moore Twining for consideration.

Based on the October 17, 2016 Home Depot Design Criteria, the maximum uniform floor slab load will be 325 pounds per square foot for the sales floor area and 450 pounds per square foot for the sales floor lumber area. According to the Home Depot Design Criteria, maximum allowable total settlement for floor slabs and foundations shall not exceed 1 inch. The maximum allowable differential settlement for floor slabs and foundations shall not exceed ½ inch in 50 lineal feet. The maximum total heaving of the floor slab and foundations shall not exceed 1 inch and the maximum differential heaving of the floor slab and foundations shall not exceed ½ inch.

The proposed development will include driveways and parking for automobile and truck traffic. Equivalent 18 kip axle loads (EAL) of 50,000 and 220,000 for a design life of 10 years were indicated in the Design Criteria Manual for the Home Depot "standard duty" and "heavy duty" pavement sections, respectively.

At the time of preparation of this report, a grading plan had not been developed. However, it is anticipated a finished floor elevation for the Home Depot will be at similar or a slightly higher elevation as the existing former K-mart building.

4.0 INVESTIGATIVE PROCEDURES

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

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

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 Moore Twining on January 29, 2021 and the week of February 22 through February 25, 2021 . The features noted are described in the background information.

4.1.2 <u>Drilling Test Borings</u>: The depths and locations of test borings were selected based on the locations of the proposed improvements, type of construction, estimated depths of influence of proposed foundation loads and the subsurface conditions encountered.

On January 29, 2021 and February 22 through 25, 2021, fifteen (15) test borings were drilled at the site to depths of 10 to about 50 feet below site grade (BSG). Three (3) borings were drilled inside the existing K-mart building using a limited access dill rig equipped with 4 inch diameter solid flight augers. The test borings outside the existing buildings were drilled using a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow-stem augers. The test borings were drilled under the direction of a Moore Twining professional geologist and staff engineer. The soils encountered in the test borings were logged. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil. Soil samples were collected and returned to our laboratory for classification and testing.

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

Test boring locations were determined by reference to existing site features. The locations, as shown on Drawing No. 2 in Appendix A, should be considered approximate. Elevations of the test borings were not surveyed as a part of the investigation. In accordance with the boring permits obtained from Sonoma County, the test borings were backfilled with neat cement. The neat cement backfill was capped with rapid set concrete or asphalt cold patch to match the existing ground surface. Some settlement should be anticipated in the boring locations.

4.1.3 <u>Cone Penetration Test (CPT) Soundings</u>: In addition to the test borings, cone penetration testing (CPT) was performed at the site. On January 29, 2021, six (6) CPT soundings were advanced to a depth of 50 feet BSG in the general area proposed for the Home Depot store. CPT methods were used to obtain nearly continuous soil behavior type and penetration resistance information for use in liquefaction evaluation. The soundings were conducted under the direction of a Moore Twining professional geologist. The approximate CPT locations are shown on Drawing No. 2 in Appendix A.

The CPT soundings were performed by Middle Earth Geo Testing, Inc., using an electronic piezocone with a 60-degree apex angle and a diameter of 35.7 millimeters (about 1½ inches) hydraulically advanced using a 30-ton CPT rig in accordance with ASTM Test Method D3441. CPT measurements of cone bearing resistance, sleeve friction, and dynamic pore water pressure were recorded at 2 inch intervals during penetration to provide nearly continuous logs of the soil behavior types. The CPT logs are presented in Appendix B.

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

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

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 Improvements: The subject site includes an existing former K-mart building, retail shop buildings and associated site improvements. The existing K-mart building consists of a CMU block exterior wall construction with interior steel columns and a slab on grade floor. A depressed loading dock with a canopy area is located on the southwest side of the building. A row of retail buildings are located to the northeast of the Kmart and are separated by a canopy covered concrete walkway.

The existing structures are generally surround by concrete walkways and/or existing asphaltic concretre pavements. Existing underground utilities are located throughout the site area.

Additional information regarding the surface conditions are noted in the Background Information of this report.

5.2 **Soil Profile:** The near surface soils encountered below the existing pavement sections generally consisted of fat clay soils extending to depths ranging from approximately 5 feet to 15 feet BSG. The fat clays were generally underlain by interbedded silty sands, clayey sands, fat clays, sandy lean clays and lean clays to the maximum depth explored of 50 feet BSG. Fat clay fill soils were encountered in boring B-5 below the pavement section to a depth of 2½ feet below site grade.

The three (3) soil borings advanced within the K-mart building area encountered a concrete slab ranging in thickness from about 3.8 to 5.5 inches, which was underlain by 8.5 to 36 inches of clayey gravel. A layer of visqueen was encountered below the slab in two (2) of the borings. In boring B-3, the visqueen was encountered directly below the concrete slab and the visqueen in boring B-2 was encountered within the clayey gravel base layer at a depth of 8 inches below the concrete slab.

A soil boring advanced in the K-mart garden center area encountered a concretre slab thickness of 2.8 inches, over 2 inches of aggregate base.

The asphalt concrete pavement sections encountered in the soil borings ranged from about 3.1 to 5.8 inches of asphalt over 5.5 to 7 inches of aggregate base.

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 locations are presented in the logs of borings in Appendix B. The stratification lines in the logs represent the approximate boundary soil types; the actual in-situ transition may be gradual.

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

FAT CLAYS: The fat clays encountered were described as medium stiff to very stiff, as determined by standard penetration resistance, N-values, ranging from 5 to 18 blows per foot. The results of five (5) relatively undisturbed samples indicated dry densities of 86.1, 87.2, 91.2, 95.5 and 107.0 pounds per cubic foot. The moisture content of the fat clay samples tested ranged from 17 to 31 percent. Two (2) atterberg limits tests indicated plasticity indexes of 44 and 55 with liquid limits of 62 and 70, respectively. A consolidation test indicated that the fat clays have a “moderate” compressibility (about 8.9 percent consolidation under a load of 16 kips per square foot). The results of two (2) expansion index tests indicated a medium to high expansion potential (EI=57 and EI=129). A direct shear test indicated an internal angle of friction of 18 degrees with a corresponding cohesion value of 840 pounds per square foot.

LEAN CLAYS: The lean clays encountered were described as medium stiff to very stiff, as indicated by standard penetration resistance, N-values, ranging from 6 to 30 blows per foot. Four (4) relatively undisturbed samples revealed dry densities of 99.4, 101.2, 107.2 and 114.7 pounds per cubic foot. The moisture content of the lean clays tested ranged from 13 to 33 percent. Two (2) consolidation tests indicated that the fat clays have a “moderate” to “high” compressibility (about 8.6 and 11.9 percent consolidation under a load of 16 kips per square foot). Three (3) atterberg limits tests indicated plasticity indexes of 21, 22 and 34 with liquid limits of 38, 39 and 48, respectively.

SILTY SANDS: The silty sands encountered were described as medium dense to very dense, as determined by standard penetration resistance, N-values, ranging from 20 to 56 blows per foot. A relatively undisturbed sample revealed a dry density of 101.2 pounds per cubic foot. The moisture content of the silty sands tested ranged from 20 to 26 percent. An atterberg limits test indicated the silty sands to be non plastic.

CLAYEY SANDS: The clayey sands encountered were described as medium dense to dense, as determined by standard penetration resistance, N-values, ranging from 11 to 36 blows per foot. A relatively undisturbed sample revealed a dry density of 107.7 pounds per cubic foot. The moisture content of the clayey sands tested ranged from 16 to 20 percent. Two (2) atterberg limits tests indicated plasticity indexes of 15 and 19 with liquid limits of 39 and 37, respectively.

Chemical Tests: A chemical test performed on three (3) near surface soil samples indicated pH values of 8.6, 7.1, and 8.6, minimum resistivity values of 440, 500 and 410 ohm-centimeters; 0.0025, 0.003 and 0.0034 percent by weight concentrations of sulfate; and 0.004, 0.0037 and 0.012 percent by weight concentrations of chloride, respectively.

5.4 **<u>Groundwater Conditions</u>:** In general, groundwater was encountered in the soil borings from depths ranging from about 7.7 feet to 19½ feet BSG. Due to the Sonoma County requirements, the boreholes were generally immediately backfilled with neat cement after drilling.

The Site Closure Summary for the previous monitoring wells at the site indicated shallowest groundwater depth of 7.87 feet below site grade.

In October 2018, piezometers were installed by our firm at the nearby site and encountered groundwater at depths ranging from about 7½ to 12 feet BSG. Additional measurements made in March 2019 from these piezometers indicated free groundwater depths ranging from about 4 to 5½ feet BSG.

It should be recognized 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/measurements 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.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of geotechnical related construction specifications are summarized in the following subsections. The evaluations were based upon the subsurface conditions determined from the investigation, our review of the project site plan, 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 (Section 7.0).

6.1 Existing Site Conditions: The subject site is occupied by a former K-mart building, retail shops, a depressed loading dock ramp, a canopy structure, exterior concrete flatwork and asphalt concrete pavements. Existing underground utilities are located throughout the site.

As part of the site preparation, all subsurface structures and improvements such as existing foundations, utilities, etc. should be removed and disposed of from the site in a legal manner, and the excavations properly backfilled. In addition, all soils which are disturbed as part of the demolition activity should be excavated to expose undisturbed soils prior to backfilling the excavations as engineered fill.

It is our understanding removal of soils was conducted as part of closure of the auto service area. According to the Site Closure Summary, 111 tons of soil was removed from the site. Records of the documentation of compaction of the backfill of these excavations were not provided. Accordingly, for the purpose of this report, fill soils placed to backfill excavations associated with the auto service center closure are considered to be undocumented fill and should be removed and compacted as engineered fill as part of the site preparation. The depth and extent of the fill is not known.

In general, from a geotechnical standpoint, existing building materials such as concrete, concrete masonry units, and asphalt can be recycled and used as general fill on the project. However, recycled materials should not be used as fill below the proposed Home Depot building unless approved by Home Depot. From a geotechnical engineering perspective, these materials may be reused as engineered fill below the proposed parking and drive areas outside the limits of the building pad preparation, provided they are properly processed and prepared. Recycled materials should be processed by crushing to different particle sizes to achieve a well graded consistency such as a road base material.

If existing monitoring wells remain at the site from prior investigations. Proper abandonment of the wells will need to be conducted as part of construction.

6.2 Overly Moist Near Surface Soils, Stabilization and Groundwater: The onsite near surface soils have a high affinity for water. The moisture contents of the near surface fat clay soils are anticipated to be overly moist to achieve compaction. Therefore, the near surface fat clays are anticipated to require chemical treatment (as necessary) during site preparation, or aerated by thinly spreading the material and repeatedly discing or ripping the soils in optimal drying climate conditions to dry the soils in order to allow compaction as engineered fill to a stable condition.

Accordingly, contractors will need to anticipate these conditions and include the costs to dry the soils to achieve moisture contents suitable for compaction.

In addition, due to the shallow groundwater conditions and near surface soils with high moisture contents, excavations may require stabilization using chemical treatment, aeration, placement of a geotextile and bridge lift of crushed rock or a combination of these methods to achieve a stable surface for conducting earthwork operations.

Based on review of groundwater data for the subject site, groundwater was encountered as shallow as 7.7 feet BSG. Therefore, the need for dewatering and stabilization measures should also be anticipated for earthwork and utility trenching. Due to the site environmental history, there may be special requirements for handling and disposing of groundwater for this site. The dewatering should be conducted in accordance with all applicable regulatory and environmental project requirements.

It is our understanding monitoring wells may be installed to allow for monitoring of the groundwater depths.

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

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on samples of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The expansion index testing was performed in accordance with ASTM D4829. The results of expansion index testing indicated that the fat clay samples tested have a medium to high expansion potential (EI=57 and EI=129).

Due to the expansion potential of the near surface soils, recommendations for placement of imported, non-expansive engineered fills below all concrete slabs on grade are included in the Site Preparation section of this report to reduce, but not eliminate, the potential for heave. As an alternative to importing granular non-expansive fill, the site soils may be treated with high calcium quicklime to reduce the soil plasticity and use lime stabilized soils as the non-expansive fill. However, laboratory testing would need to be conducted in order to evaluate the effectiveness of lime treatment and to estimate the quantity of lime needed to achieve sufficient reduction in the soil plasticity. The potential for heave of concrete slabs on grade and other improvements sensitive to expansive soils movement should be recognized and considered in design and future maintenance. In addition, the native subgrade soils below the non-expansive fill will require moisture conditioning to reduce the potential for excessive heave. Maintaining sufficient moisture content in the clay subgrade soils below the imported non-expansive fill both during earthwork and after preparation of the subgrade soils will be critical to reducing the potential for excessive expansive soils movement.

The contractor should be required to maintain the moisture conditions of the clay soils and not allow them to dry below the recommended moisture contents in this report. In the event drying occurs, methods such as scarification, moisture conditioning and/or presaturation will be required by the contractor to re-establish the required moisture content conditions prior to placement of the non-expansive fill and overlying improvements. Where the clay subgrade soils are allowed to dry prior to construction of overlying improvements, the potential for future heave would increase significantly. Since exterior sidewalks, curbs, etc. are typically constructed some period of time after grading, the moisture conditioning conducted during earthwork can revert to drier conditions which would increase the potential for excessive heave of overlying improvements. Therefore, it is critical that the moisture conditioning of the subgrade soils be maintained by periodic wetting or other means to maintain adequate moisture conditions until the final subgrade preparation and placement of non-expansive fill and aggregate base.

In addition, due to the expansive soil conditions, providing sufficient cutoffs such as turned down edges for concrete slabs and curbs adjacent to landscape areas and maintaining as consistent a moisture content as possible in the soils within uncovered areas (such as landscaped or unimproved areas) and preventing over-irrigation and poor drainage in landscape areas will be critical to reducing the potential for subsequent shrink and swell type movement of the expansive soils. Also, joints in pavements, such as joints in concrete paving and joints between asphalt concrete and concrete paving, curbs, etc. should be sealed and maintained regularly to reduce infiltration of surface runoff into the subgrade, which can result in heaving of the subgrade over time. Differential heave which occurs at joints in pavements and flatwork from expansive soils movement can require future grinding and/or patching at joints to maintain slopes, etc.

6.4 **<u>Static Settlement and Bearing Capacity of Shallow Foundations:</u>** The potential for excessive total and differential static settlements of foundations and slabs-on-grade is a geotechnical concern evaluated for this building site. 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 structure 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.

Based on the subsurface data and laboratory testing performed as part of this report, static settlement calculations were performed. Calculations indicate that static settlement of foundations placed directly on the native soils at the proposed foundation depth would exceed the Home Depot design criteria for differential settlement of ½ inch in 50 feet. In order to meet the total combined (static plus seismic) settlement criteria of 1 inch total and ½ inch differential in 50 feet, the structure loads would need to be supported on a minimum of 6 feet of engineered fill below foundations. Due to the shallow groundwater conditions, specialty deep ground modification approaches should also be considered for support of the foundations as an alternative. However, in the event an exception can be allowed by Home Depot to allow 1 inch of total static settlement, ½ inch differential static settlement, ½ inch total seismic settlement, and ¼ inch differential seismic settlement, the structure loads could be supported on a minimum of 3 feet of engineered fill below foundations. Refer to the discussion in Section 6.7 of this report.

6.5 Seismic Ground Rupture and Design Parameters: The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The closest active fault is the Rogers Creek - Healdburg Fault, which is located approximately 4.7 miles west of the site. Accordingly, the potential for ground rupture at the site is considered low.

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

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

A table providing the recommended seismic coefficient and earthquake spectral response acceleration values for the project site is included in the Foundation Recommendations section of this report. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.728g 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 6.51 was determined for the site based on deaggregation analysis (United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/).

6.6 Liquefaction and Seismic Settlement: The site is located in a liquefaction hazard area based on local agency mapping (Association of Bay Area Governments). Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing usually results. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the common characteristics for liquefaction.

Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by test borings and CPT soundings, and the results of laboratory testing. The analyses were conducted for soils encountered in CPT soundings CPT-1 through CPT-6 using the software program LiquefyPro developed by CivilTech. A horizontal ground acceleration of 0.728g, a maximum considered earthquake of 6.51 and a groundwater depth of 5 feet were used in the analysis. The N-values generated based on the CPT results were used to determine the cyclic stress ratio needed to initiate liquefaction. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

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).

Based on our analysis, preliminary estimates of total seismic settlement are about ½ inch total and ¼ inch differential seismic settlement in 50 feet.

6.7 Mitigation of Static and Seismic: Based on our analysis, in order to meet the total combined (static plus seismic) settlements to within 1 inch total and ½ inch differential in 50 feet, the building pad would need to be prepared by over-excavation and placement of about 6 feet of engineered fill below the bottom of foundations. Due to the shallow groundwater conditions and highly plastic clay soils, excavation to prepare the pad in this manner would be expected to require substantial dewatering and treatment of wet soils. However, in the event a static settlement of 1 inch total and ½ inch differential in 50 feet, along with ½ inch total seismic settlement and ¼ inch differential seismic settlement in 50 feet can be accepted, the structure loads could be supported on a minimum of 3 feet of engineered fill below foundations. Note that regardless of the approach for foundation support and settlement criteria, the non-expansive fill materials will still need to be placed for support of the slab on grade and to reduce potential expansive soils movement.

It should be noted that special handing and disposal of groundwater may be required due to the past environmental history of the site. In addition, due to the shallow groundwater and overly moist soil conditions, stabilization and processing the soils as engineered fill could be a significant cost at this site. Thus, alternatives are presented herein.

As an alternative to over-excavation and compaction to support foundations, specialty ground modification methods such as cement deep soil mixing or rigid inclusions are potential methods can be used to limit the settlements to within tolerable limits for the subject site and would allow for less excavation in order to prepare the building pad. In order to reduce the potential for excessive static settlement, this approach would require that the deep ground modification elements are used for direct support of foundation loads, but that the slab would be supported on grade using a non-expansive fill. Deep ground modification would be a design-build option and typically would achieve a higher bearing pressure for design of spread foundations, which has the benefit of reducing the size of the foundations.. The costs of the deep ground modification should be compared with the over-excavation approach.

6.8 **Asphaltic Concrete (AC) Pavements:** Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report. The thicknesses of the asphalt concrete and the underlying aggregate base materials 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 loading of heavy trucks called a traffic index (T.I). In evaluation of the pavement design for this project, samples of the onsite soils anticipated to be representative of the soils which will support pavements were obtained and R-value testing was performed in accordance with ASTM D2844. The R-value test results are summarized in Appendix C of this report.

The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highways Design Manual. The traffic loading data were obtained from the Design Criteria Manual provided by Home Depot U.S.A., Inc. For the proposed Home Depot store, the "standard duty" pavement should be designed for a life of 10 years and an EAL (18 kips) of 50,000 axles. An EAL of 50,000 equates to a traffic index of 6.5. The "heavy duty" pavement was designed for a life of 10 years and an EAL (18 kips) of 220,000 axles. This equates to a traffic index of 7.5. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated. Based on the soil encountered, an R-value of 5 was used for design.

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

In evaluation of the pavement design for this project, samples of the onsite soils anticipated to be representative of the soils which will support pavements were obtained and R-value testing performed in accordance with ASTM D2844.

The EALs for each of the pavement sections were converted to the number of 5-axle trucks per day, one direction, anticipated for the proposed store. The EAL for the "standard duty" pavement section of 50,000 was converted to 14 axles or 6 five-axle trucks per day. The EAL for the "heavy duty" pavement section is 220,000 or 26 five-axle trucks per day. The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements."

The PCC pavement sections were designed for a life of 10 years, a load safety factor of 1.1, a single axle weight of 12,000 pounds, a tandem axle weight of 36,000 pounds. A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 6-inch layer of aggregate base material (minimum R-value of 78) was used for pavement design.

6.10 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. 1 below.

Table No. 1
Soil Resistivity and Corrosion Potential Ratings

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

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

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

6.11 Sulfate Attack of Concrete: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The test results are then compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The test results are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

The soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters.

7.0 CONCLUSIONS

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

7.1 From a geotechnical engineering standpoint, the site is suitable for the proposed construction with regard to support of shallow foundations and concrete slabs-on-grade, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation, design level investigation, and observation of clearing, earthwork and foundation installation activities by Moore Twining are integral to this conclusion.

7.2 The near surface soils encountered below the existing pavement sections generally consisted of fat clay soils extending to depths ranging from approximately 5 feet to 15 feet BSG. The fat clays were generally underlain by interbedded silty sands, clayey sands, fat clays and lean clays to the maximum depth explored of 50 feet BSG. Fat clay fill soils were encountered in boring B-5 below the pavement section to a depth of 2½ feet below site grade. Inside the existing building, the borings drilled encountered a clayey gravel fill material extending to depths of about 8½ to 36 inches.

7.3 Groundwater was encountered in soil borings at depths ranging from about 7.7 feet to 19½ feet BSG. Shallower groundwater conditions are anticipated since the borings were not left open for an extended period of time to determine stabilized groundwater depths, as restricted by the well (soil boring) permit. It is our understanding the project environmental consultant may be installing monitoring wells for future groundwater depth monitoring.

7.4 Due to the potential for excessive static settlement, the near surface soils are not adequate to support the proposed foundation loading. In order to meet the total combined (static plus seismic) settlement criteria of 1 inch total and ½ inch differential in 50 feet (Home Depot Design Criteria Manual), the structure loads would need to be supported on a minimum of 6 feet of engineered fill below foundations. However, due to the shallow groundwater conditions and earthwork cost impacts to dewatering, addressing wet soils, soils stabilization, etc., specialty deep ground modification approaches and/or alternative settlement criteria could also be considered for support of the foundations. Ground modification would be expected to allow for higher bearing pressures for use in design of foundations, thereby reducing the size of the foundations. For example, in the event an exception can be allowed by Home Depot to allow 1 inch of total static settlement, ½ inch differential static settlement, ½ inch total seismic settlement, and ¼ inch differential seismic settlement, the structure loads could be supported on a minimum of 3 feet of engineered fill below foundations. Refer to the discussion in Section 6.7 of this report.

7.5 Based on the results of testing conducted for this investigation, the near surface clay soils encountered exhibit a high plasticity and medium to high expansion (nearly very high) potential. In order to reduce the potential for excessive heave of slabs on grade, this report recommends non-expansive fill below concrete slabs on grade. As an alternative to importing granular, non-expansive fill materials, it may be possible to lime treat the onsite soils for use as a non-expansive fill.

7.6 Due to the shallow groundwater conditions, waterproofing of below grade improvements such as the loading dock may be required. As more information is available with regard to anticipated site grading, the extent of subsurface structures and elevations of features such as depressed loading docks, and the data from future monitoring of groundwater depths in the wells, methods to reduce impacts from the shallow groundwater conditions can be evaluated further. In addition, the extent of groundwater treatment and disposal methods for extracted groundwater will need to be evaluated based on the groundwater quality and regulatory agency requirements.

7.7 Based on the shallow groundwater conditions and the clay soil conditions, stormwater infiltration systems do not appear feasible at this site.

7.8 Based on our analysis, preliminary estimates of total seismic settlements are about ½ inch total and ¼ inch differential seismic settlement in 50 feet.

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

7.10 The analytical results of a soil sample analysis indicate that the near-surface soils exhibit a "extremely corrosive" corrosion potential to buried metal objects.

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

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 This report is preliminary and was not intended to include all of the soil borings at the frequency listed in the Home Depot Design Criteria Manual. Future geotechnical investigation of the site will need to be conducted to prepare final design level recommendations and in order to achieve the minimum boring requirements of Home Depot.

8.1.2 Based on the findings of this investigation, the structure loads for the proposed Home Depot store building should be either supported on the minimum recommended engineered fill below foundations achieved by over-excavation below the foundations. However, due to the shallow groundwater conditions and earthwork cost impacts associated with dewatering, addressing wet soils, soils stabilization, etc., specialty deep ground modification approaches and/or alternative settlement criteria could also be considered for support of the foundations to reduce construction impacts. Note that regardless of the approach for foundation support and settlement criteria, the non-expansive fill materials recommended below the slabs on grade within the building pad limits will still need to be placed for support of the slabs on grade and to reduce the potential for expansive soils movement.

8.1.3 The proposed excavation and deep ground modification (if conducted) activities should be reviewed by the project environmental consultant and any applicable comments should be incorporated into the project requirements.

8.1.4 Shallow groundwater conditions are prevalent at the site. Thus, construction dewatering should be anticipated for the project. As more information is available with regard to anticipated site grading, extent of subsurface structures such as elevations of depressed loading docks, and the data from future monitoring of groundwater depths in the wells, methods to reduce impacts from the shallow groundwater conditions can be evaluated further. Waterproofing or other measures may be required for below grade features. In addition, the extent of groundwater treatment and disposal/permitting methods for extracted groundwater will need to be evaluated based on the groundwater quality and regulatory agency requirements.

8.1.5 A dewatering plan should be designed by an appropriate professional hired by the Contractor. The plan should be provided as a submittal to Moore Twining and Home Depot’s environmental consultant for review and comment a minimum of 2 weeks prior to the start of construction. The plan should identify the components of the dewatering system, the anticipated duration of dewatering, the groundwater depths targeted for the design and operation of the system, the means for which the groundwater depths will be monitored, the groundwater treatment systems and discharge points.

8.1.6 Grading plans were not available at the time this report was prepared. When grading plans are developed, the plans should be provided to Moore Twining for review and consideration. Updated recommendations may be appropriate after review of the plans.

8.1.7 A preconstruction meeting including, as a minimum, the owner, general contractor, specialty ground improvement contractor, earthwork contractor, contractor’s land surveyor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

8.1.8 A demolition plan should be developed to identify the existing surface and subsurface improvements to be removed and to remain.

8.1.9 The contractor should be required to monitor the existing improvements to remain within influence of the proposed excavations and construction activities for settlement/movement during construction.

8.1.10 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 notify the client in writing and conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect information as required to prepare accurate bids.

8.1.11 The Contractor should use appropriate grading equipment such as low-pressure equipment, steel tracks, etc. to achieve the required over-excavation, compaction and subgrade stabilization to minimize rutting and subgrade instability.

8.1.12 Contractors should also be aware that wet soils are anticipated that will be significantly above the moisture content required for proper compaction and could require soil drying or chemical treatment for stabilization to achieve the required relative compaction. No change orders will be allowed for wet weather conditions, wet soil, soil instability, etc. including chemical treatment, geotextile fabric, rock, soil import, etc.

8.1.13 Based on the high moisture contents determined for the near surface soils, the Contractor should anticipate unstable soil conditions will be encountered during excavations and installation of slabs-on-grade, foundations, utilities, etc. Therefore, the soils will require stabilization. The Contractor should note that the base bid should include stabilization of the bottom 18 inches of the over-excavation with 6 percent high calcium quicklime.

8.2 Site Grading and Drainage

8.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least ten feet away from the structures to preclude ponding of water adjacent to foundations. Adjacent exterior grades which are paved should be sloped at least 1 percent away from the foundations, or as required by the California Building Code, whichever is more stringent.

8.2.2 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. Care should be taken to maintain a leak-free sprinkler system.

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

8.2.4 Perimeter curbs should be extended to the bottom of the aggregate base section, where irrigated landscape areas meet pavements.

8.2.5 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 proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

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

8.2.7 Due to the potential for expansive soils movement due to wetting, storm water infiltration facilities which discharge water into the near surface soils have the potential to cause excessive static settlement of the proposed improvements, including foundations, site improvements, etc. In addition, the shallow groundwater conditions impact the feasibility of stormwater infiltration at this site. Accordingly, storm water infiltration systems are not recommended for this project. If any storm water collection systems are planned for the project, they should be located at least 25 feet away from structures. Improvements adjacent to stormwater collection systems which are sensitive to differential movement, heave, etc. should be protected by use of cutoff trenches, waterproofing membranes, or other design elements.

8.3 Site Preparation

8.3.1 Existing surface and subsurface improvements in the areas of new construction 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. Where present, existing utility 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 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 All surface topsoil, vegetation, roots, organics, surface and subsurface improvements (if any) should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic top soils. 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.

8.3.3 The area of undocumented fill placed for the auto service area closure should be located and removed to expose undisturbed, native soils. After approval of the bottom of the over-excavation by a representative of Moore Twining, the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned (this includes drying and/or wetting the soils as necessary to achieve the specified moisture content of the soils) to between two (2) and five (5) percent above optimum moisture content and compacted to at least 90 percent relative compaction of the maximum dry density as determined by ASTM Test Method D1557 to achieve a stable compacted subgrade. Areas of instability should be stabilized by chemical soil treatment or placement of a bridge lift of 1½ inch crushed gravel encapsulated in a geotextile filter fabric prior to placement of fill.

8.3.4 **Over-Excavation of Building Pad**: After site stripping and removal of existing surface and subsurface improvements, the near surface soils throughout the building pad limit indicated in Section 8.3.5 of this report (no slot cutting of foundation areas will be allowed) and all foundation should be over-excavated to the following based on the appropriate design criteria specified by Home Depot for the project:

Over-excavate to achieve all of the below criteria for a combined static and seismic settlement of 1 inch total and ½ inch differential in 50 feet for foundations:

- Over-excavate to at least 3 feet below preconstruction site grades;
- Over-excavate to remove all undocumented fill soils;
- Over-excavate to at least 12 inches below the bottom of existing improvements to be removed;
- Over-excavate to at least 6 feet below the bottom of all foundations (including truck dock); and,
- Over-excavate to 12 inches below the bottom of the proposed aggregate base

<u>Over-excavate to achieve all of the below criteria for a static settlement of 1 inch total and ½ inch differential in 50 feet and ½ inch seismic total and ¼ inch differential in 50 feet:</u>

- Over-excavate to at least 3 feet below preconstruction site grades;
- Over-excavate to remove all undocumented fill soils;
- Over-excavate to at least 12 inches below the bottom of existing improvements to be removed;
- Over-excavate to at least 3 feet below the bottom of all foundations (including truck dock); and,
- Over-excavate to 12 inches below the bottom of the proposed aggregate base.

8.3.5 The building pad is defined as the areas to be occupied by the building, garden center, all foundations, adjacent slabs-on-grade, adjacent sidewalks, vestibules, building apron, lumber canopy and drive-through area, rear lumber area, loading dock, materials storage areas, utility pads, stairs, ramps, stoops, canopy footings and loading pads, and to a minimum horizontal distance of five (5) feet beyond the concrete slabs and a minimum of five (5) feet beyond all foundations. Regardless of the approach for preparation of the building pad, all concrete slabs on grade within the building pad preparation limits should be supported on 6 inches of aggregate base over a minimum of 30 inches of non-expansive fill, over the native subgrade soils prepared as recommended in this report.

8.3.6 After approval of the bottom of the building pad over-excavation by a representative of Moore Twining and review of the contractor's survey data, the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned (this includes drying and/or wetting the soils as necessary to achieve the specified moisture content of the soils) to between two (2) and five (5) percent above optimum moisture content and compacted to at least 90 percent relative compaction of the maximum dry density as determined by ASTM Test Method D1557 to achieve a stable compacted subgrade. The moisture content and compaction of the subgrade soils should be maintained until placement of the aggregate base.

8.3.7 High moisture contents are anticipated within the near surface soils and stabilization of wet soils will be required as part of site preparation. Drying of the soils, such as by chemical soil treatment should be anticipated to "dry up" the soils for use as engineered fill due to these conditions. The Contractor's base bid should include stabilization of the bottom 18 inches of the building pad over-excavation with 6 percent high calcium quicklime

8.3.8 The project civil engineer should show the horizontal limits of over-excavation for the building pad on the project plans.

8.3.9 Areas to receive fill, asphaltic concrete pavements, PCC pavements, and exterior slabs (outside the building and overbuild zones) should be prepared by excavation to achieve the following criteria, whichever provides the deeper fill:

- Over-excavate to at least 12 inches below preconstruction site grades;
- Over-excavate to at least 12 inches below the bottom of existing improvements to be removed;
- Over-excavate to remove all undocumented fill soils (encountered to depths of about 1½ feet BSG); and
- Over-excavate to 12 inches below the bottom of the proposed aggregate base.

The zone of over-excavation and compaction (overbuild zone) should extend laterally a minimum of 3 feet outside the perimeters of exterior slabs and walkways that are outside the building's overbuild zone. The bottom of the excavation should be scarified to a minimum depth of 8 inches, conditioned (wetted or aerated) as necessary, and compacted as engineered fill and to achieve a firm non-yielding condition prior to backfilling to pad grade. The Contractor should anticipate that the exposed bottom of the excavation and the on-site soils used as fill will need to be chemically treated, air dried, etc. to achieve the required moisture contents and the required relative compaction. The Contractor should include the costs to chemically treat, air dry, etc. these soils in their bids. Following the scarification and compaction of the bottom of the excavation, the excavation may be backfilled with engineered fill.

Exterior slabs-on-grade should be underlain by a minimum of 6 inches of aggregate base over a minimum of 18 inches of imported, non-expansive fill over the depth of engineered fill recommended above aggregate base and Portland cement concrete pavement should be underlain by a minimum of 6 inches of aggregate base over 12 inches of imported, non-expansive engineered fill over the depth of engineered fill recommended above.

8.3.10 Following stripping and removal of existing surface and subsurface improvements, areas to receive miscellaneous lightly (less than 1.5 kip per foot) loaded foundations such as site walls and retaining walls, should be over-excavated to a minimum of 1 foot below the bottom of foundations; to at least 2 feet below preconstruction site grades; to the depth required to remove existing undocumented fills (if any); and to at least 12 inches below subsurface improvements (structures, utilities, etc.) to be removed, whichever is greater. The over-excavation for retaining walls/screen walls should extend to at least 3 feet beyond the edge of the foundations. The bottom of the over-excavation should be scarified to a depth of at least 8 inches, moisture conditioned and compacted as engineered fill.

8.3.11 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. In addition, the contractor should survey the layer of non-expansive soils (aggregate base, non-expansive imported fill, etc.) top and bottom, to verify that the minimum thicknesses specified in this report have been achieved. Moore Twining is not responsible for surveying and verifying the horizontal and vertical extent of over-excavation and compaction. This is the sole responsibility of the contractor. It is recommended that the contractor prepare an over-excavation plan that conforms to the requirements of this report, the project plans, and specifications for submittal to Home Depot, the civil engineer and Moore Twining for review and comment a minimum of 7 days prior to start of earthwork. The requirements for demolition, as presented in this report and the demolition plan, should be incorporated into the over-excavation plan prepared by the contractor. The contractor should hire a land surveyor to verify in writing to Home Depot 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). The verification shall include an as-built plan signed by the contractor's surveyor showing the horizontal over-excavation limits and elevations of the bottom of the over-excavation in relation to the proposed building pad improvements. This verification should be provided prior to requesting pad certification from Moore Twining or excavating for foundations.

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

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

8.3.14 The Contractor should use appropriate equipment to achieve the required over-excavation, compaction and subgrade stabilization to prevent rutting and subgrade instability.

8.3.15 Final grading should produce building pads ready to receive the slab-on-grade which is smooth, planar, and resistant to rutting. Both the finished pad (before aggregate base is placed) and the aggregate base section should not depress more than one-half (½) inch under the wheels of a fully loaded concrete truck. 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.

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

8.4 Engineered Fill

8.4.1 The on-site near surface soils encountered are predominantly fat clay soils and should not be used where non-expansive fill materials are recommended. The existing near surface soils that are free of organics (less than 3 percent by weight), free of debris, and oversized materials greater than 3 inches in largest dimension will be suitable for use as fill material at depths in excess of 36 inches below the bottom of the proposed concrete slabs on grade within the building pad preparation limits (i.e., building slab and concrete slabs adjacent to the building), below a depth of 24 inches below the bottom of the proposed exterior slabs outside the building pad preparation limits, and below a depth of 18 inches below the bottom of PCC pavement, provided they are properly moisture conditioned and compacted. The onsite soils will not be suitable for use as engineered fill where imported granular fill is recommended. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

As an alternative to importing granular non-expansive fill materials, it may be possible to lime treat the onsite soils for use as a non-expansive fill. In order to evaluate the feasibility of lime treatment, laboratory testing would need to be conducted to determine the optimum lime content and confirm sufficient reduction in soil plasticity.

8.4.2 From a geotechnical engineering standpoint, onsite asphalt concrete and concrete building materials from demolition could be recycled for use as fill on the project for areas outside the building pad. If this use is proposed, these recycled materials should be processed to a maximum particle size of 2 inches and a minimum of 60 percent passing the No. 4 sieve.

8.4.3 Flyash may not be used for treatment of soils on the project.

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

8.4.5 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.

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

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 20
Plasticity Index (ASTM D4318)	Less than 15
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Resistivity	> 3,000 ohms-cm
R-value	≥15

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. 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.

8.4.7 On-site clay soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between one (1) and four (4) percent above optimum moisture content, and compacted to 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.

8.4.8 On-site granular soils or imported granular soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to 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.

8.4.9 Where lime treatment of soils is conducted, the lime treatment shall be conducted in accordance with the State of California Standard Specifications, with exception that the relative compaction should be determined in accordance with ASTM procedures based on the maximum dry density in accordance with the engineered fill recommendations of this report.

8.4.10 Utility trenches should be a minimum of 24 inches in width to allow for in-place density testing by traditional (nuclear density test) methods and the backfill should be compacted in accordance with the recommendations for engineered fill.

8.4.11 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at the minimum frequency listed in Table No. 2, below.

Table No. 2
Minimum In-place Density Test Frequency

Area	**Minimum Test Frequency**
Building Pad	1 test per 2,500 square feet per lift
Pavements	1 test per 5,000 square feet per lift
Utility Pipe and Structure Backfill	1 test per 100 linear feet of trench per compacted lift

8.4.12 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 or pipe manufacturer for use as backfill, or for stabilization of trenches, 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. In addition, periodic slurry cutoffs should be provided along trenches where gravel is placed to reduce potential impacts from groundwater migration through the gravel materials. 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.

8.4.13 Aggregate base below the building slabs should be non-recycled and should comply with State of California Department of Transportation requirements for a Class 2 aggregate base or Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. The aggregate base used below the building pad should not contain recycled materials. However, a recycled Class 2 aggregate base may be used for pavement areas outside the building pad, provided that the recycled materials are accepted by the Owner and adequate quality control testing is conducted. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.5 <u>Shallow Spread Foundations</u>

8.5.1 Foundations may be supported on spread or continuous footings placed entirely on engineered fill soils as recommended in the Site Preparation section of this report. Spread and continuous footings supported on engineered fill soils may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design. It should be noted that ground modification such as rigid inclusions or deep soil mixing could be considered to reduce earthwork construction impacts associated with excavation of wet soils and dewatering. If ground modification is used for the project, the foundations should be designed in accordance with the recommendations of the design build ground modification.

8.5.2 Perimeter foundations should extend to a minimum depth of 36 inches below the top of the floor slab and a minimum of 36 inches below the lowest adjacent finished grade. Interior footings should extend to a minimum of 30 inches below the top of the interior floor slab. All footings should have a minimum width of 18 inches, regardless of load.

8.5.3 Foundations should be designed based upon combined static and seismic settlement of 1 inch total and ½ inch differential in 50 feet for foundations underlain by 6 feet of engineered fill; or foundations should be designed based upon static settlement of 1 inch total and ½ inch differential in 50 feet and ½ inch seismic total and ¼ inch differential in 50 feet for foundations supported on 3 feet of engineered fill. Home Depot should determine the allowable settlement criteria for the project.

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 The following seismic factors were developed for the site using the Ground Motion Parameter Calculator provided by SEOAC and OSHPD (http://seismicmaps.org), based upon a site latitude of 38.249652 degrees and a site longitude of -122.631503 degrees. The data provided in Table No. 3 are based upon the procedures of Sections 1613.2.1 through 1613.2.4 of the 2019 California Building Code, ASCE 7-16 and Supplement No. 1 to ASCE 7-16. The data in Table No. 3 were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 2 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. 3

Seismic Factor	2019 CBC Value
Site Class	E
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.728g
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-16 (PGA)	0.662g
Spectral Response At Short Period (0.2 Second), Ss	1.57
Spectral Response At 1-Second Period, S_1	0.6
Site Coefficient (based on Spectral Response At Short Period), Fa	See Note 1
Site Coefficient (based on spectral response at 1-second period) Fv	See Note 1
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	See Note 1
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note 1
Five percent damped design spectral response accelerations for short period, S_{DS}	See Note 1
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	See Note 1

Note 1: 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.

8.5.6 Pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot per foot of embedment may be used to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained design (Section 1806.8.2.1). A value of 150 pounds per square foot per foot of depth may be used.

8.5.7 At the time of pier construction and until the concrete is placed, the shaft excavation should have stable sidewalls and all sloughed soil should be completely removed from the bottom of the excavation. If the drilled hole exhibits instability, it should be cased. Moore Twining should observe the excavation to confirm that the pier was constructed as described above, and the soils encountered are similar to those indicated in this report.

8.5.8 Moore Twining should observe the bottom of foundation excavations prior to the placement of reinforcing steel and utilities. The Contractor shall provide a minimum of 48 hours notice for these observations.

8.5.9 The moisture contents of the footing excavations and prepared subgrade soils should be maintained in accordance with the recommendations for engineered fill until placement of concrete. If the excavations are allowed to dry, conditioning and remedial measures should be conducted to establish the minimum recommended moisture contents and relative compaction at no cost to Home Depot.

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

8.5.11 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 0.25 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

8.5.12 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot for level soil conditions. The upper 12 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance.

8.6 Site Retaining Walls (Less Than 5 Feet in Height) and Loading Dock Retaining Wall

8.6.1 Lightly loaded foundations (i.e., less than 1.5 kips/foot line loading) for short retaining walls extending a minimum of 30 inches deep designed using an allowable soil bearing pressure of 1,500 pounds per square foot or less may be supported on shallow footings placed entirely on 24 inches of engineered fill.

8.6.2 Retaining walls should be constructed with imported non-expansive granular free-draining backfill placed within the zone extending from a distance of 1 foot laterally 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 backfill will reduce the effects of expansive soil pressures on the wall. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	70 - 100
Percent Passing No. 200 Sieve	15 maximum
Plasticity Index	Less than 5

8.6.3 The import fill material should be tested and approved as recommended under the subsection entitled “Engineered Fill” in the recommendations section of this report.

8.6.4 Retaining walls should be constructed with a drain system including, as a minimum, drain pipes surrounded by at least 1 cubic foot of crushed ¾ inch or ½ inch rock backfill fully encapsulated in Mirafi 140 N, or equivalent. The final selection of filter fabric should be as recommended by the fabric manufacturer for the specific site conditions. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Drainage should be directed to the site storm drain system. The drainage system should be designed by the wall designer and detailed on the plans.

8.6.5 For loading dock area retaining walls only, as an alternative to using drain pipes behind the wall to adequately reduce the potential for hydrostatic pressures behind the wall, weep holes may be used, provided that a continuous crushed rock (minimum 1 cubic foot per lineal foot) and filter fabric section is provided directly behind the wall. The weep holes cannot have the potential for clogging. The weep holes should discharge directly to an approved drainage.

8.6.6 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 0.25 can be used for design.

8.6.7 The allowable passive resistance of the onsite soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

8.6.8 The active and at-rest pressures of the wall backfill using imported granular fill meeting the recommendations of Section 8.6.2 in a drained condition may be assumed to be equal to the pressures developed by a fluid with a density of 43 and 65 pounds per cubic foot, respectively. These pressures also 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.

8.6.9 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.

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

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

8.6.12 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.

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

8.7 Interior Concrete Slabs-on-Grade

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

8.7.2 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed floor slab. Concrete slabs on grade supported on subgrade soils prepared as recommended in this report should be designed for a total static settlement and heave of 1 inch total and ½ inch differential over 50 feet.

8.7.3 A modulus of subgrade reaction of 150 psi/inch may be used for design of the interior floor slab when the subgrade preparation is conducted in accordance with the recommendations of this report.

8.7.4 Concrete slabs on grade within the building pad should be supported on a minimum of 6 inches of non-recycled Class 2 aggregate base over a minimum of 30 inches of imported non-expansive engineered fill over the depth of engineered fill required below the foundations. The minimum thickness of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction purposes.

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

8.7.6 The moisture content of the upper 12 inches of the non-expansive fill soils below the aggregate base section should be at least 3 percent above optimum moisture content 48 hours prior to placing the aggregate base section.

8.7.7 A vapor retarder should be placed below interior slabs where moisture could permeate into the interior and create problems. Recent guidance from Home Depot's concrete consultant (Structural Services Incorporated) recommends that a vapor retarder should be used below all interior concrete slabs on grade. Thus, a vapor retarder should be included below the building slabs and specified by the appropriate design professional. Refer to the American Concrete Institute's Guide to Concrete Floor and Slab Construction (ACI 302-1R-15) for selection and installation of moisture vapor retarders. It is recommended that Stegowrap 15 be used where moisture could permeate into the interior and create problems. The vapor retarder should overlay the 6 inches of aggregate base. It should be noted that placing the PCC slab directly on the vapor retarder may increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab unless a watertight roofing system is in place prior to slab construction to reduce the amount vapor emission through the slab-on-grade. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM D1745 Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs and ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of floor covering, floor covering adhesive or other slab material applications be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

8.7.8 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.

8.7.9 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations.

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

8.7.11 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.52 or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the 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.

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

8.7.13 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) use perimeter pour-strips at tilt-wall locations to avoid damage to slab-wall connections; 2) design for a differential slab movement of ½ inch relative to interior columns; 3) provide aggregate base below the slabs, 4) it is expected that erection of concrete tilt-up wall panels and roof steel may require cranes. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.7.14 For tilt up construction, a perimeter pour strip between the wall footing and the adjacent interior slab should be incorporated into the project design. After the walls are erected and a majority of the differential movement has occurred, the pour strip should be placed.

8.7.15 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill and/or an aggregate base section 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.

8.7.16 If the pad subgrade 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 loading.

8.7.17 Aggregate base shall be compacted to a minimum relative compaction of 95 percent of the maximum dry density determined in accordance with ASTM D1557. The Contractor shall test the aggregate base for sulfate content and provide the results to the Owner, Architect and Moore Twining for approval prior to delivery of the aggregate base to the site.

8.8 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. outside the building pad.

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

8.8.2 Subgrade soils for exterior slabs, should be prepared as recommended in the "Site Preparation" section of this report. Exterior slabs within the Home Depot building pad preparation limits should be supported on a minimum of 6 inches of aggregate base over a minimum of 30 inches of imported, non-expansive fill, over subgrade soils prepared as recommended in this report. Upon completion of the over-excavation and compaction of subgrade soils outside the building pad preparation limits, the exterior slabs and curb and gutter should be supported on a minimum of 6 inches of Class 2 aggregate base over a minimum of 18 inches of imported non-expansive engineered fill over subgrade soils prepared in accordance with recommendations of this report.

8.8.3 The clay subgrade soils should be not be allowed to dry prior to placement of the non-expansive fill. The moisture content of the onsite clayey subgrade soils below the imported, non-expansive fill should be verified to be at least 3 percent above optimum moisture content to a minimum depth of 12 inches within 48 hours of placement of the non-expansive fill below slab-on-grade or curbs and gutters. If necessary to achieve the recommended moisture content, the subgrade should be over-excavated, moisture conditioned as necessary and compacted as engineered fill

8.8.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend at least the bottom of the non-expansive fill below exterior slabs (i.e., a minimum depth of 24 inches below the bottom of exterior slabs).

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

8.9 Asphaltic Concrete (AC) Pavements

8.9.1 Areas for AC pavement should be prepared in accordance with the recommendations section entitled, "Site Preparation." The upper 12 inches of subgrade beneath the aggregate base should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.9.2 The following pavement sections are based on an R-value of 5 and a traffic index of 6.5 for the "Standard Duty Pavements," and a traffic index of 7.5 for the "Heavy Duty Pavements." 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.

Traffic Index = 6.5 "Standard Duty Pavements"

AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	Min. Compacted Subgrade, inches
3.5	14.5	12

Traffic Index = 7.5 "Heavy Duty Pavements"

AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	Min. Compacted Subgrade, inches
4.0	17.5	12

AC - Asphaltic Concrete compacted as recommended in Section 8.9.10 of this report
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

8.9.3 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 the potential for subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.9.4 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.

8.9.5 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.

8.9.6 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.

8.9.7 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the State of California Standard Specification Requirements.

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

8.9.9 The asphaltic concrete, including the joint density, should be compacted to a minimum 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.

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

8.10 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 and was based on the Home Depot traffic loading requirements. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

8.10.1 Areas to receive PCC slabs-on-grade should be prepared in accordance with the recommendations section entitled, "Site Preparation." After over-excavation and compaction, the upper 12 inches of subgrade beneath the aggregate base and imported non-expansive engineered fill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.10.2 The "standard duty" pavements and light vehicular loaded pavements were designed based on an 18 kip ESAL of 50,000 using a 10 year design. A design k-value of 130 psi/in was used considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over 12-inch layer of Imported, non-expansive fill over native compacted soils.

Pavement Component	Thickness, Inches
Portland Cement Concrete	7.0
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Imported, non-expansive Fill (95% Minimum Relative Compaction)	12.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

8.10.3 The "heavy duty" pavement section was designed based on an 18 kip ESAL of 220,000, a design period of 10 years, and a k-value of 130 psi/in considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over a 12-inch layer of Imported, non-expansive fill over native compacted soils.

Pavement Component	Thickness, Inches
Portland Cement Concrete	7.5

Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Imported, non-expansive Fill (95% Minimum Relative Compaction)	12.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

8.10.4 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 should be considered carefully and applied with clear details on the project plans by the pavement designer/detailer.

8.10.5 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 8.10.4, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement. 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.

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

8.10.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels, Diamond Dowels, etc. are recommended for construction joints to transfer loads. The joint details should be specified by the pavement design engineer and provided on the plans.

8.10.8 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.

8.10.9 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.

8.10.10 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.

8.10.11 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.

8.10.12 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.

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

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

8.10.15 Pavement construction should conform to the State of California Standard Specifications.

8.11 Slopes and Temporary Excavations

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

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

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

8.11.4 Shoring 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.

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

8.12 Utility Trenches

8.12.1 Based on the saturated near surface soils encountered, the Contractor should anticipate the need for stabilization of the bottom of utility trenches. In addition, excavations encroaching groundwater will require dewatering provisions.

8.12.2 This report recommends placement of a non-expansive select fill or lime treated engineered fill below the interior and exterior concrete slabs, curbs and gutters and PCC paving. Trench backfill conducted in areas of imported granular fill in the building pad, exterior slabs, PCC paving and curb and gutter areas, etc. will be required to selectively excavate, stockpile and backfill the trenches with the low-expansive fill such that the trench backfill meets the recommendations of this report for the minimum thickness of granular, low-expansive fill below concrete slabs on grade. If the select fill is blended with onsite soils during excavation, the mixed soils will not be acceptable for final trench backfill in the building pad area and an import, granular fill meeting the recommendations of this report (see Engineered Fill section) will be required. Trenching conducted in areas of chemically treated soils are not anticipated to yield spoils which will be suitable for reuse as trench backfill, unless the materials were selectively excavated and re-processed. Accordingly, aggregate base materials are recommended for trench backfill in areas of chemically treated engineered fill.

8.12.3 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.

8.12.4 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site soils or imported, non-expansive materials meeting the recommendations in the Engineered Fill section (Section 8.4) of this report. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.12.5 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 D2321-00 listed in the table below (minimum manufacturer requirements), or as necessary to provide sufficient space to achieve the required compaction, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Minimum Trench Widths for HDPE Pipe with Sand Bedding Initial Backfill

Inside Diameter of HDPE Pipe (inches)	Outside Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321-00
12	14.2	30
18	21.5	39
24	28.4	48
36	41.4	64
48	55	80
60	67.3	96

8.12.6 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill, 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.

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

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

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

8.12.10The plans should note that the backfill for all utility trenches, including electrical lines, irrigation lines, etc. should be compacted in accordance with the requirements for engineered fill included in Section 8.4 of this report, including the non-expansive fill required below interior and exterior concrete slabs, etc.

8.12.11Utility 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.

8.13 Corrosion Protection

8.13.1 Based on the National Association of Corrosion Engineers corrosion severity rating listed in Section 6.12 of this report, the analytical results of sample analyses indicate a “extremely corrosive” corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on these conditions. 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.

8.13.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (negligible exposure). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio is recommended for slabs on grade as recommended for exposed concrete slabs to reduce shrinkage.

8.13.3 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 the plan review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

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

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

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

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

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

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

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

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

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

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

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in 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

Zubair Anwar, RCE
Project Engineer

DRAFT

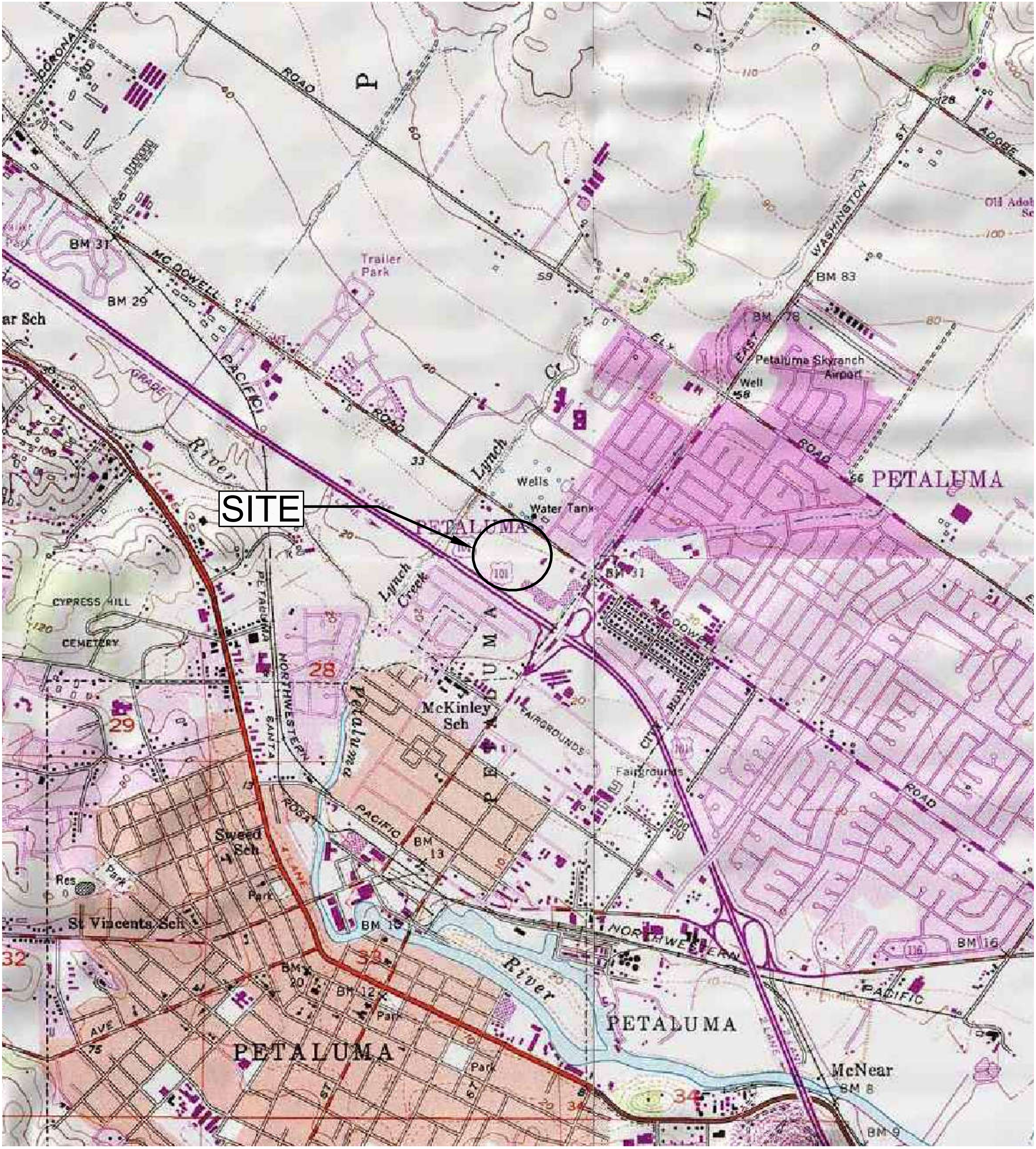
Read L. Andersen, RGE
Manager

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map

Drawing No. 2 - Test Boring and CPT Sounding



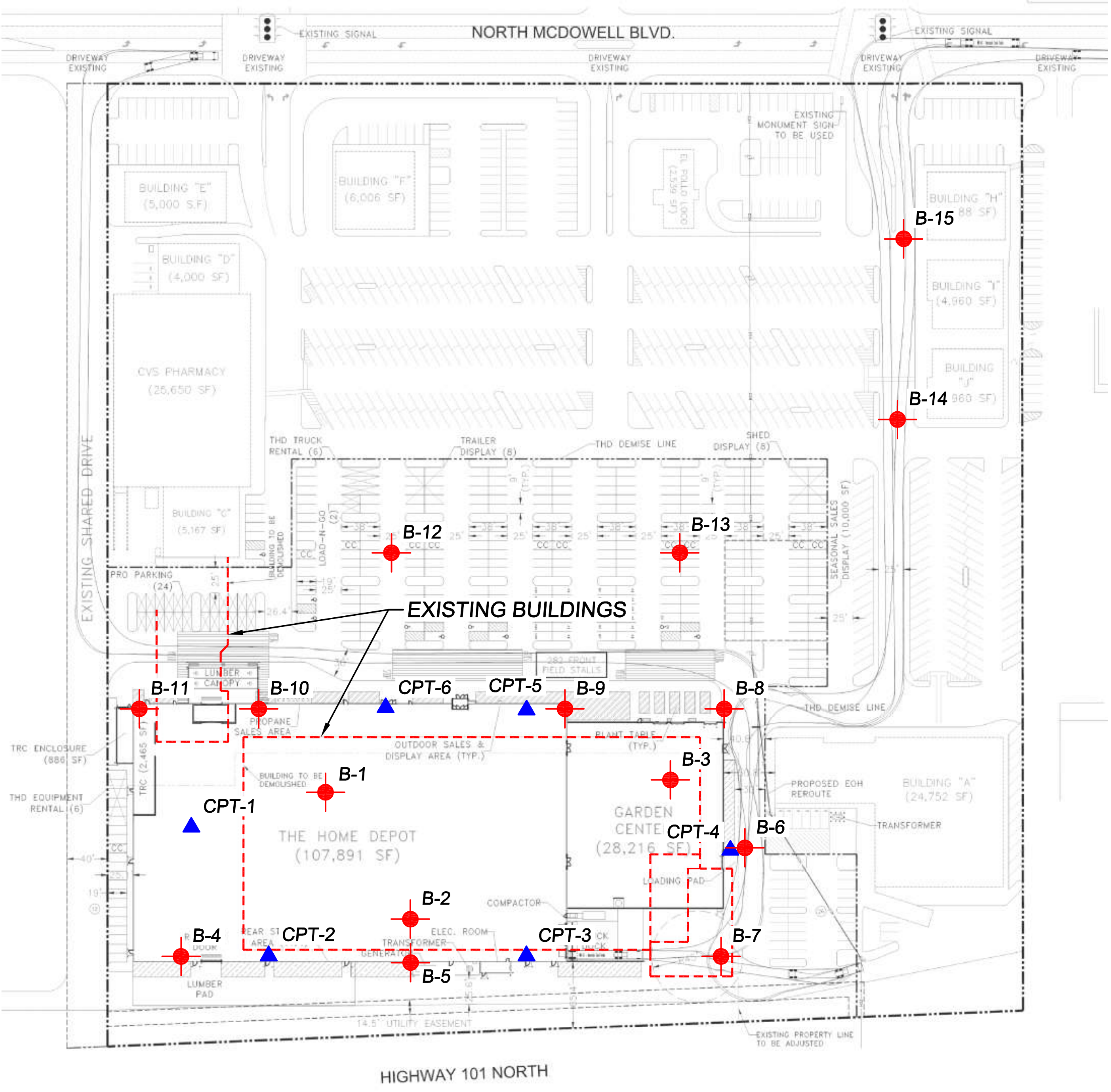
SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
COTATI, CALIFORNIA QUADRANGLE 1979, PHOTOREVISED 1980

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP
PROPOSED HOME DEPOT STORE
261 NORTH MCDOWELL BOULEVARD
PETALUMA, CALIFORNIA

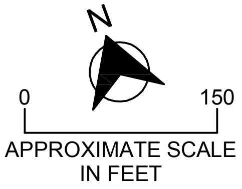
FILE NO.: 050S1-01-01	DATE: 03/29/21
DRAWN BY: RM	APPROVED BY:
PROJECT NO. D050S1.01	DRAWING NO. 1

MTA Since 1898
MOORE TWINING
ASSOCIATES, INC.



🔴 APPROXIMATE TEST BORING LOCATION

▲ APPROXIMATE CPT LOCATION



TEST BORING AND CPT LOCATION MAP PROPOSED HOME DEPOT STORE 261 NORTH MCDOWELL BOULEVARD PETALUMA, CALIFORNIA	FILE NO. 050S1-01-02	DATE DRAWN: 03/29/21	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. D050S1.01	DRAWING NO. 2	

APPENDIX B

LOGS OF BORINGS AND CONE PENETRATION TESTING

This appendix contains the final logs of borings and CPTs. 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 Takeover

Project Number: D050S1.01

Drilled By: ClearHeart Drilling

Drill Type: DR8K Track Rig

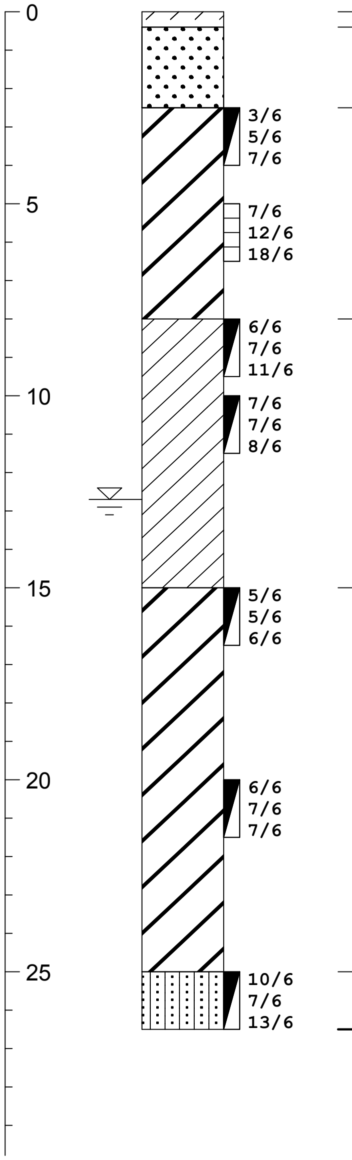
Auger Type: 4' Solid Flight Augers

Hammer Type: 140 Pound Cathead Hammer

Logged By: AH

Date: January 29, 2021

Depth to Groundwater First Encountered During Drilling: 12.7 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		PCC	Portland Cement Concrete = 3.8 inches			
			Clayey Gravel = 25.7 inches			
	3/6 5/6 7/6	CH	FAT CLAY WITH SAND; very stiff, moist, high plasticity, mottled, black		12	
5	7/6 12/6 18/6		Mottled light gray and dark gray	DD=87.2 pcf LL=62 PI=44 Sand=29.6% -#200=70.4%	30	27.4
	6/6 7/6 11/6	CL	LEAN CLAY; very stiff, moist, medium to high plasticity, light brown, with manganese and iron oxide staining		18	
10	7/6 7/6 8/6		Stiff with manganese staining		15	
15	5/6 5/6 6/6	CH	FAT CLAY; stiff, wet, high plasticity, brown		11	
20	6/6 7/6 7/6				14	
25	10/6 7/6 13/6	SM	SILTY SAND; medium dense, wet, fine to coarse grained, brown, with some interbedded clayey sand.		20	
			Bottom of boring B-1 at 26.5 feet			

Notes: ** After drilling groundwater depth was measured at 7.7 feet.*

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-2

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: ClearHeart Drilling

Date: January 29, 2021

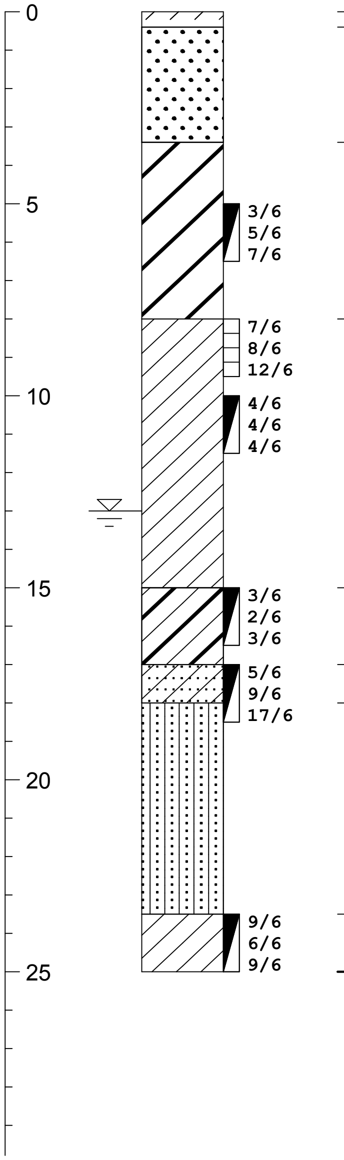
Drill Type: DR8K Track Rig

Auger Type: 4' Solid Flight Augers

Hammer Type: 140 Pound Cathead Hammer

Depth to Groundwater First Encountered During Drilling: 13 ft

ELEVATION/ DEPTH (feet) | **SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA**



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
PCC	Portland Cement Concrete Slab = 4.3 inches			
	Clayey Gravel= 36 inches (Layer of Visqueen at 8" depth)			
CH	FAT CLAY; stiff, moist, high plasticity, black Mottled, dark gray and black		12	
CL	LEAN CLAY WITH SAND; stiff, moist, high plasticity, light brown Medium Plasticity	DD=101.2 pcf LL=41 PI=26	20 8	16.7
CL-CH	LEAN TO FAT CLAY; medium stiff, wet, high plasticity, light brown		5	
SC	CLAYEY SAND; medium dense, wet, fine to medium grained, brown		26	
SM	SILTY SAND; medium dense, wet, fine to medium grained, brown, trace coarse sand and fine gravel	DD=101.2 pcf Sand=86.1% -#200=13.9%		19.8
CL	LEAN CLAY WITH SAND; stiff, wet, medium to high plasticity, light brown		15	
	Bottom of Boring B-2 at 25 feet			

Notes: ** After drilling groundwater depth was measured at 9 feet.*

Test Boring: B-3

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: ClearHeart Drilling

Drill Type: DR8K Track Rig

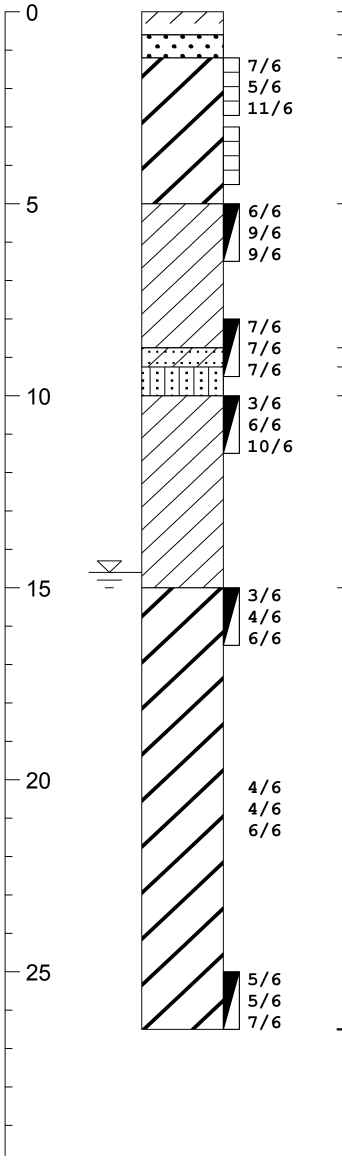
Auger Type: 4' Solid Flight Augers

Hammer Type: 140 Pound Cathead Hammer

Logged By: AH

Date: January 29, 2021

Depth to Groundwater First Encountered During Drilling: 14.6 feet*



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		PCC	Portland Cement Concrete Slab = 5.5 inches over Layer of vasqueen			
			Clayey Gravel = 8.5 inches			
	7/6 5/6 11/6	CH	FAT CLAY; moist, high plasticity, dark brown to black, with some fine gravel	EI=57	16	
				DD=95.5 pcf ø=18° C=840 psf LL=70 PI=55		20.1
5	6/6 9/6 9/6	CL	Stiff, no gravel		18	
			LEAN CLAY; very stiff, moist, medium to high plasticity, light grayish brown			
	7/6 7/6 7/6		LEAN CLAY WITH SAND; stiff, moist, medium plasticity, brown		14	
		SC	CLAYEY SAND; medium dense, moist, fine to medium grained, brown, trace of coarse sand and fine gravel			
10	3/6 6/6 10/6	SM	SILTY SAND; medium dense, moist, fine grained, brown		16	
		CL	LEAN CLAY WITH SAND; very stiff, moist, medium to high plasticity, grayish brown			
15	3/6 4/6 6/6	CH	FAT CLAY; stiff, wet, high plasticity, brown		10	
20	4/6 4/6 6/6		with trace fine gravel		10	
25	5/6 5/6 7/6				12	
			Bottom of Boring B-3 at 26.5 feet			

Notes: ** After drilling groundwater depth was measured at 9.0 feet.*

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

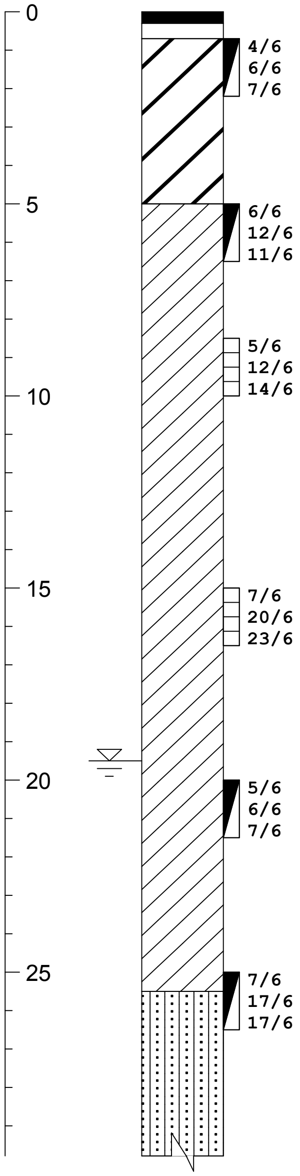
Date: February 24, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 19.5 ft



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	4/6 6/6 7/6	AC	ASPHALT CONCRETE = 3.1 inches		13	
		AB	Aggregate Base = 5.5 inches			
		CH	FAT CLAY; stiff, moist, high plasticity, black			
5	6/6 12/6 11/6	CL	SANDY LEAN CLAY; stiff, moist, low to medium		23	
	5/6 12/6 14/6		Increase in percent sand	DD=107.2 pcf LL=38 PI=21	26	21.0
15	7/6 20/6 23/6		Very stiff	DD=114.7 pcf	43	12.7
20	5/6 6/6 7/6		Stiff		13	
25	7/6 17/6 17/6		Hard		32	
		SM	SILTY SAND: dense, wet, fine to coarse, brown	Gravel=7.9% Sand=75.0% -#200=17.1%		

Notes:

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: EV

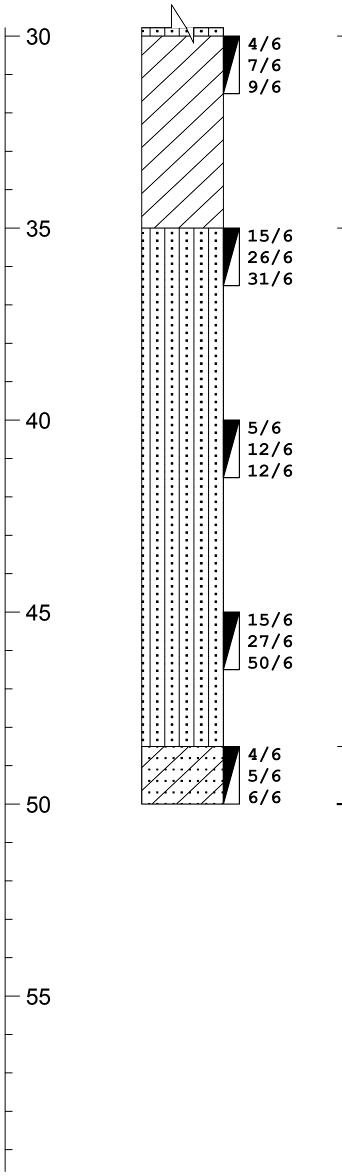
Date: February 24, 2021

Drill Type: CME 75

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

Depth to Groundwater First Encountered During Drilling: 19.5 ft

Hammer Type: 140 Pound Auto Trip Hammer



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	4/6 7/6 9/6	CL	SANDY LEAN CLAY; very stiff, low medium plasticity, wet, brown		16	
35	15/6 26/6 31/6	SM	SILTY SAND; very dense, wet, fine to coarse, brown		56	
40	5/6 12/6 12/6		Medium dense	Gravel=11.0% Sand=74.9% -#200=14.1% LL=NV PI=NP	24	
45	15/6 27/6 50/6		Very dense		77	
	4/6 5/6 6/6	SC	CLAYEY SAND; medium dense, wet, fine to medium, brown	Gravel=1.9% Sand=67.9% -#200=30.5% LL=37 PI=19	11	
50			Bottom of Boring at 50 feet BSG.			
55						

Notes:

MOORE TWINING
ASSOCIATES, INC.

Test Boring: B-5

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: EV

Date: February 23, 2021

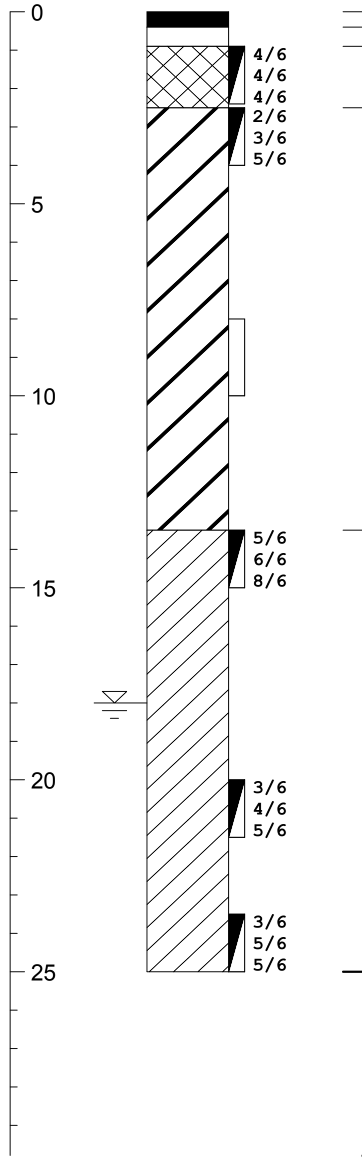
Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 18 feet

ELEVATION/ DEPTH (feet) | **SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA**



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
AC	ASPHALT CONCRETE = 4.5 inches			
AB	Aggregate Base = 6.5 inches		8	
FILL CH	FAT CLAY; medium stiff, moist, high plasticity,black		8	
	SANDY FAT CLAY; stiff, moist, low to medium plasticity, brown	Full Push		
CL	SANDY LEAN CLAY; stiff, moist, medium plasticity, brown	DD=114.7 pcf	12	12.7
			9	
CL	LEAN CLAY; stiff, wet, medium plasticity, brown		10	
	Bottom of Boring at 25 feet BSG.			

Notes: ** After drilling groundwater depth was measured at 9.5 feet.*

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

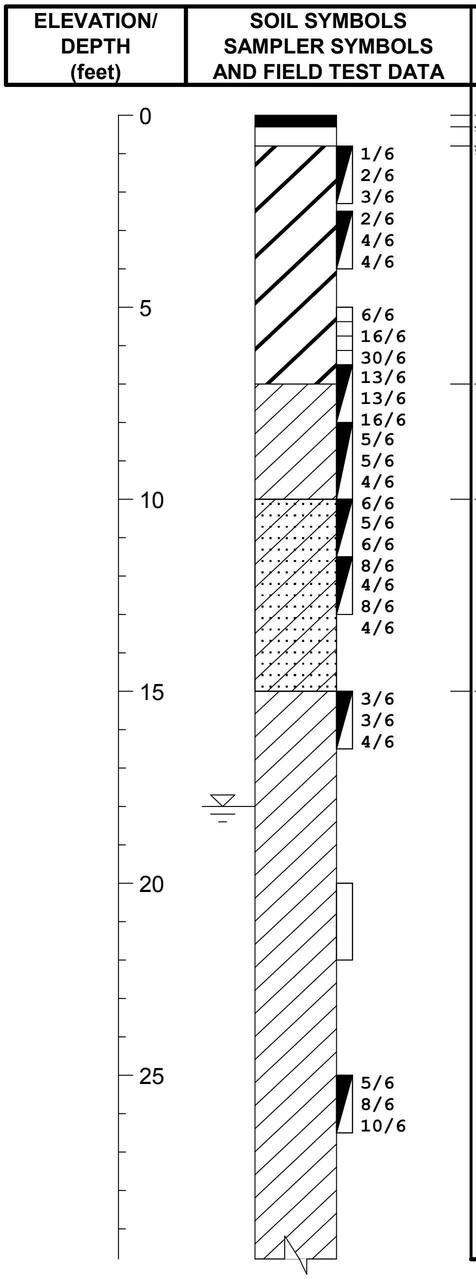
Date: February 22, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 18 feet



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
AC	ASPHALT CONCRETE = 3.6 inches			
AB	Aggregate Base = 5.5 inches		5	
CH	FAT CLAY; medium stiff, moist, high plasticity, black		8	28.5
	Very stiff	DD=107.1 pcf	46	17.4
			29	20.4
CL	SANDY LEAN CLAY;very stiff, moist, medium plasticity, brown Increase sand with depth	LL=29 PI=14	9	22.8
SC	CLAYEY SAND;medium dense, moist, fine to coarse, gray brown	Gravel=7.1% Sand=57.5% -#200=35.5% LL=39 PI=15	14	19.8
			15	
CL	LEAN CLAY; medium stiff, moist, medium plasticity, brown		7	26.2
	Wet			
		Full Push		
	Very stiff		18	26.1

Notes:

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

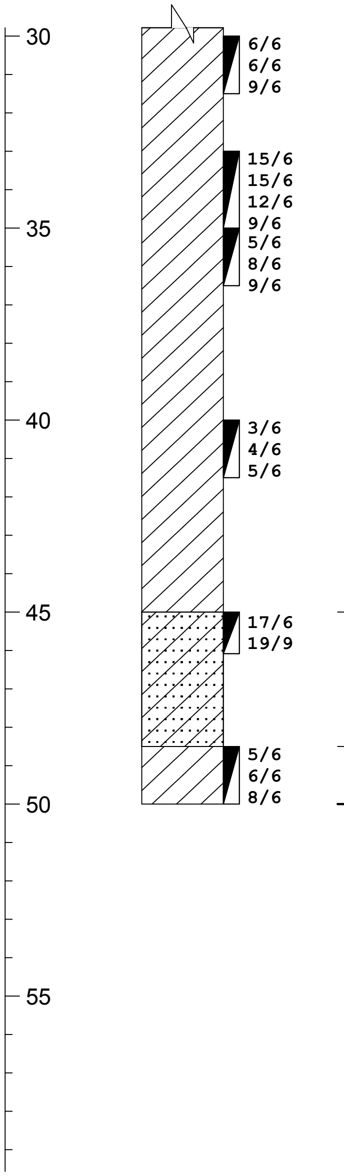
Date: February 22, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 18 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	6/6 6/6 9/6		Stiff		15	22.7
	15/6 15/6 12/6 9/6		Very stiff		27	27.3
35	5/6 8/6 9/6			Sand=19.5% -#200=80.5% LL=48 PI=34	17	
40	3/6 4/6 5/6		Medium stiff		9	
45	17/6 19/9	SC	CLAYEY SAND; dense, wet, fine to coarse, brown		36	
	5/6 6/6 8/6	CL	SANDY LEAN CLAY; stiff, wet, medium plasticity, brown		14	
50			Bottom of boring at 50 feet BSG.			
55						

Notes:

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

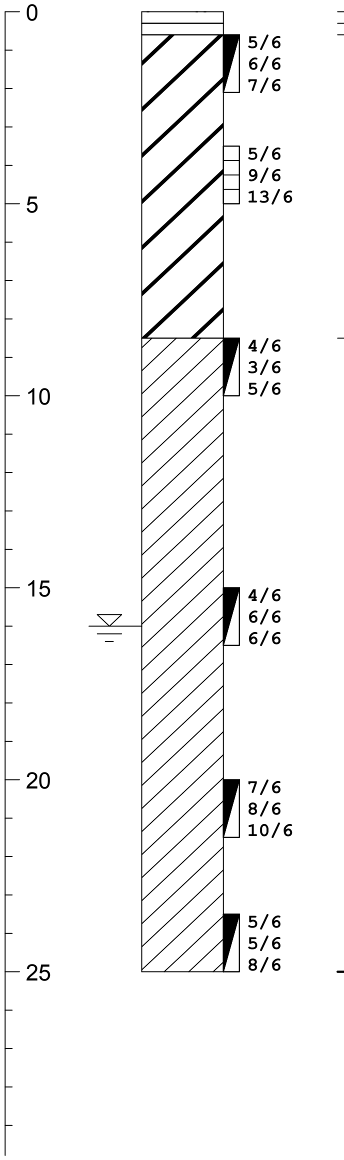
Date: February 25, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 16 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	5/6 6/6 7/6	PCC	Portland Cement Concrete = 2.8 inches		13	26.4
		AB	Aggregate Base = 2 inches			
	5/6 9/6 13/6	CH	FAT CLAY; stiff, moist, high plasticity,black	DD=91.2 pcf	22	24.5
	4/6 3/6 5/6	CL	LEAN CLAY; medium stiff, moist, medium plasticity, brown, some fine sand		8	27.3
15	4/6 6/6 6/6		Stiff, increase in percent sand		12	23.3
20	7/6 8/6 10/6		Very stiff, wet		18	24.8
	5/6 5/6 8/6				13	29.6
25			Bottom of Boring at 25 feet BSG.			

Notes:

Test Boring: B-8

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: EV

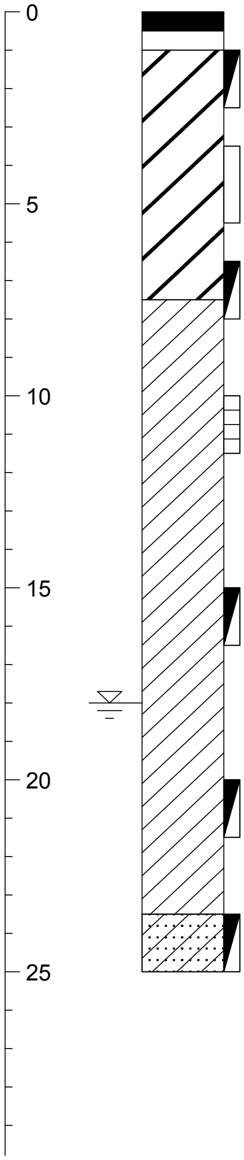
Date: February 23, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 18 feet



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC	ASPHALT CONCRETE = 5.8 inches			
	5/6 6/6 7/6	AB	Aggregate Base = 6 inches		13	
		CH	FAT CLAY; stiff, moist, high plasticity, black			
	7/6 7/6 6/6		Seam of Clayey Sand		13	24.5
		CL	SANDY LEAN CLAY; stiff, moist, medium plasticity, brown			
	8/6 11/6 15/6	CL	LEAN CLAY; very stiff, moist medium plasticity, brown	DD=99.4 pcf	26	24.9
	3/6 5/6 7/6		Stiff		12	23.3
	4/6 4/6 6/6				10	
	6/6 8/6 7/6	SC	CLAYEY SAND; medium dense, wet, fine to coarse, orange brown		15	
			Bottom of Boring at 25 feet BSG.			

Notes:

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-9

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

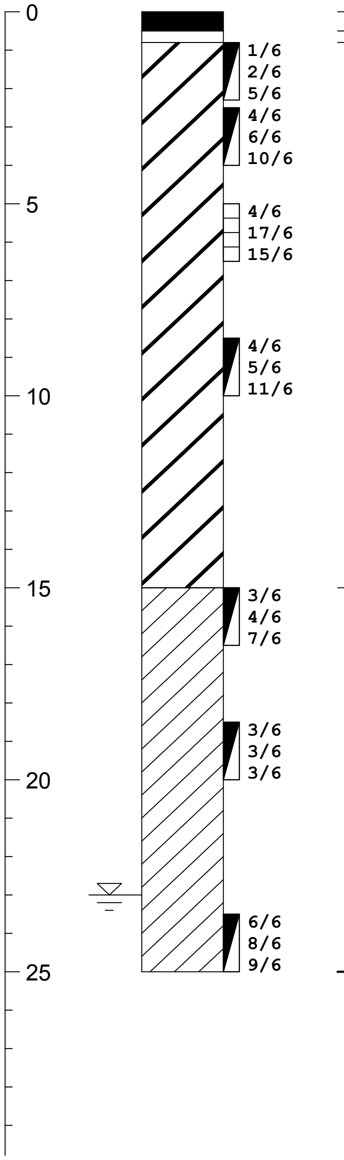
Date: February 23, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 23 feet



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 = 3.9 inches			
	1/6 2/6 5/6	AB	Aggregate Base = 6 inches		7	
	4/6 6/6 10/6	CH	FAT CLAY; medium stiff, moist, high plasticity, black		16	
			Very stiff			
5	4/6 17/6 15/6		Some fine to coarse sand	DD=86.1 pcf	32	31.1
	4/6 5/6 11/6				16	
10						
15	3/6 4/6 7/6	CL	SANDY LEAN CLAY; stiff, moist medium plasticity, brown		13	
	3/6 3/6 3/6		Medium stiff		6	
20						
	6/6 8/6 9/6	CL	LEAN CLAY with Sand; very stiff, wet, medium plasticity, brown	Sand=18.5% -#200=81.5% LL=39 PI=22	17	
25			Bottom of Boring at 25 feet BSG.			

Notes: ** After drilling groundwater depth was measured at 17.5 feet.*

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-10

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

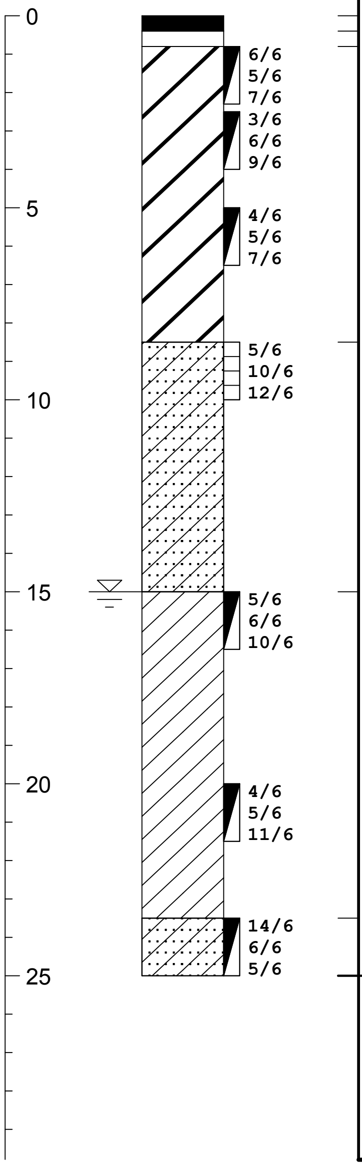
Date: February 25, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: 15 feet



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 = 3.3 inches			
	6/6 5/6 7/6	AB	Aggregate Base = 7 inches		12	29.3
	3/6 6/6 9/6	CH	FAT CLAY; stiff, moist, high plasticity, black		15	29.4
5	4/6 5/6 7/6				12	24.2
	5/6 10/6 12/6	SC	CLAYEY SAND; medium dense, moist, fine to coarse, brown	DD=107.7 pcf	22	19.4
15	5/6 6/6 10/6	CL	LEAN CLAY; very stiff, moist medium plasticity, brown		16	25.5
20	4/6 5/6 11/6		Some sand		16	33.3
	14/6 6/6 5/6	SC	CLAYEY SAND; medium dense, wet, fine to coarse, dark brown		11	16.1
25			Bottom of Boring at 25 feet BSG.			

Notes: ** After drilling groundwater depth was measured at 10 feet.*

Test Boring: B-11

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: EV

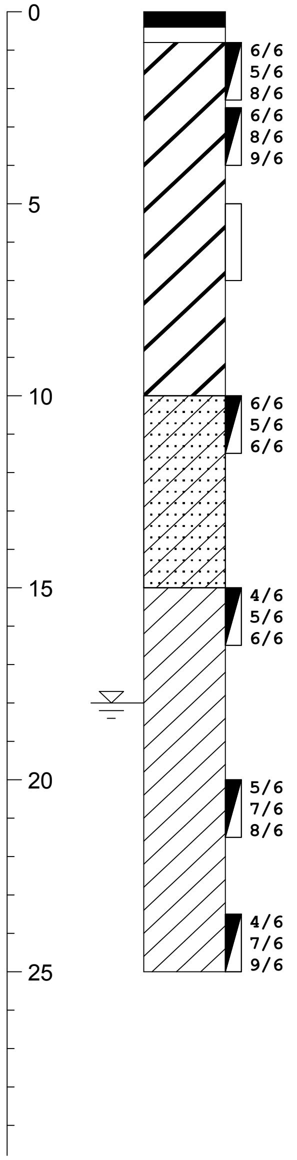
Date: February 25, 2021

Drill Type: CME 75

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

Depth to Groundwater First Encountered During Drilling: 18 feet

Hammer Type: 140 Pound Auto Trip Hammer

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC	ASPHALT CONCRETE = 3.8 inches			
		AB	Aggregate Base = 6.5 inches		13	29.6
		CH	FAT CLAY; stiff, moist, high plasticity, black Very stiff		17	28.3
		SC	CLAYEY SAND; medium dense, moist, fine to coarse, brown		11	18.5
		CL	SANDY LEAN CLAY; stiff, moist medium plasticity, brown		11	
			Wet		15	
		CL	LEAN CLAY; stiff, wet, medium to high plasticity, brown		11	
			Bottom of Boring at 25 feet BSG.			

Notes: ** After drilling groundwater depth was measured at 10 feet.*

Test Boring: B-12

Project: Proposed Retail Takeover

Project Number: D050S1.01

Logged By: AH

Drilled By: EV

Date: February 25, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: NE

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 = 5.1 inches			
	4/6 5/6 5/6	AB	Aggregate Base = 7 inches		10	
	4/6 8/6 9/6	CH	FAT CLAY; stiff, moist, high plasticity, black Very stiff		17	
5	9/6 12/6 18/6	CL	SANDY LEAN CLAY; very stiff, moist, medium, plasticity, brown		30	
	9/6 16/6 12/6	SC	CLAYEY SAND; medium dense, moist, fine to coarse, brown		28	
10			Bottom of Boring at 10 feet BSG.			
15						
20						
25						

Notes:

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-13

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

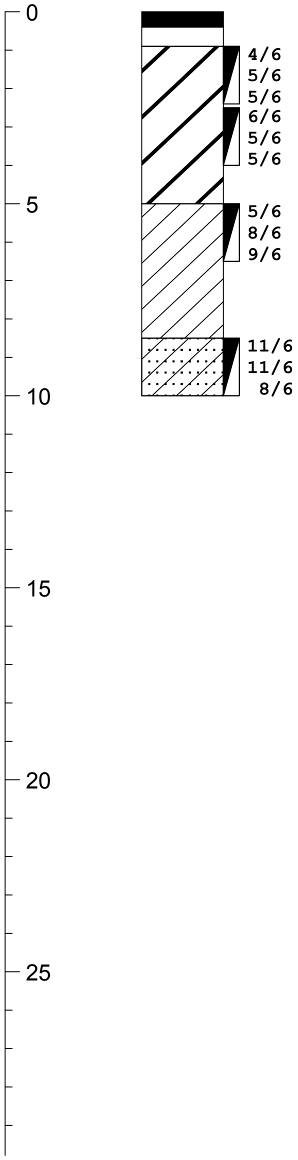
Date: February 25, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: NE



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
		AC	ASPHALT CONCRETE = 3.3 inches			
	4/6 5/6 5/6	AB	Aggregate Base = 7 inches		10	
	6/6 5/6 5/6	CH	FAT CLAY; stiff, moist, high plasticity, black		10	
	5/6 8/6 9/6	CL	SANDY LEAN CLAY; very stiff, moist, medium plasticity, brown		17	
	11/6 11/6 8/6	SC	CLAYEY SAND; medium dense, moist, fine to coarse, brown, trace fine gravel		19	
			Bottom of Boring at 10 feet BSG.			

Notes:

Test Boring: B-14

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Drill Type: CME 75

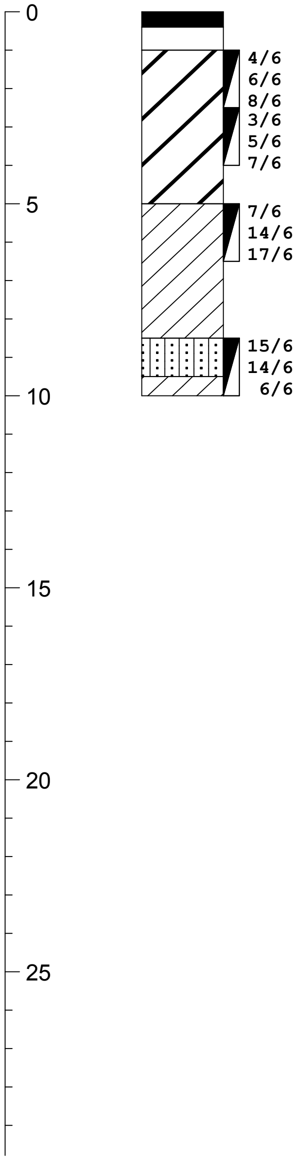
Auger Type: 6-5/8" O.D. Hollow Stem Augers

Hammer Type: 140 Pound Auto Trip Hammer

Logged By: AH

Date: February 25, 2021

Depth to Groundwater First Encountered During Drilling: NE



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 = 5.4 inches			
	4/6 6/6 8/6	AB	Aggregate Base = 7 inches		14	26.3
	3/6 5/6 7/6	CH	FAT CLAY; stiff, moist, high plasticity, black		12	25.7
5	7/6 14/6 17/6	CL	LEAN CLAY; very stiff, moist, medium plasticity, brown to gray		22.5	22.5
	15/6 14/6 6/6	SM	SILTY SAND; medium dense, moist, fine to coarse, gray		20	26.2
		CL	LEAN CLAY; very stiff, moist, medium, plasticity, brown			18.8
10			Bottom of Boring at 10 feet BSG.			

Notes:

Test Boring: B-15

Project: Proposed Retail Takeover

Project Number: D050S1.01

Drilled By: EV

Logged By: AH

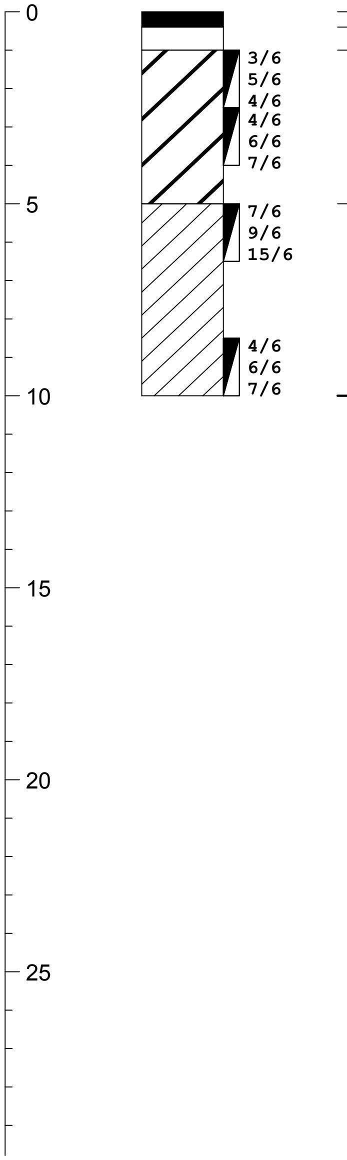
Date: February 25, 2021

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip Hammer

Depth to Groundwater First Encountered During Drilling: NE



USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
AC	ASPHALT CONCRETE = 4.3 inches			
AB	Aggregate Base = 6.5 inches	No sample recovery	9	
CH	FAT CLAY; stiff, moist, high plasticity, black		13	
CL	LEAN CLAY; very stiff, moist, medium plasticity, gray		24	
	Stiff, gray to brown		13	
	Bottom of Boring at 10 feet BSG.			

Notes:

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
▨	CL fraction
▨	Fat clay
▨	Lean clay
▥	Silty sand
▨	Lean to fat clay
▨	Clayey sand
■	Asphalt concrete

Symbol	Description
□	Blank
▩	Fill
△	Portland cement concrete
Misc. Symbols	
▽	Water table during drilling
⌇	Boring continues
Soil Samplers	
◩	Standard penetration test

Notes:

1. Exploratory borings were drilled on 1/29/21 using a limited access drill rig equipped with 4" outside diameter solid flight augers and 2/22/21 thru 2/25/21 using CME 75 drill rig equipped with 6-5/8" outside diameter hollow stem augers.

2. Groundwater was encountered in the borings, see logs.

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)
Sand = Percent retained on the No. 4 sieve (%)
-#200 = Percent passing the No. 200 sieve (%)
Sand = Percent passing the No. 4 sieve and retained on No. 200 sieve (%)
ø = Internal Angle of Friction (degrees)
pcf = Pounds per cubic foot
O.D. = Outside diameter
N/A = Not applicable
NV = Non Viscous

LL = Liquid Limit (%)
PI = Plasticity Index (%)
EI = Expansion Index
c = Cohesion (psf)
psf = Pounds per square foot
AMSL = Above mean sea level
N/E = Not encountered
NP = Non Plastic

KEY TO SYMBOLS

Symbol	Description
Soil Samplers	
▤	California Modified split barrel ring sampler
▯	Undisturbed thin wall Shelby tube

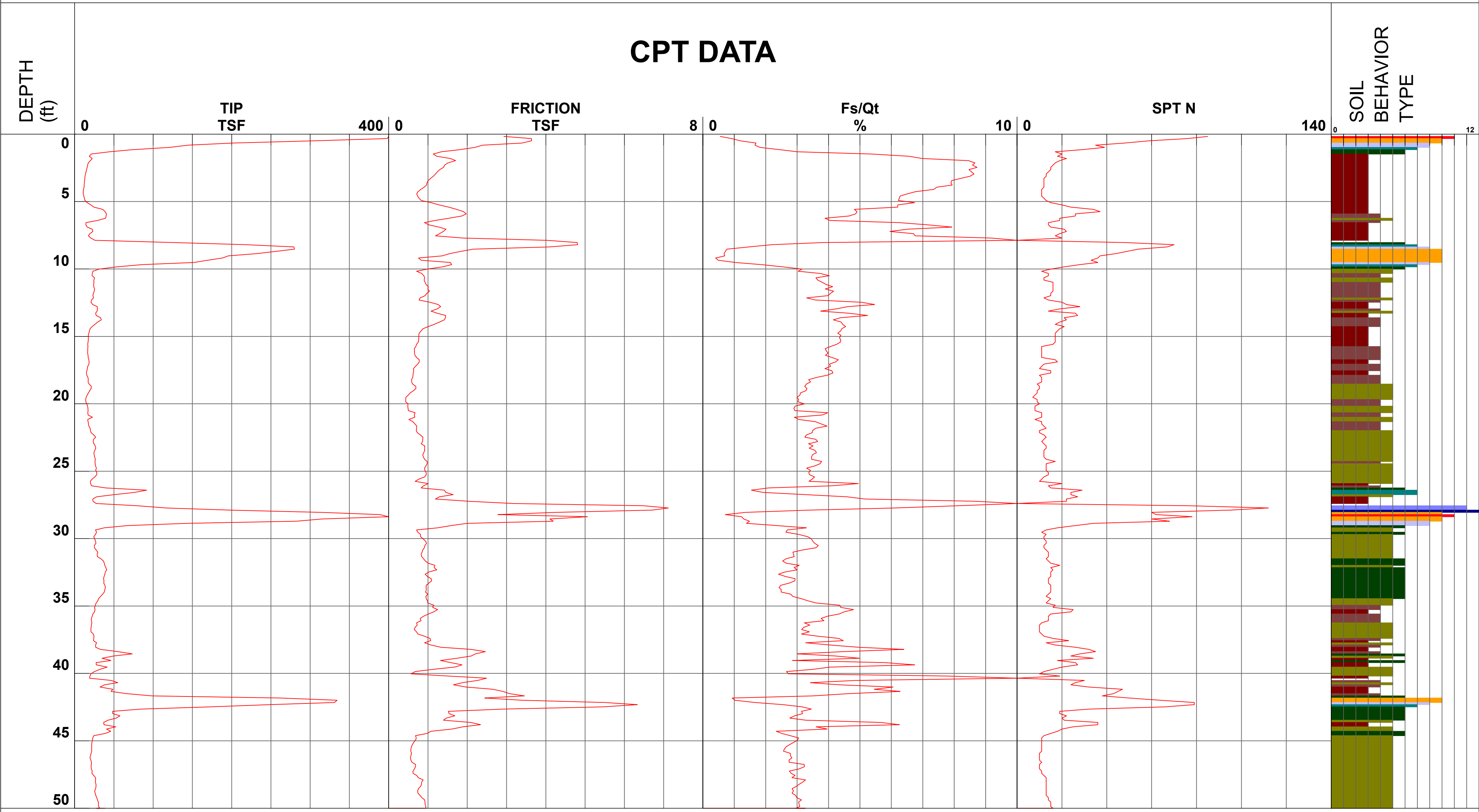
Middle Earth
GEO TESTING INC.

Moore Twining & Associates

Project 261 N McDowell Blvd
Job Number D050S1.01
Hole Number CPT-01
EST GW Depth During Test 10.00 ft
Operator BH-ZG
Cone Number DDG1587
Date and Time 1/29/2021 11:46:36 PM
Filename SDF(231).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

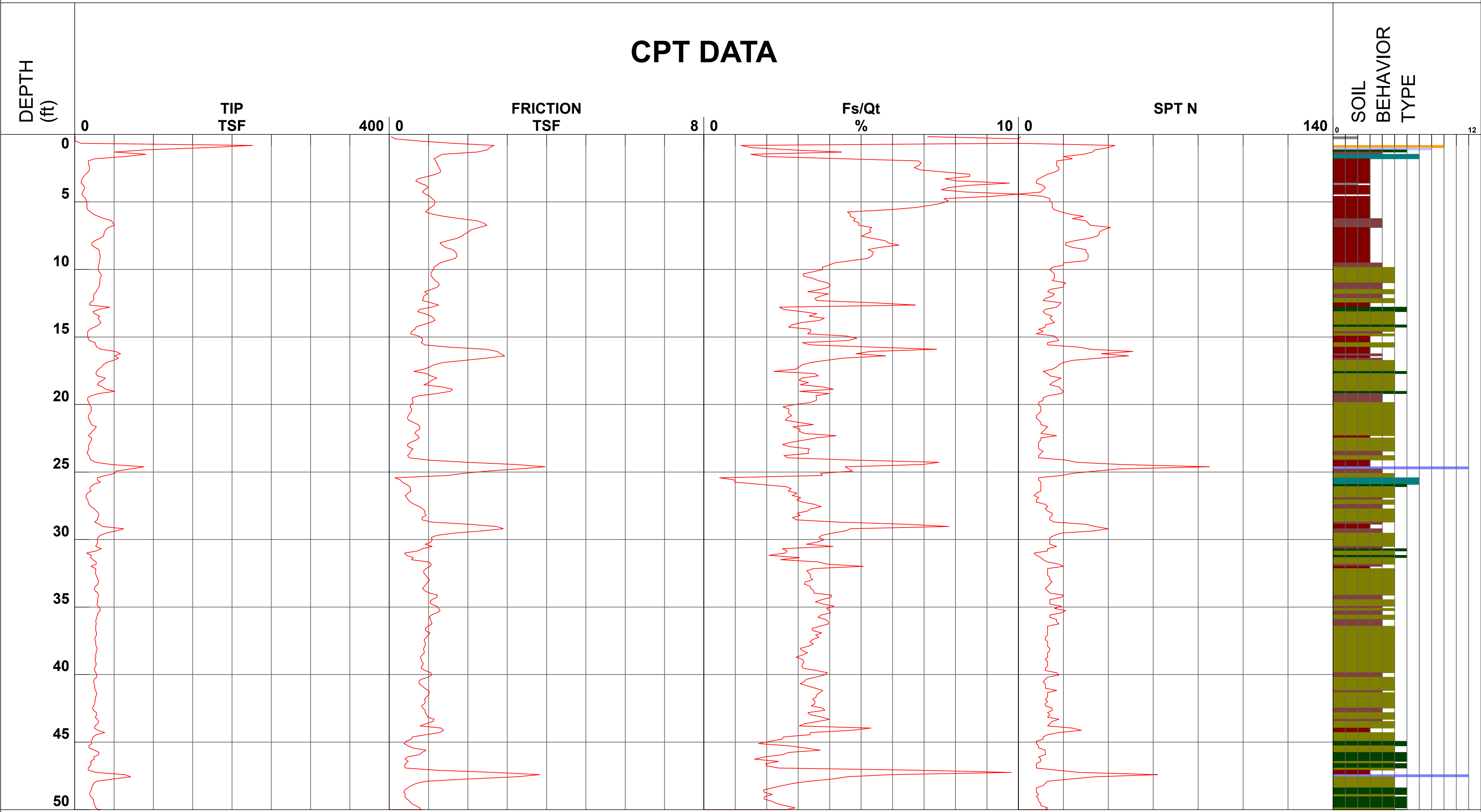
Moore Twining & Associates

Middle Earth GEO TESTING INC.

Project 261 N McDowell Blvd
Job Number D050S1.01
Hole Number CPT-02
EST GW Depth During Test 9.00 ft
Operator BH-ZG
Cone Number DDG1587
Date and Time 1/29/2021 9:59:36 AM
Filename SDF(230).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
Geo Testing Inc.

Moore Twining & Associates

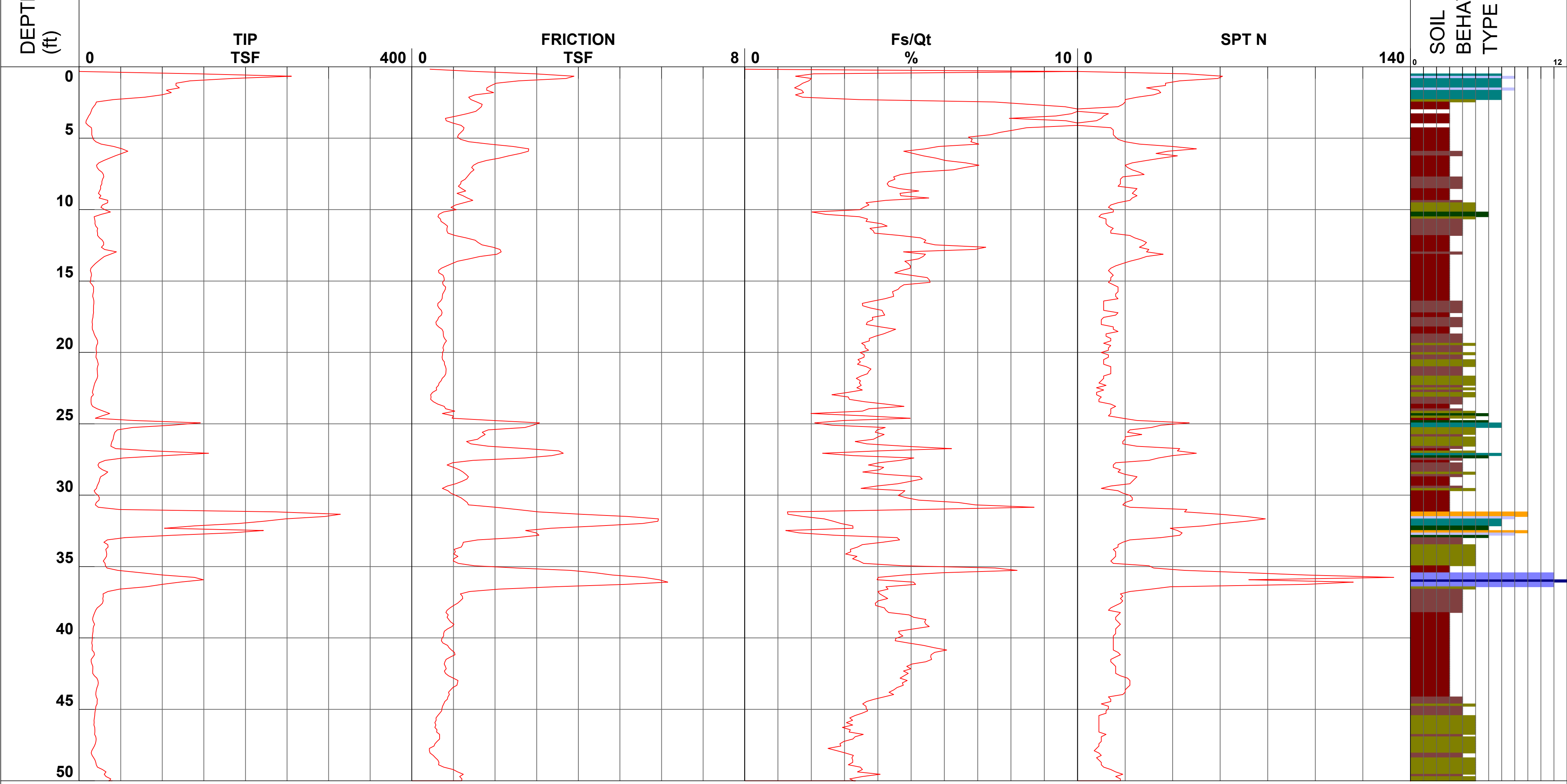
Project 261 N McDowell Blvd
Job Number D050S1.01
Hole Number CPT-03
EST GW Depth During Test 10.00 ft

Operator BH-ZG
Cone Number DDG1587
Date and Time 1/29/2021 1:03:46 PM

Filename SDF(232).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
Geo Testing Inc.

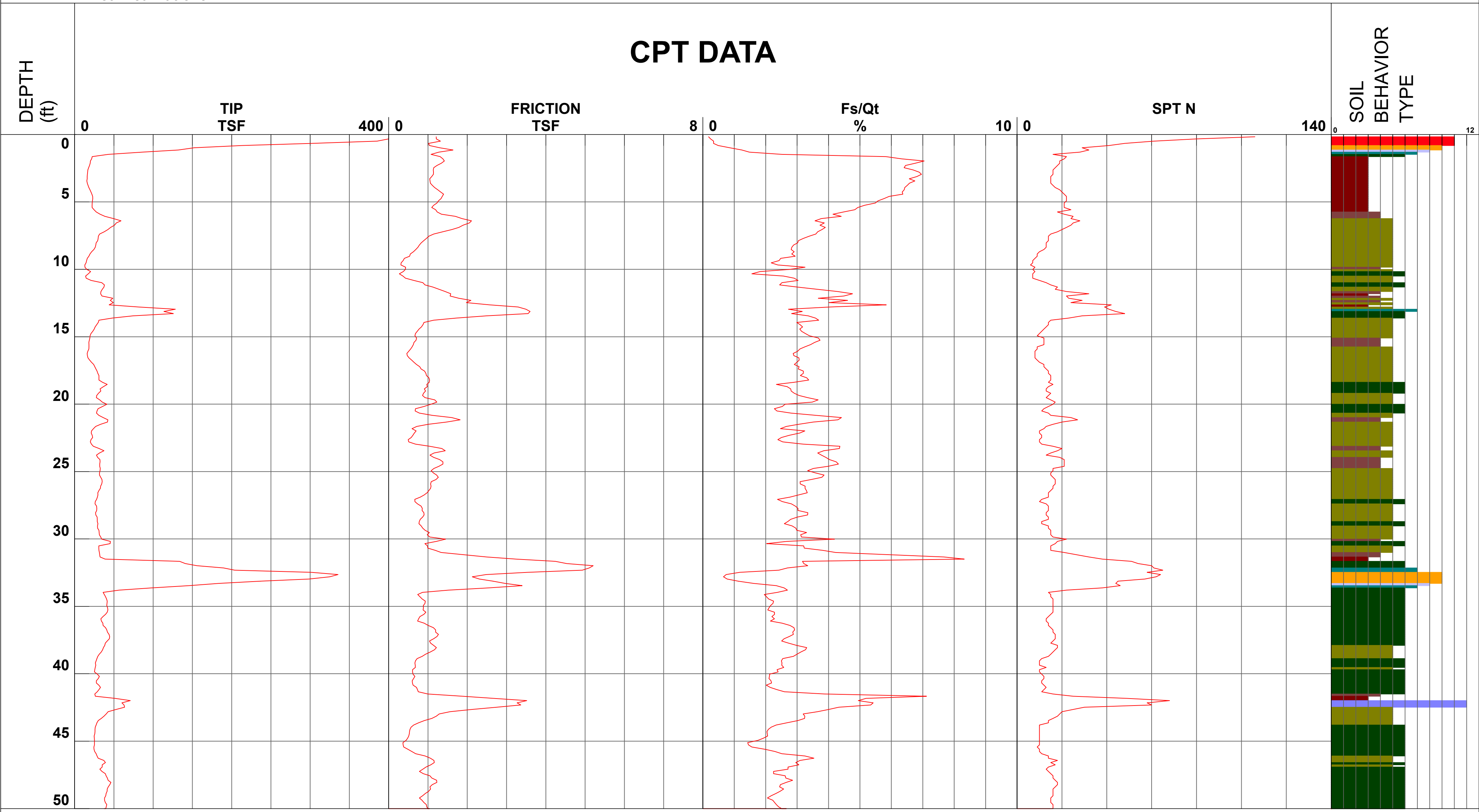
Moore Twining & Associates

Project 261 N McDowell Blvd
Job Number D050S1.01
Hole Number CPT-04
EST GW Depth During Test 9.00 ft

Operator BH-ZG
Cone Number DDG1587
Date and Time 1/29/2021 8:30:41 AM

Filename SDF(229).cpt
GPS
Maximum Depth 50.69 ft

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

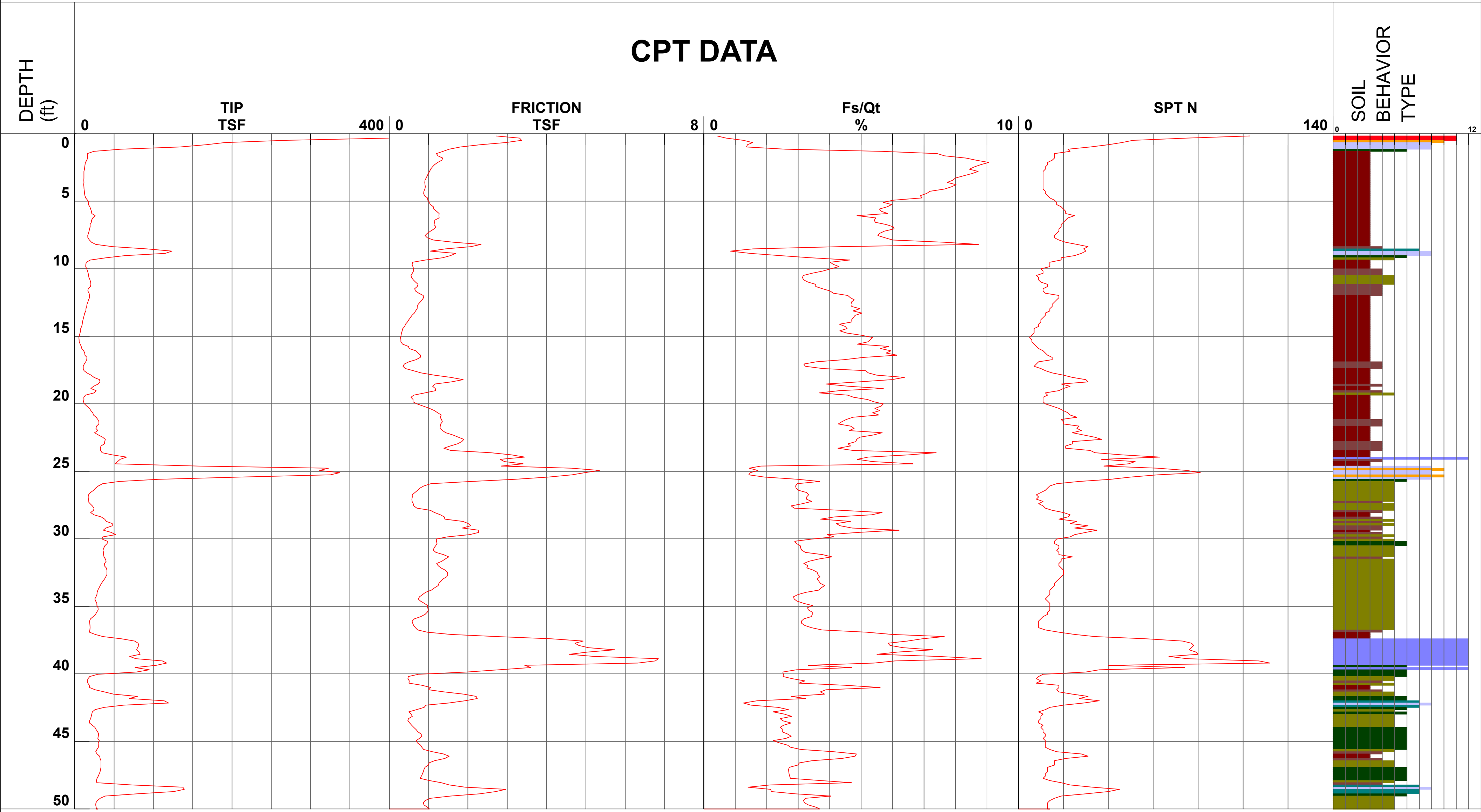
S*Soil behavior type and SPT based on data from UBC-1983

Moore Twining & Associates

Project: 261 N McDowell Blvd
Job Number: D050S1.01
Hole Number: CPT-05
EST GW Depth During Test: 7.00 ft
Operator: BH-ZG
Cone Number: DDG1587
Date and Time: 1/29/2021 2:16:55 PM
Filename: SDF(233).cpt
GPS:
Maximum Depth: 50.85 ft

Net Area Ratio .8

CPT DATA



- ⬜ 1 - sensitive fine grained
- ⬛ 2 - organic material
- 🟥 3 - clay
- 🟫 4 - silty clay to clay
- 🟩 5 - clayey silt to silty clay
- 🟩 6 - sandy silt to clayey silt
- 🟦 7 - silty sand to sandy silt
- 🟪 8 - sand to silty sand
- 🟧 9 - sand
- 🟥 10 - gravelly sand to sand
- 🟦 11 - very stiff fine grained (*)
- 🟦 12 - sand to clayey sand (*)

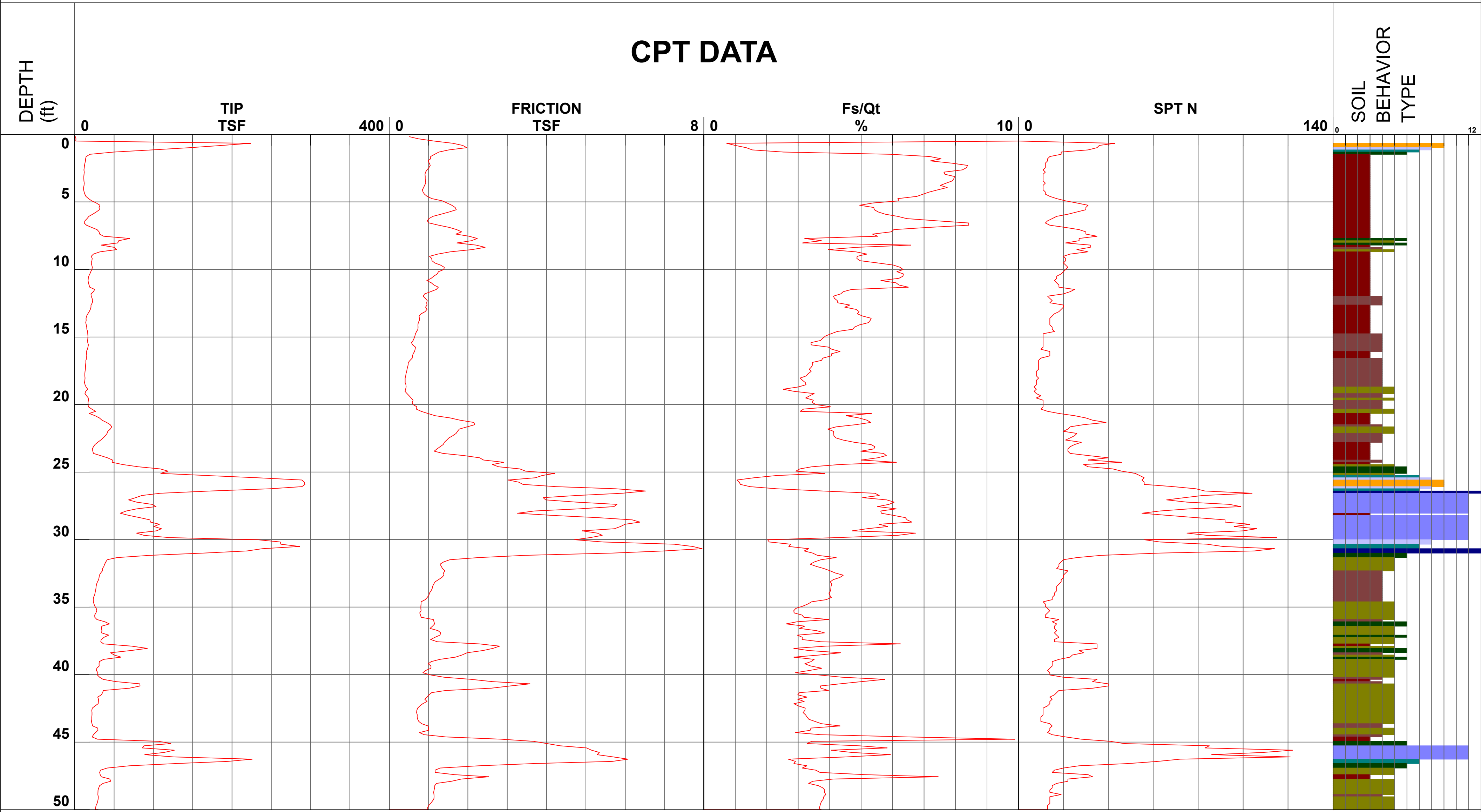
Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Moore Twining & Associates

Project 261 N McDowell Blvd | Operator BH-ZG | Filename SDF(235).cpt
Job Number D050S1.01 | Cone Number DDG1587 | GPS
Hole Number CPT-06a | Date and Time 1/29/2021 3:28:11 PM | Maximum Depth 50.69 ft
EST GW Depth During Test 7.00 ft

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

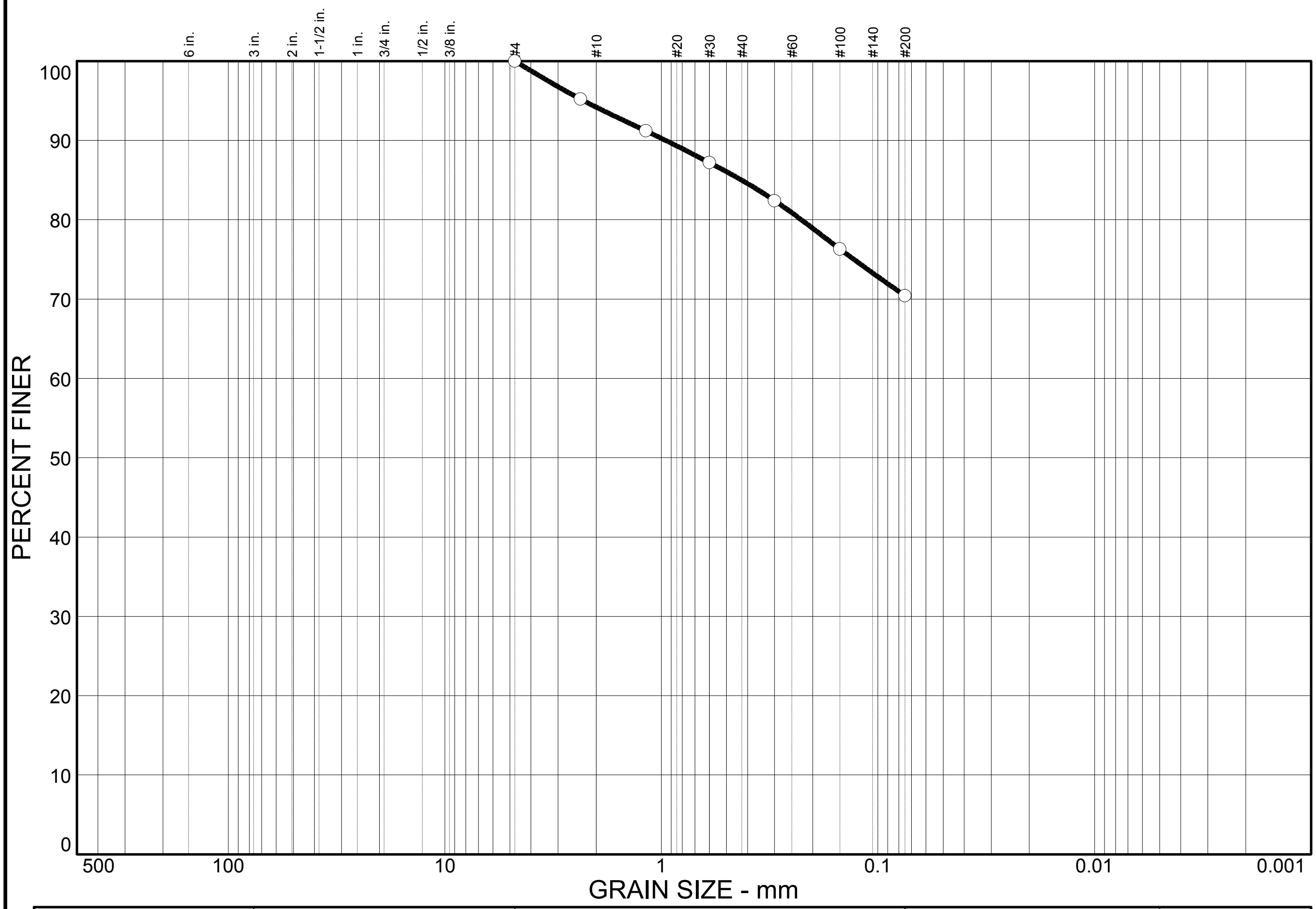
RESULTS OF LABORATORY TESTS

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., clay, silt, sand, and gravel.
Atterberg Limits (ASTM D4318)	Determines the moisture content at which the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state.
Consolidation (ASTM D2435)	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.
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.

These Included:	To Determine:
Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM G187)	The potential of the soil to corrode metal.
pH (ASTM D4972)	The acidity or alkalinity of subgrade material.

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	29.6	70.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	95.2		
#16	91.2		
#30	87.2		
#50	82.4		
#100	76.3		
#200	70.4		

* (no specification provided)

Material Description

Fat clay with sand

Atterberg Limits

PL= 18 LL= 62 PI= 44

Coefficients

D_{85}= 0.427 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CH AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 1/29/21
Location: **Elev./Depth:** 5.5-6.5'

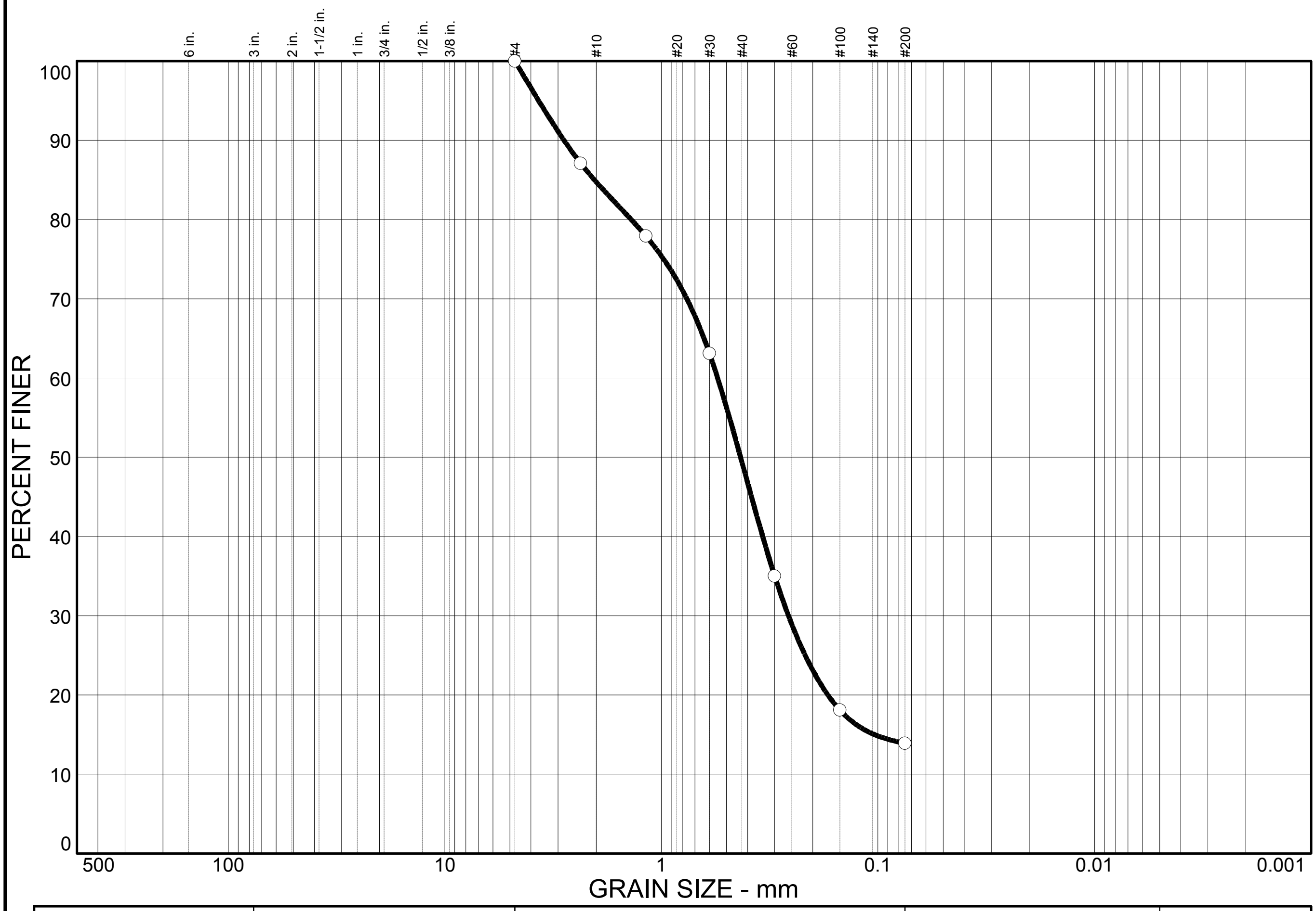
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	86.1	13.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	87.1		
#16	77.9		
#30	63.1		
#50	35.0		
#100	18.1		
#200	13.9		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.04 D_{60}= 0.550 D_{50}= 0.431

D_{30}= 0.259 D_{15}= 0.104 D_{10}=

C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample No.: B-2 **Source of Sample:** **Date:** 1/29/21

Location: **Elev./Depth:** 18-18.5'

Moore Twining Associates, Inc.

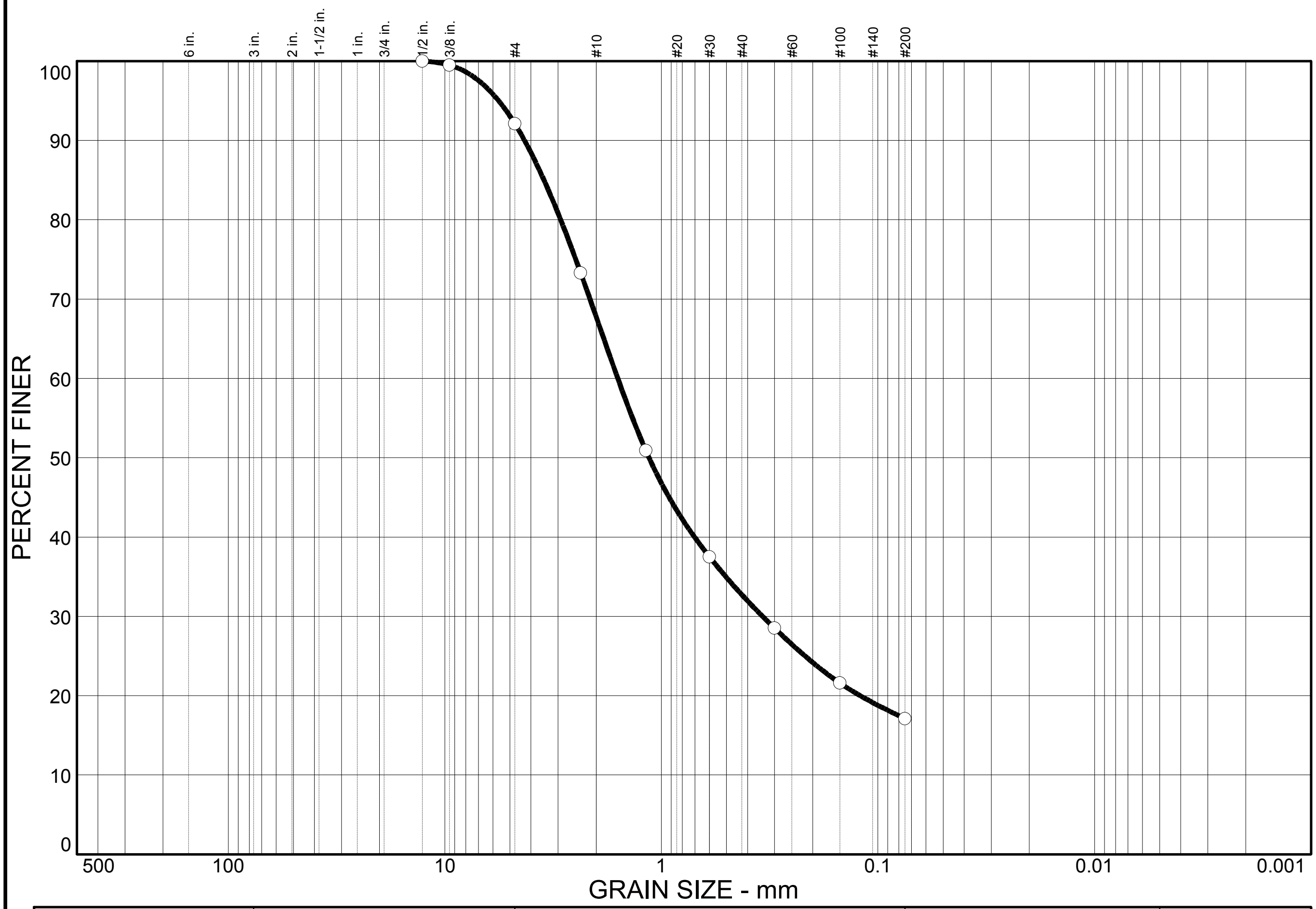
Fresno, CA

Client: Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.9	75.0	17.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.5		
#4	92.1		
#8	73.3		
#16	50.9		
#30	37.5		
#50	28.5		
#100	21.6		
#200	17.1		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.49 D_{60}= 1.59 D_{50}= 1.14
D_{30}= 0.341 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 25-26.5'

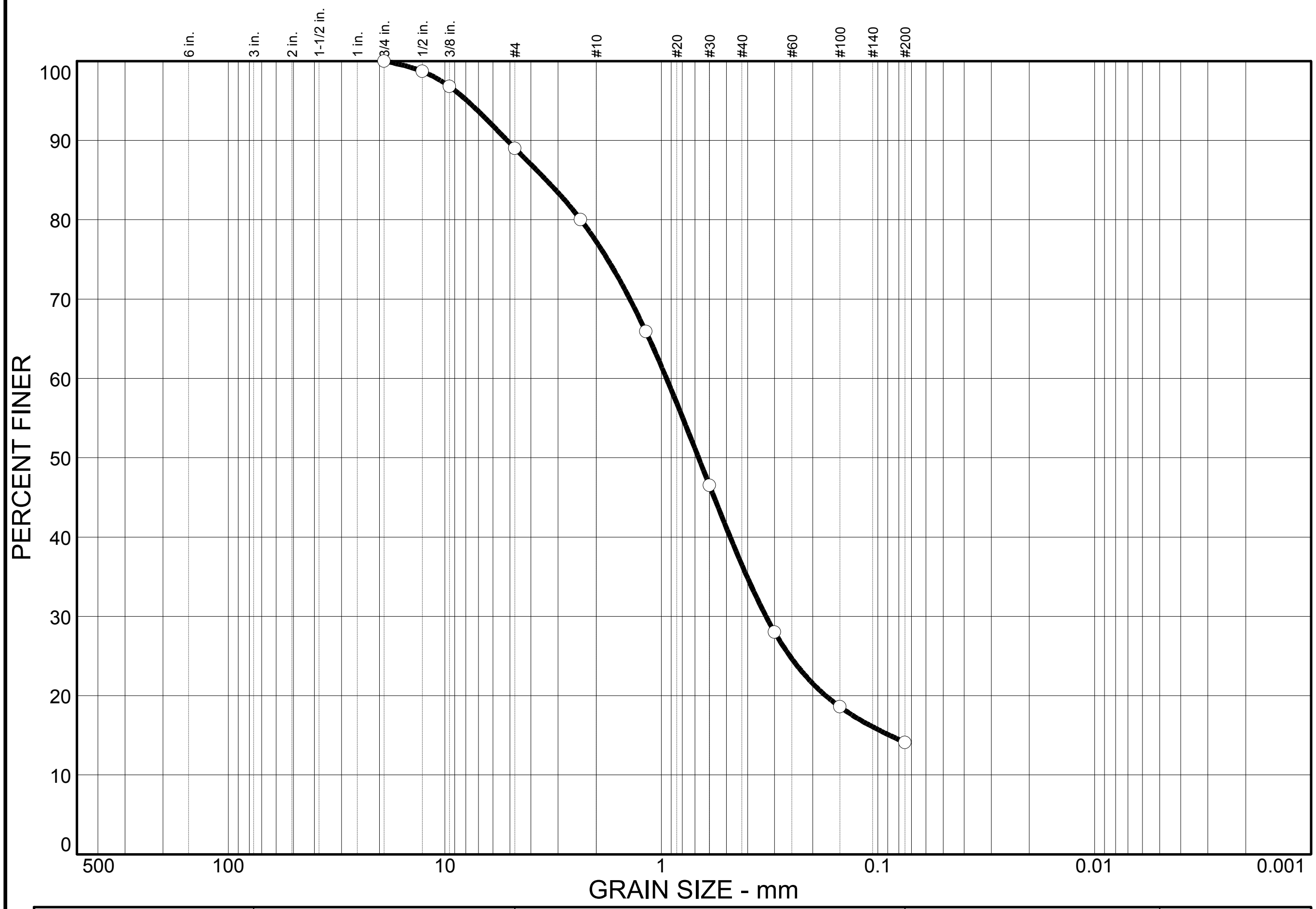
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	11.0	74.9	14.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.7		
3/8 in.	96.8		
#4	89.0		
#8	80.0		
#16	65.9		
#30	46.5		
#50	28.0		
#100	18.6		
#200	14.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 3.40 D_{60}= 0.948 D_{50}= 0.674
D_{30}= 0.329 D_{15}= 0.0883 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 40-41.5'

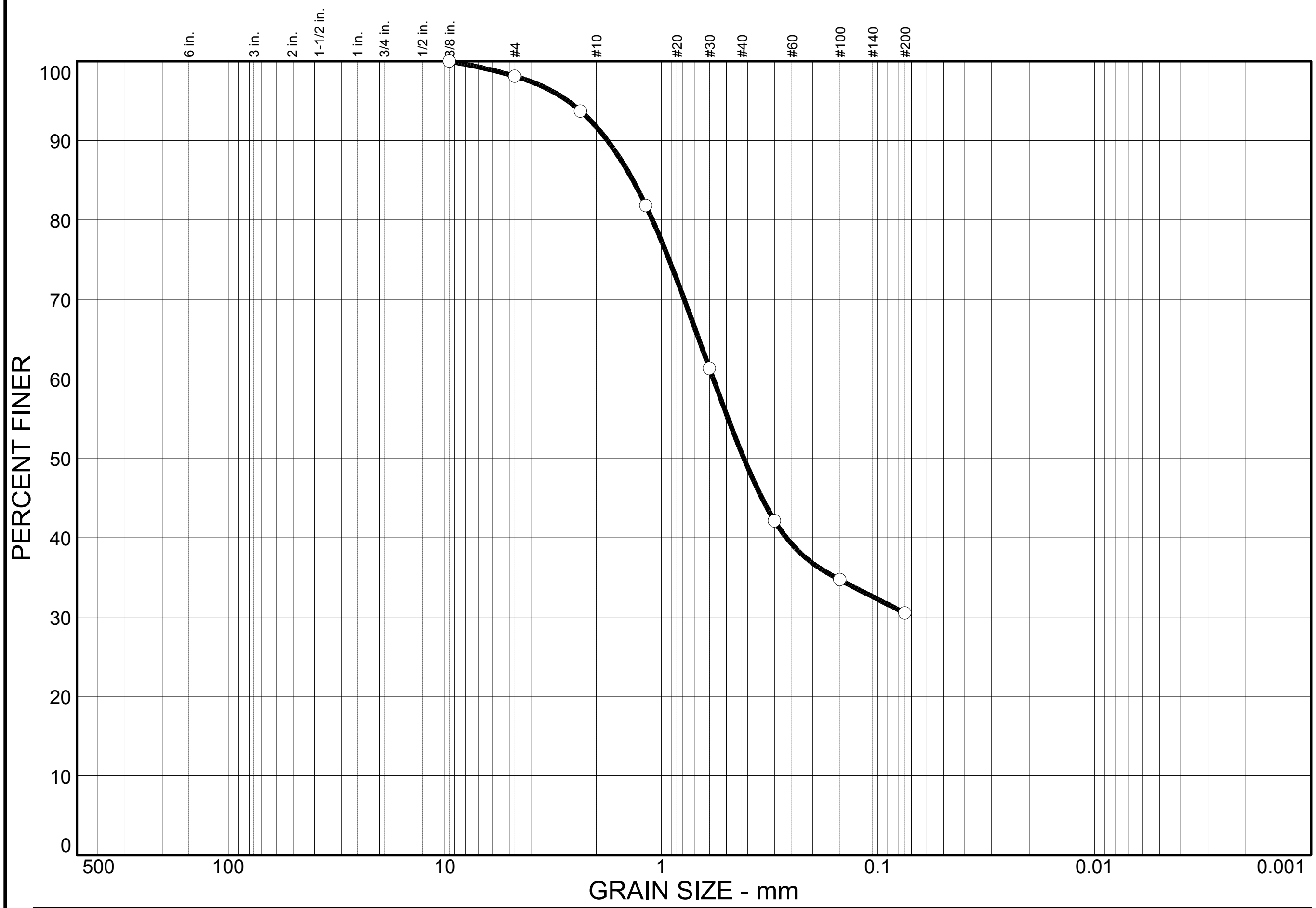
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.9	67.6	30.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.1		
#8	93.7		
#16	81.8		
#30	61.3		
#50	42.1		
#100	34.7		
#200	30.5		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 18 LL= 37 PI= 19

Coefficients

D_{85}= 1.35 D_{60}= 0.576 D_{50}= 0.416
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 48.5-50'

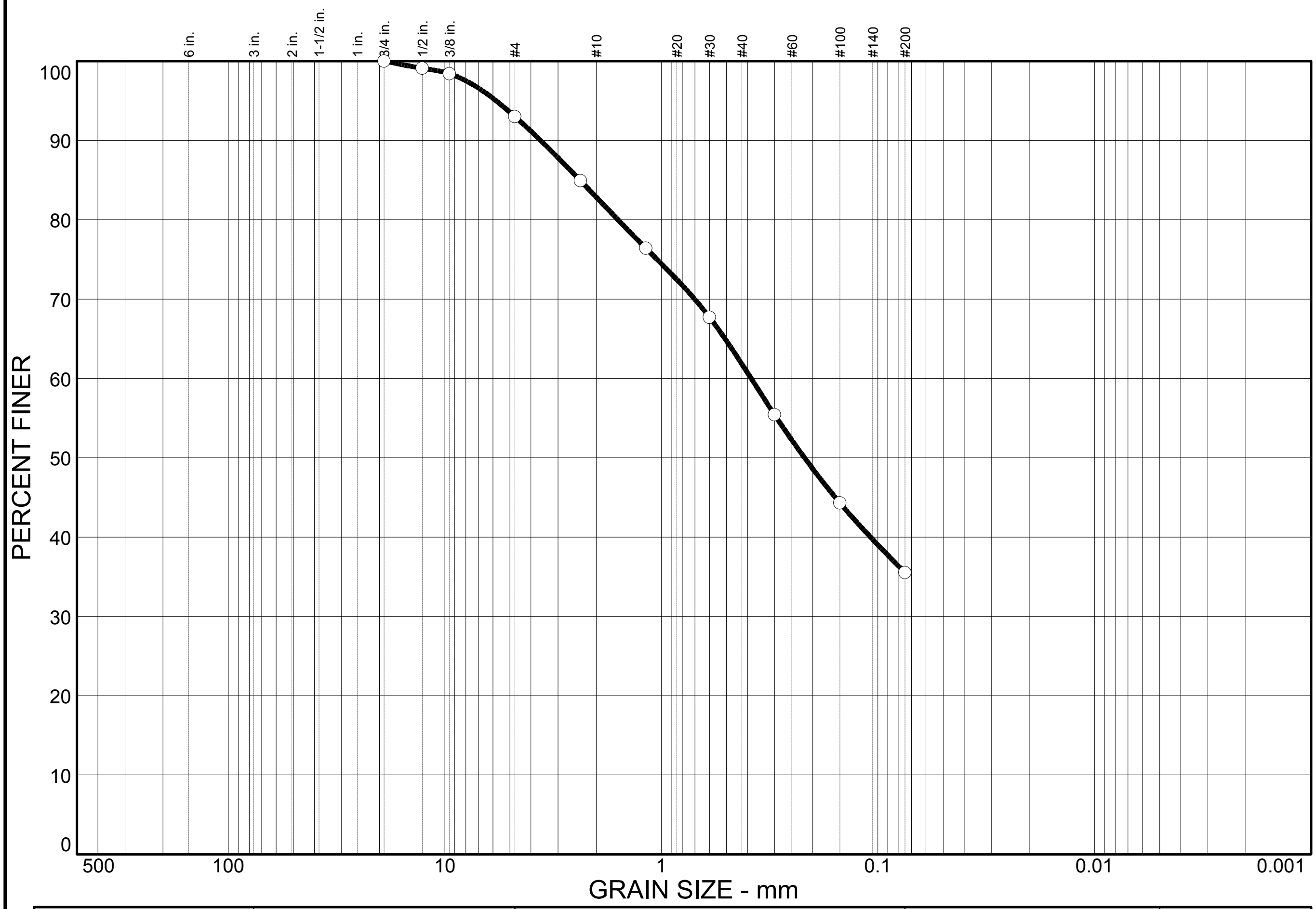
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.0	57.5	35.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	99.1		
3/8 in.	98.4		
#4	93.0		
#8	84.9		
#16	76.4		
#30	67.7		
#50	55.4		
#100	44.3		
#200	35.5		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 15 LL= 39 PI= 24

Coefficients

D_{85}= 2.38 D_{60}= 0.385 D_{50}= 0.219
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 10-11.5'

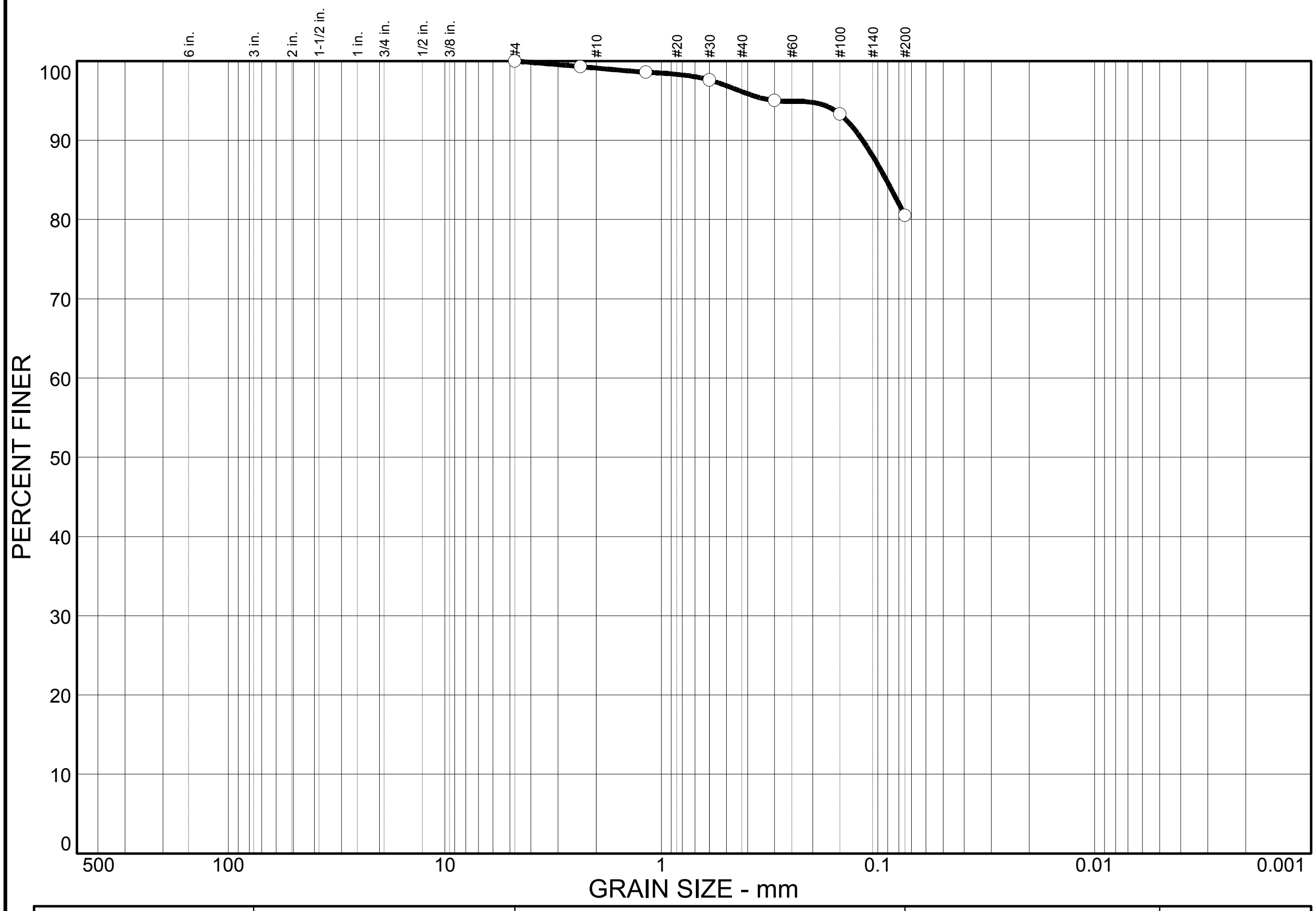
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	19.5	80.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.3		
#16	98.6		
#30	97.6		
#50	95.0		
#100	93.3		
#200	80.5		

* (no specification provided)

Material Description

Lean clay with sand

Atterberg Limits

PL= 14 LL= 48 PI= 34

Coefficients

D_{85}= 0.0916 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 35-36.5'

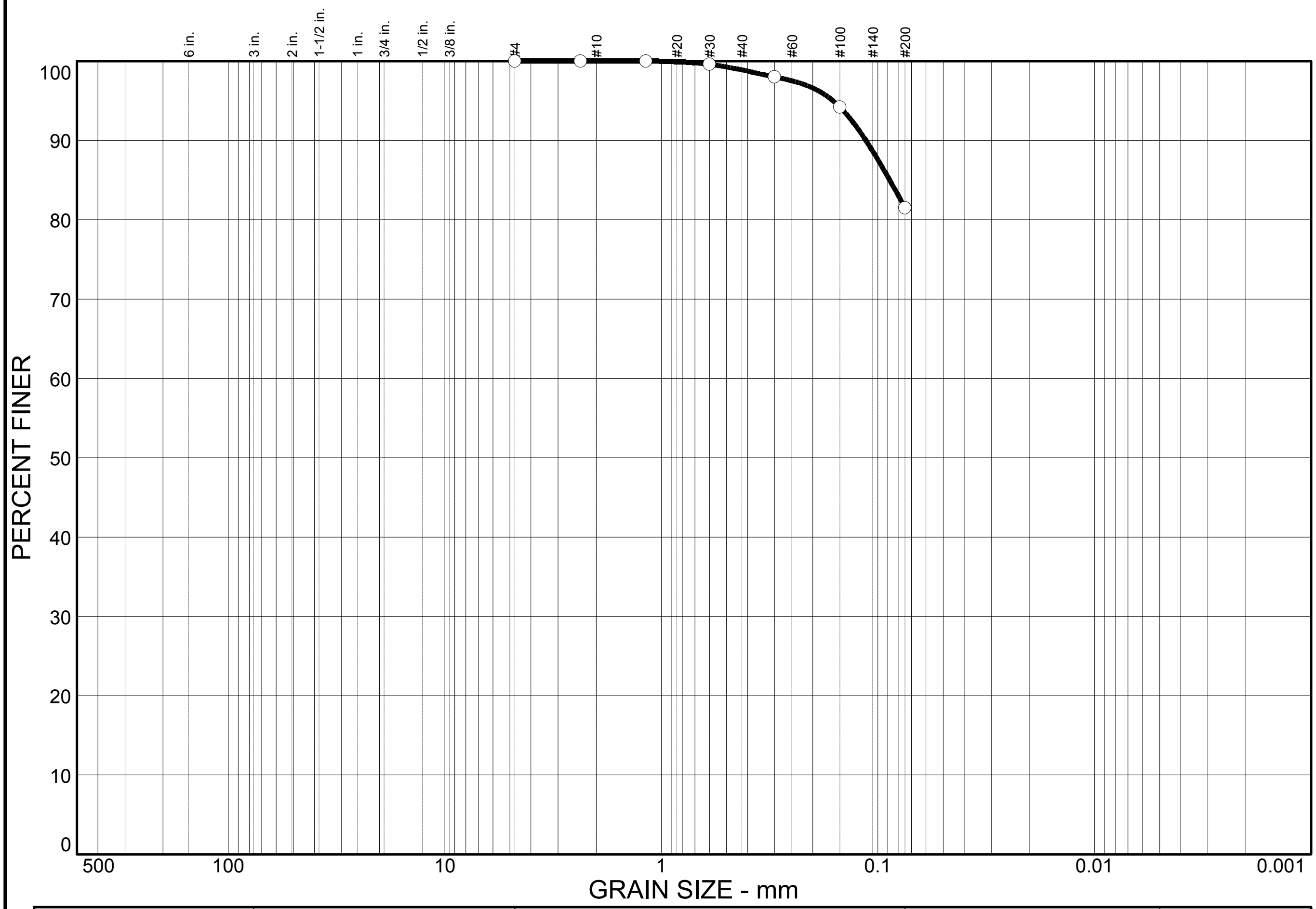
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	18.5	81.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	100.0		
#30	99.6		
#50	98.0		
#100	94.2		
#200	81.5		

* (no specification provided)

Material Description

Lean clay with sand

Atterberg Limits

PL= 17 LL= 39 PI= 22

Coefficients

D_{85}= 0.0883 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample No.: B-9 **Source of Sample:** **Date:** 2/22/21
Location: **Elev./Depth:** 23.5-25'

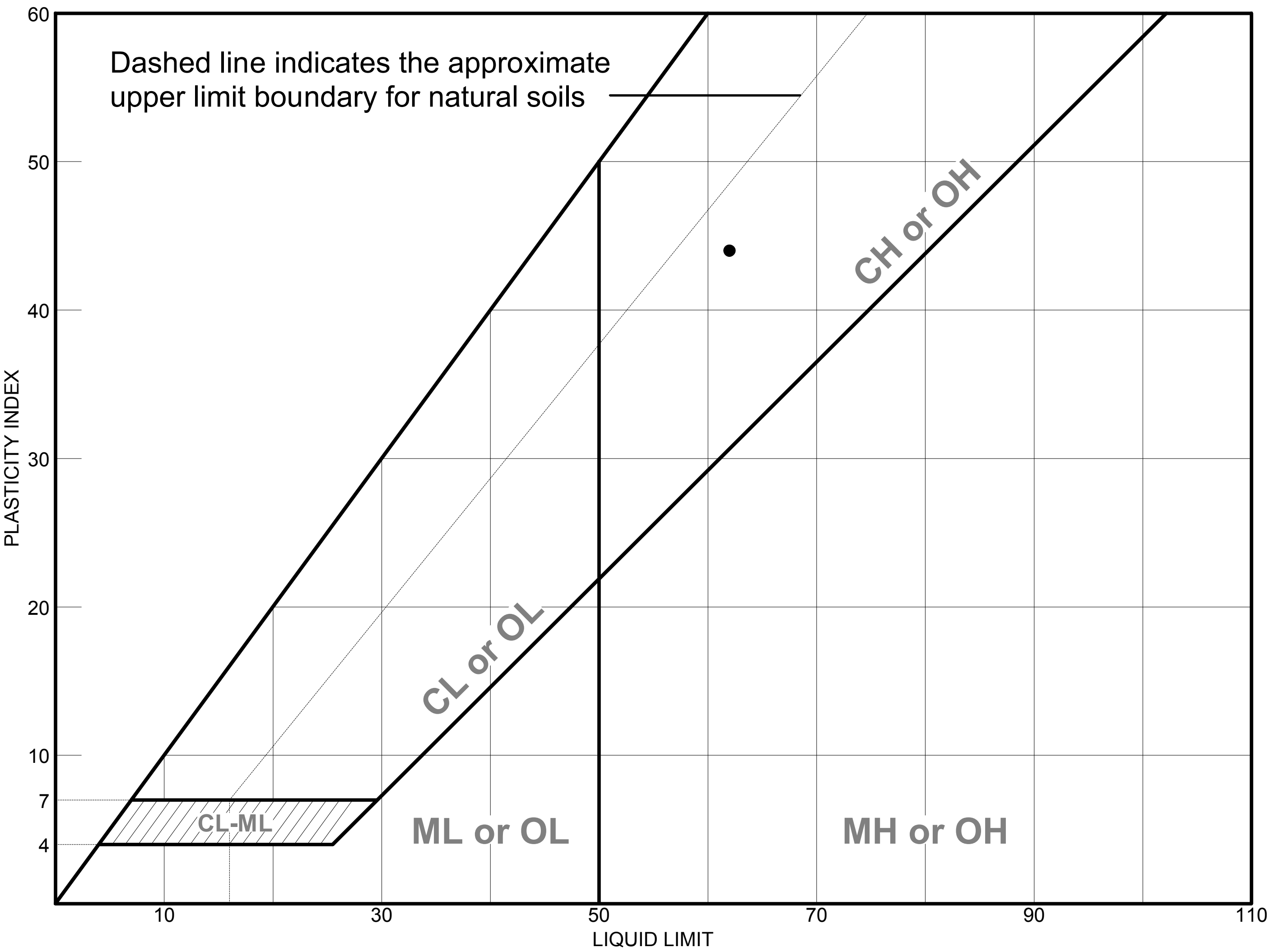
Moore Twining Associates, Inc.

Fresno, CA

Client: Home Depot USA, Inc.
Project: Proposed Retail Takeover Petaluma

Project No: D050S1.01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Fat clay with sand	62	18	44	85.0	70.4	CH

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 5.5-6.5'

Remarks:

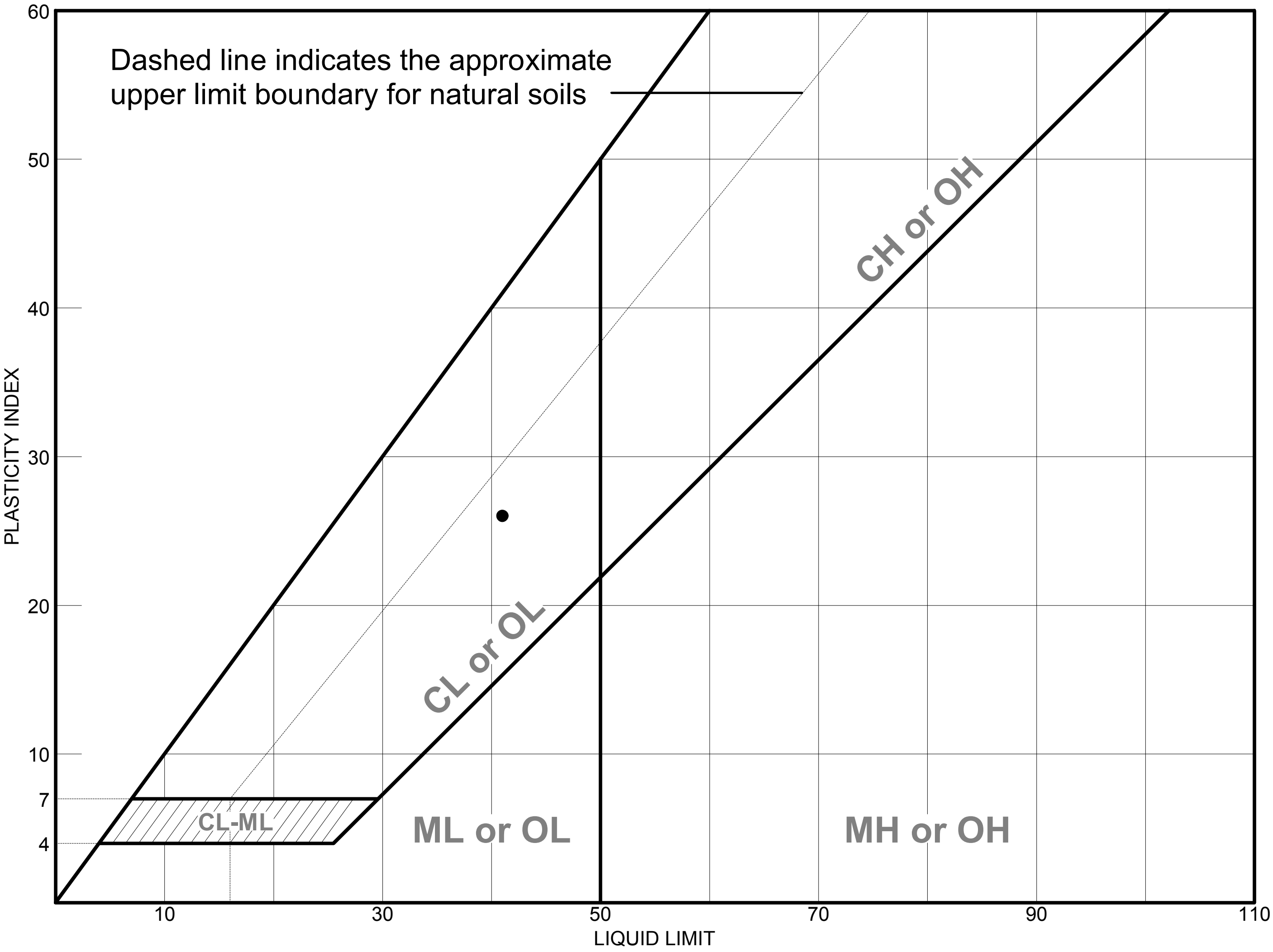
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		41	15	26			

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 8-9.5'

Remarks:

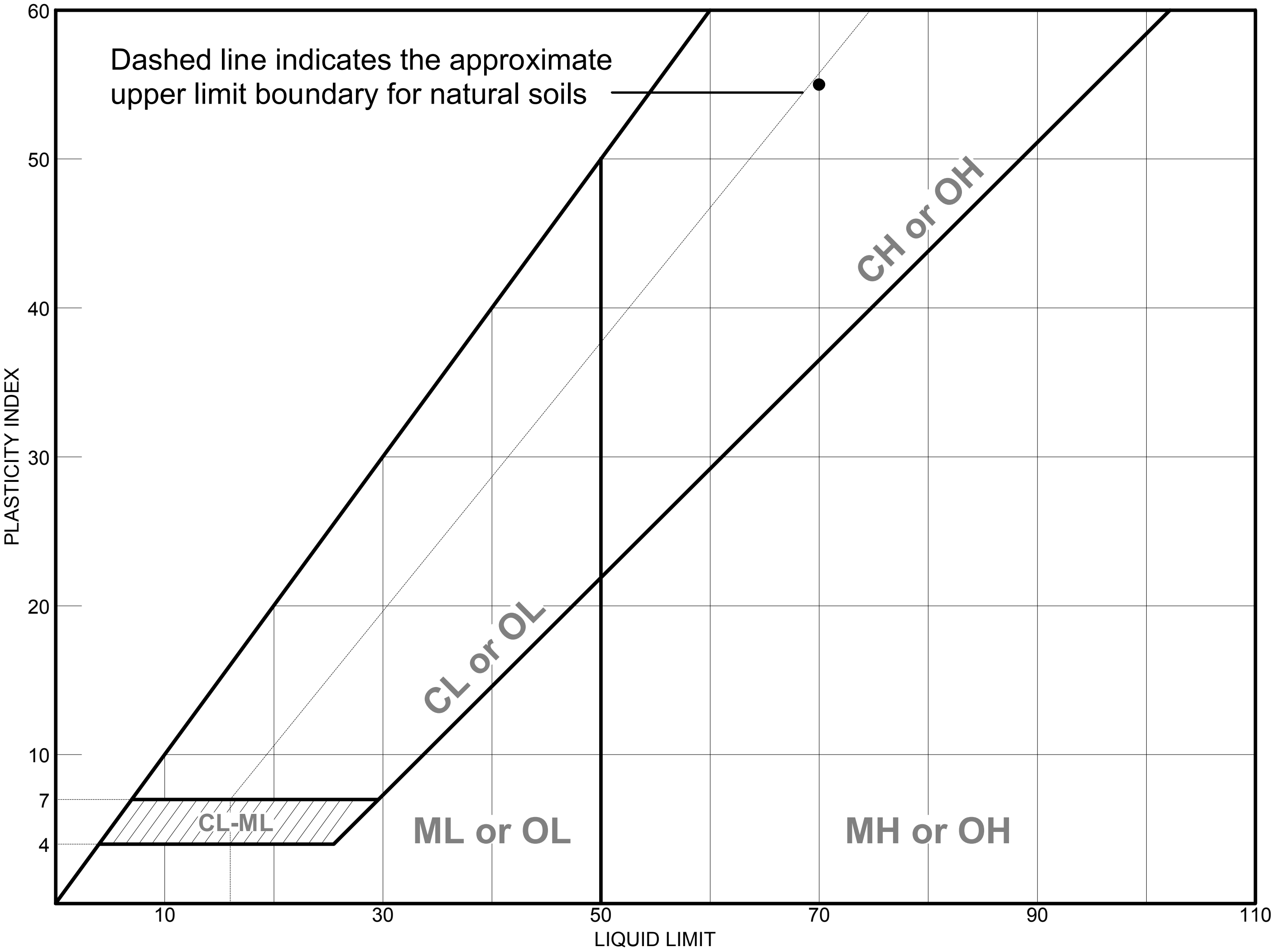
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		70	15	55			

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-3 **Elev./Depth:** 3-4.5'

Remarks:

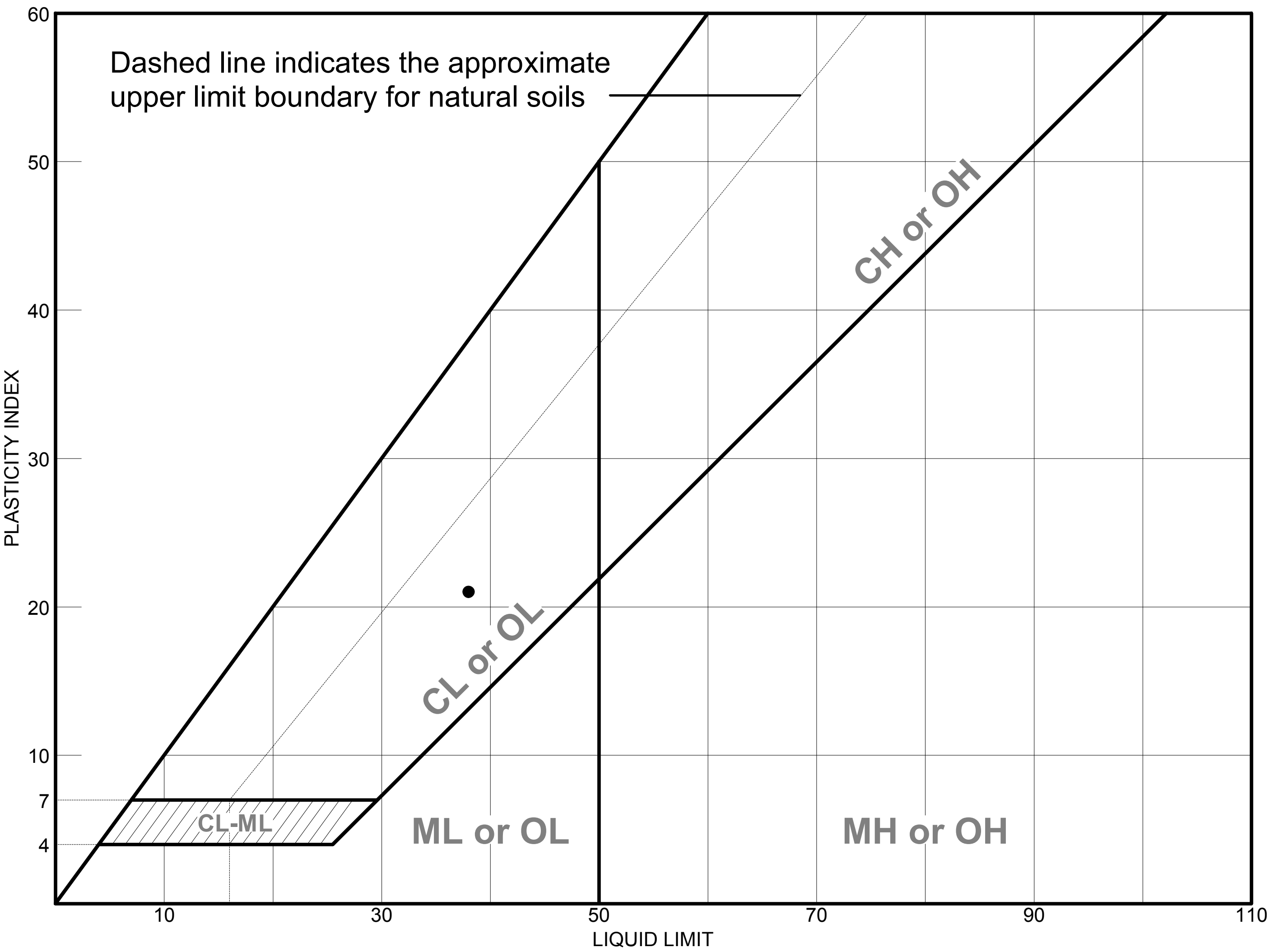
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		38	17	21			

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 8.5-10'

Remarks:

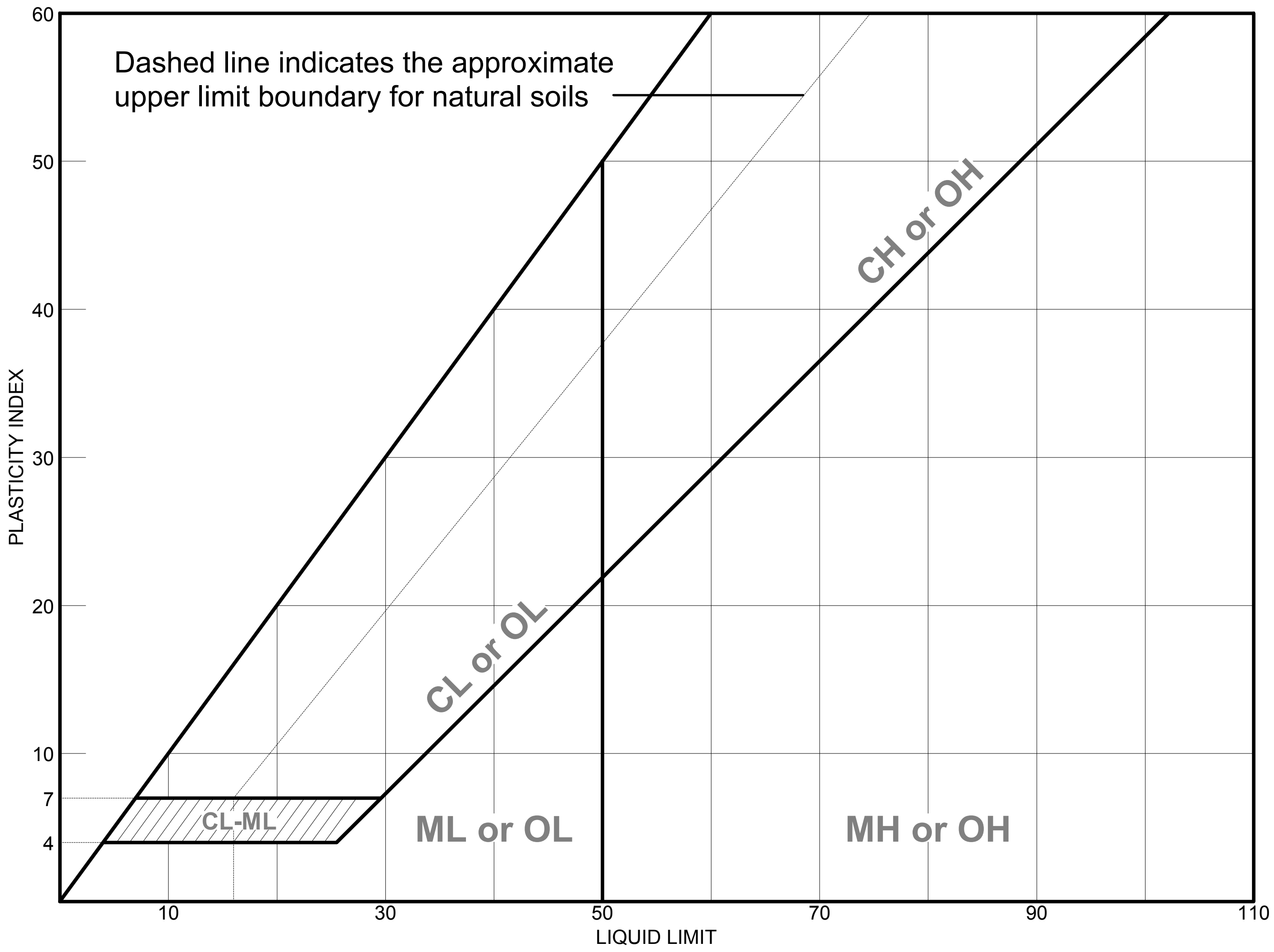
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty sand	NV	NP	NP	36.5	14.1	SM

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 40-41.5'

Remarks:

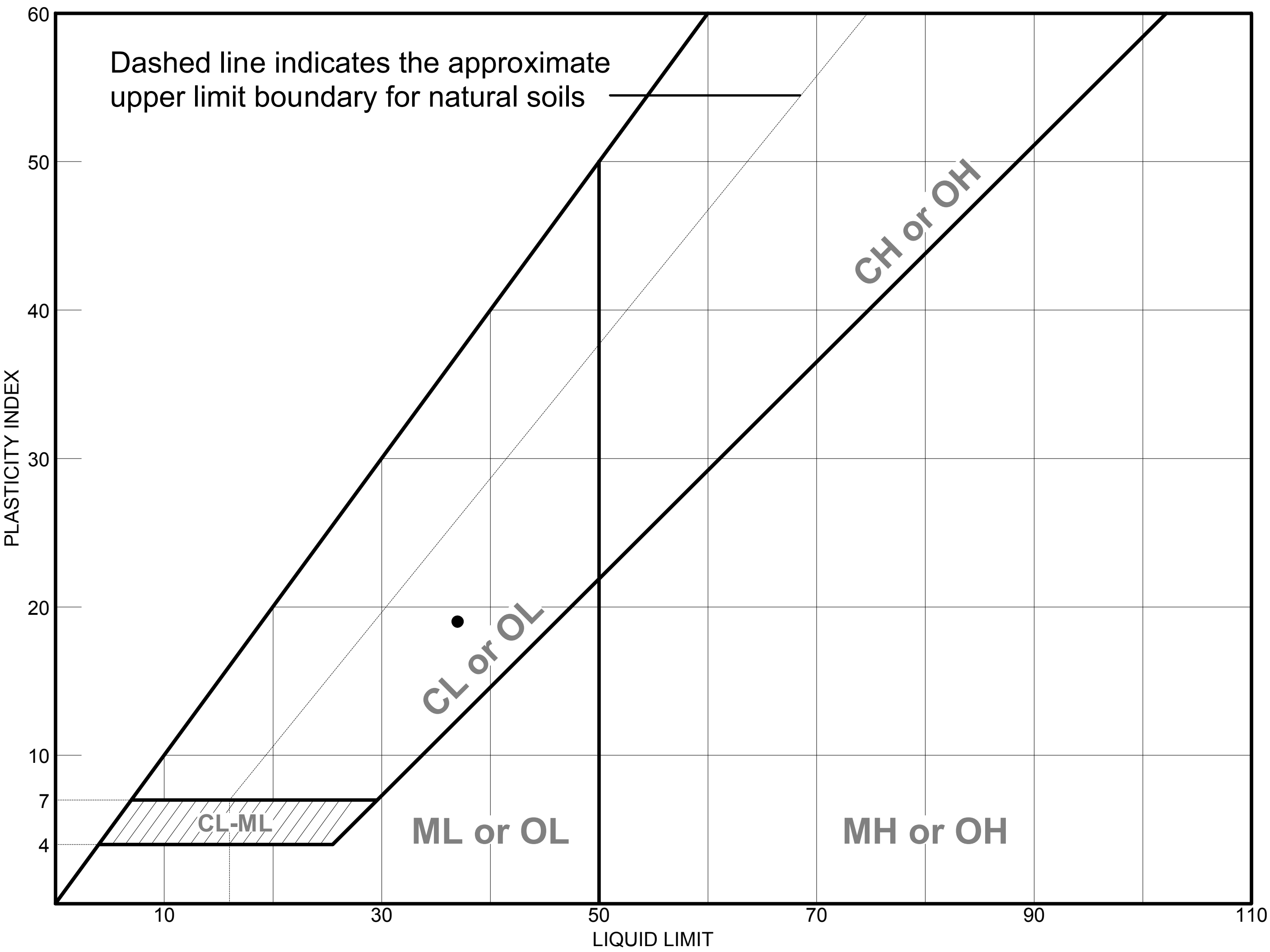
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	37	18	19	50.6	30.5	SC

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-4 **Elev./Depth:** 48.5-50'

Remarks:

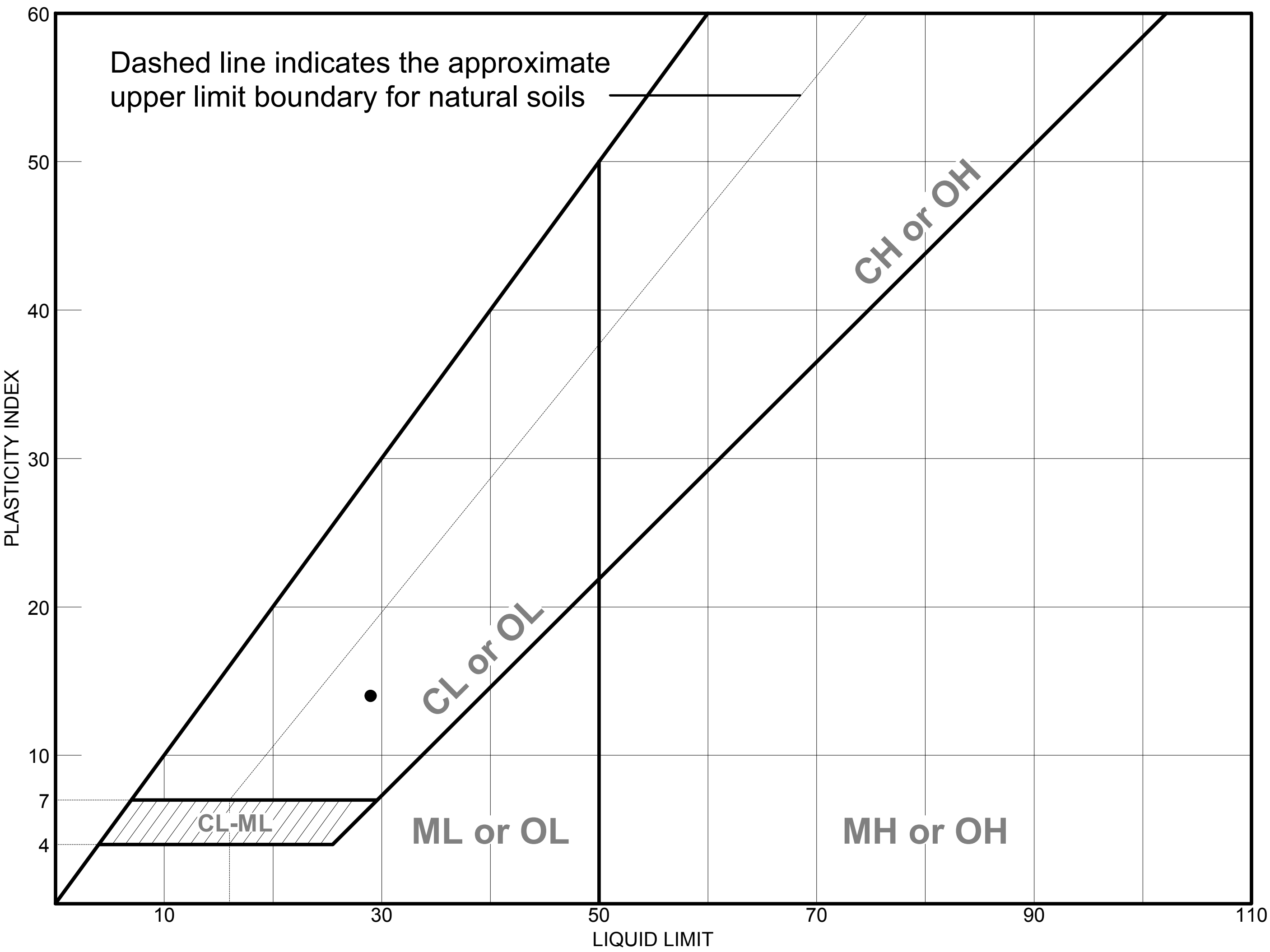
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		29	15	14			

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 8-10'

Remarks:

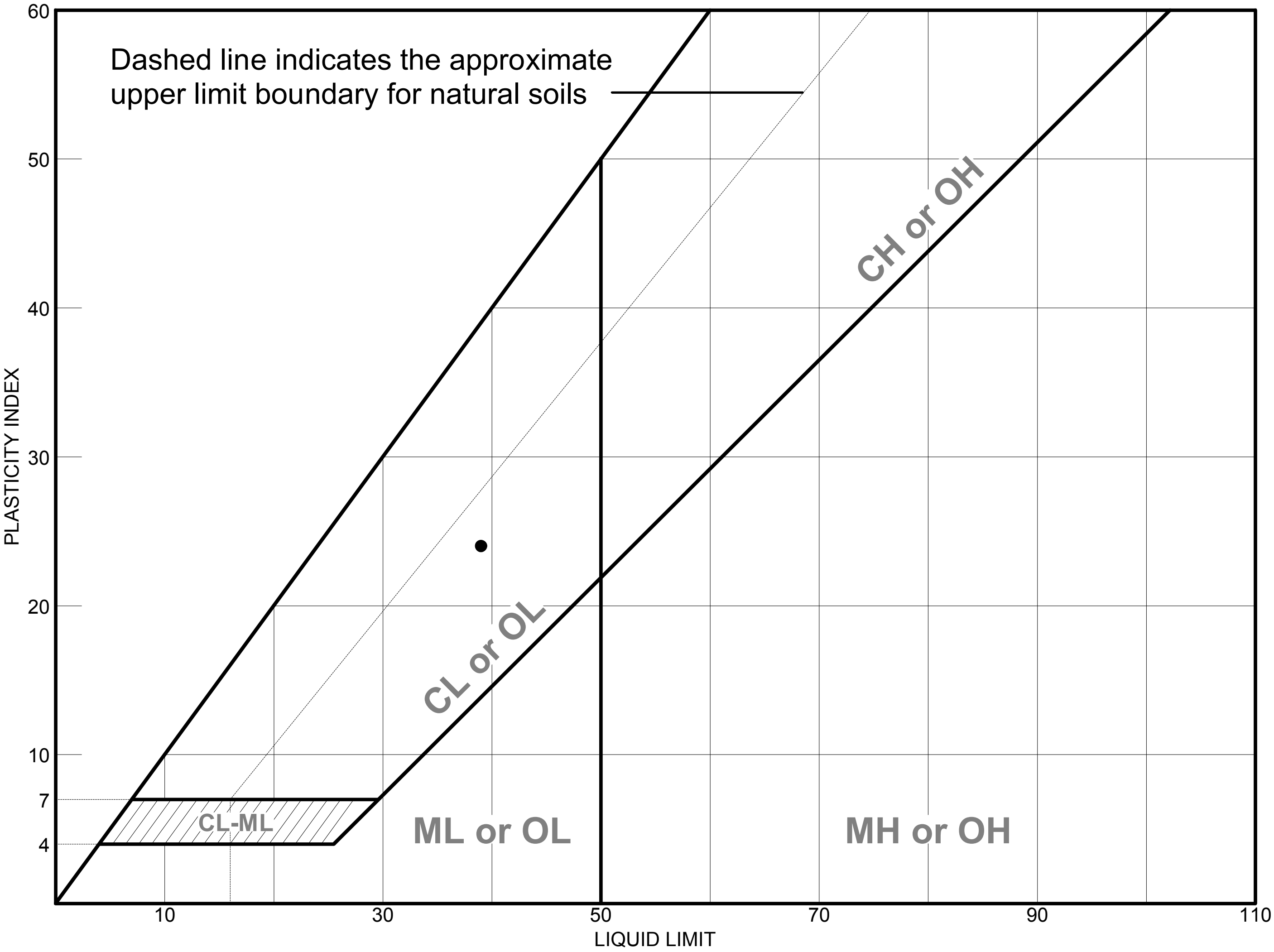
●

Moore Twining Associates, Inc.

Fresno, CA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	39	15	24	61.8	35.5	SC

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 10-11.5'

Remarks:

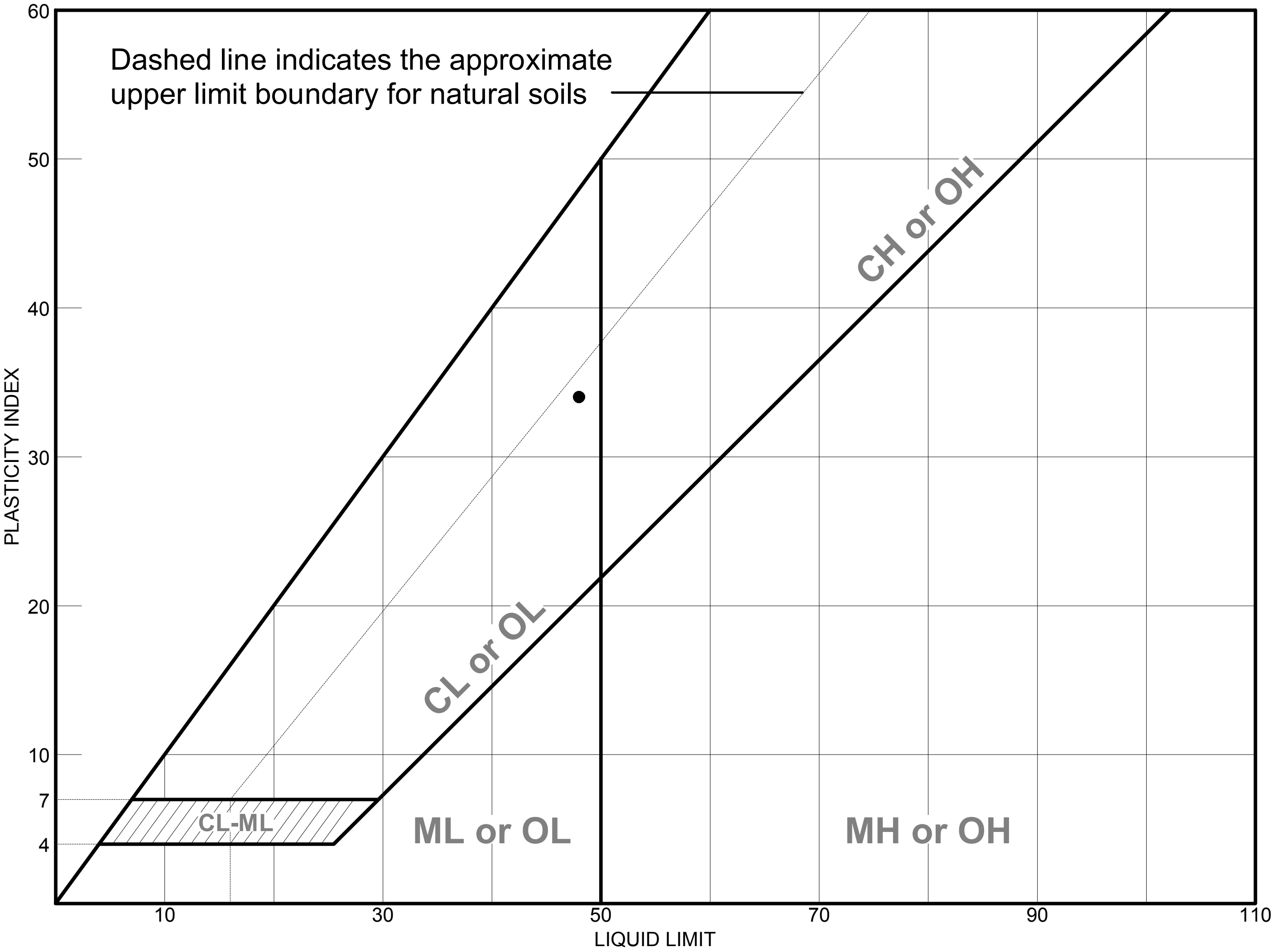
●

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	48	14	34	96.1	80.5	CL

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 35-36.5'

Moore Twining Associates, Inc.

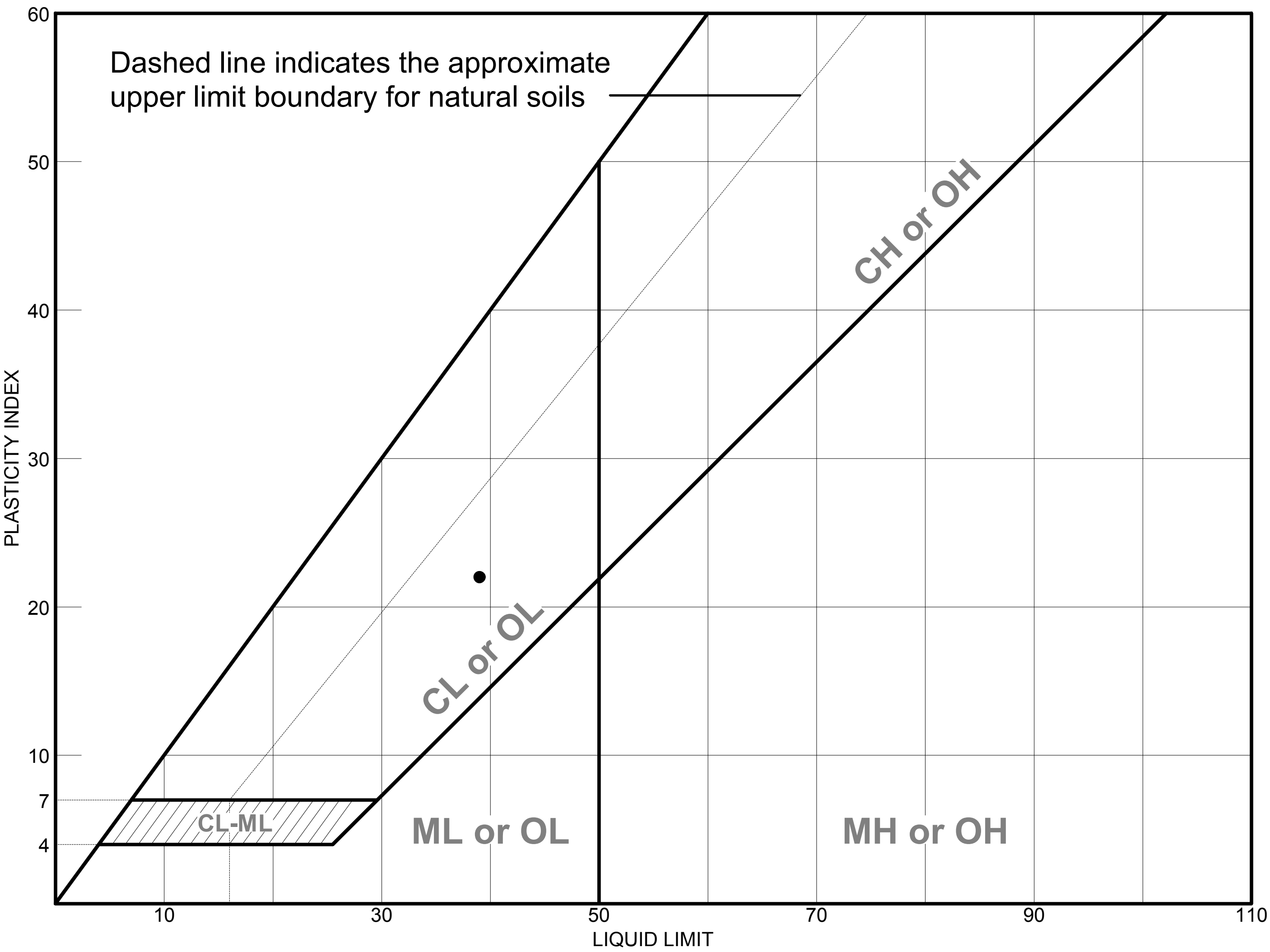
Fresno, CA

Remarks:

● 26.52
24.73

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay with sand	39	17	22	98.9	81.5	CL

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

● **Source:** **Sample No.:** B-9 **Elev./Depth:** 23.5-25'

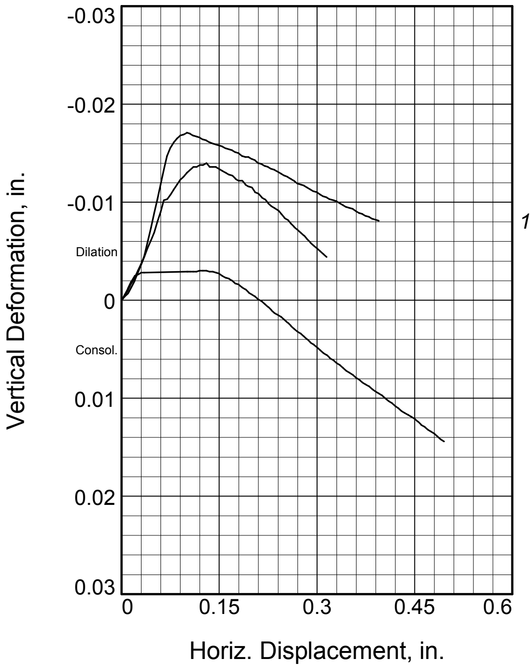
Remarks:

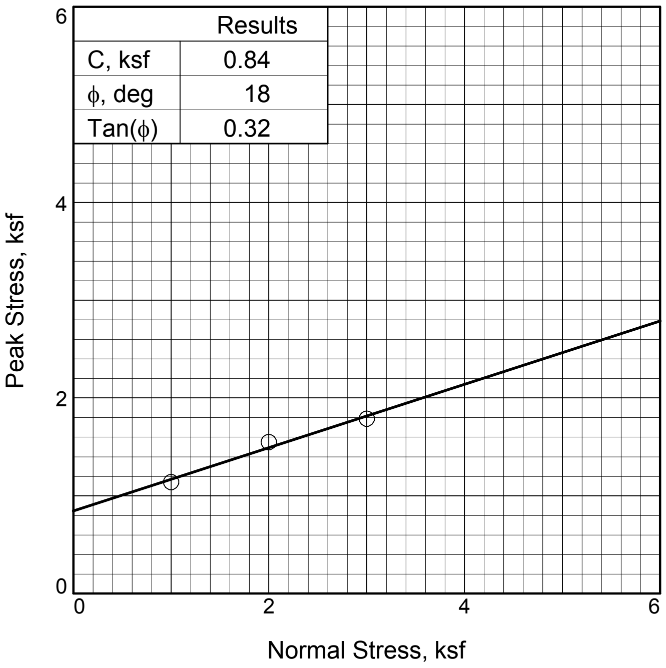
●

Moore Twining Associates, Inc.

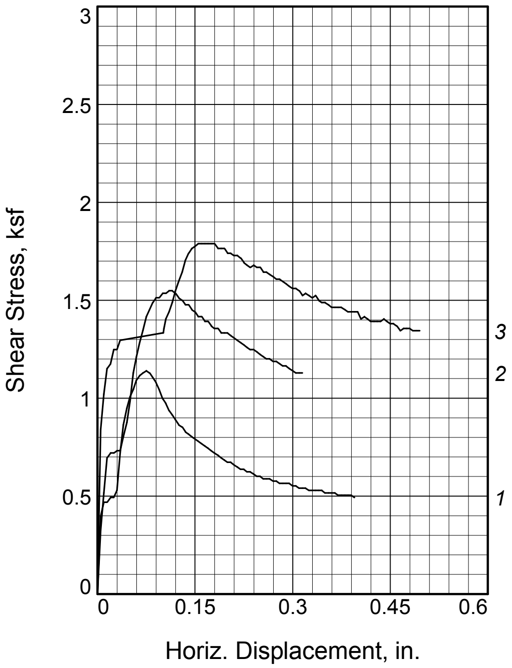
Fresno, CA

Figure





	Results
C, ksf	0.84
ϕ, deg	18
Tan(ϕ)	0.32



	Sample No.	1	2	3
Initial	Water Content, %	25.1	30.1	27.6
	Dry Density, pcf	93.1	89.9	92.2
	Saturation, %	85.5	95.1	92.1
	Void Ratio	0.7775	0.8399	0.7935
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	27.6	30.3	27.7
	Dry Density, pcf	95.2	91.7	95.1
	Saturation, %	99.1	100.0	99.1
	Void Ratio	0.7379	0.8031	0.7397
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.98	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	1.14	1.55	1.79
	Displacement, in.	0.08	0.11	0.16
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	3.1	4.6	6.4

Sample Type:
Description:

LL= 70 **PL=** 15 **PI=** 55
Specific Gravity= 2.65
Remarks:

Figure ________

Client: Home Depot USA, Inc.

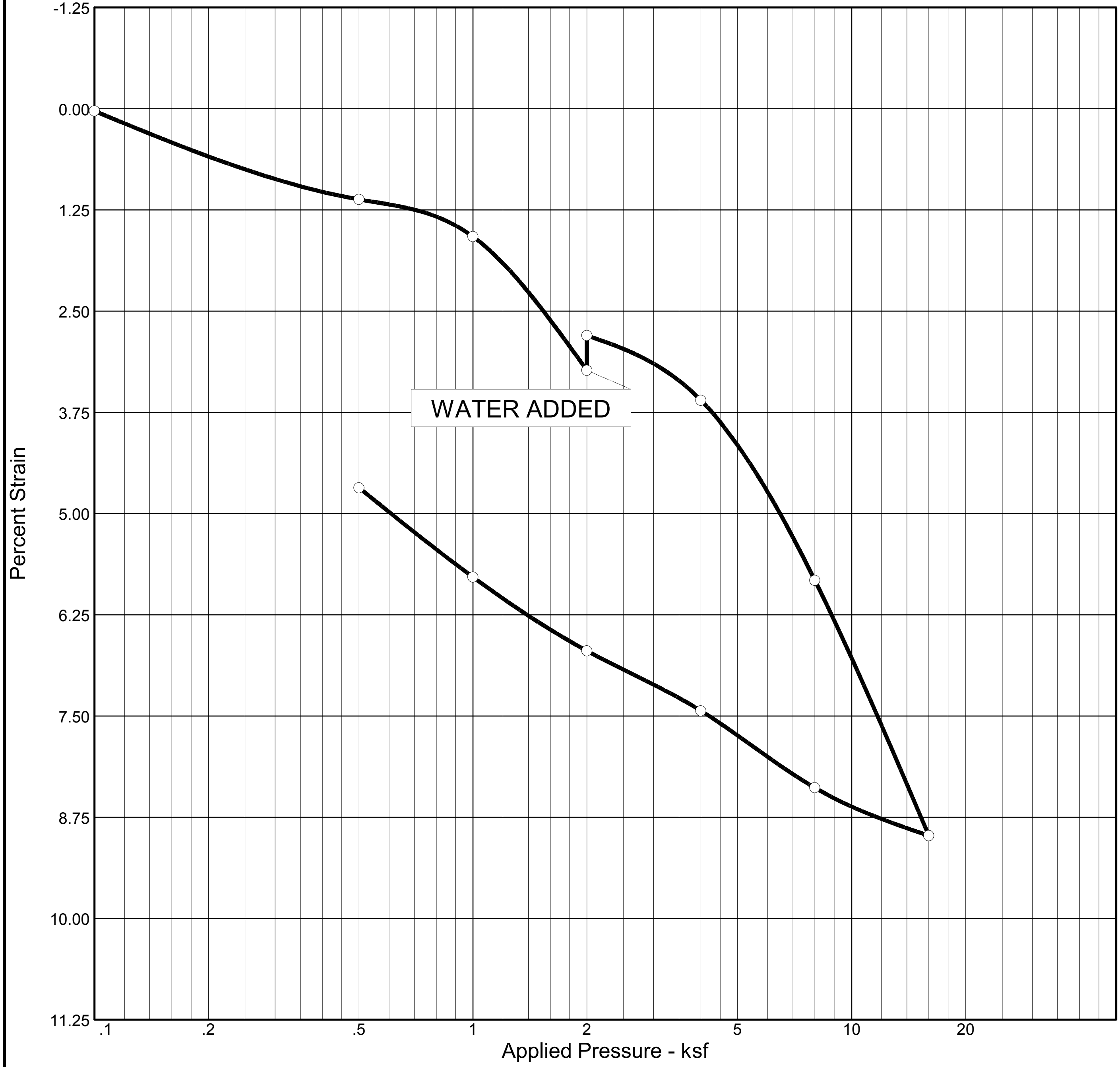
Project: Proposed Retail Takeover Petaluma

Sample Number: B-3 **Depth:** 3-4.5'

Proj. No.: D050S1.01 **Date Sampled:** 1/29/21

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

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.											
92.0 %	30.3 %	88.3			2.65		4.06	0.20	0.05	3.21	0.4	0.874

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

Source: **Sample No.:** B-1 **Elev./Depth:** 5-6.5'

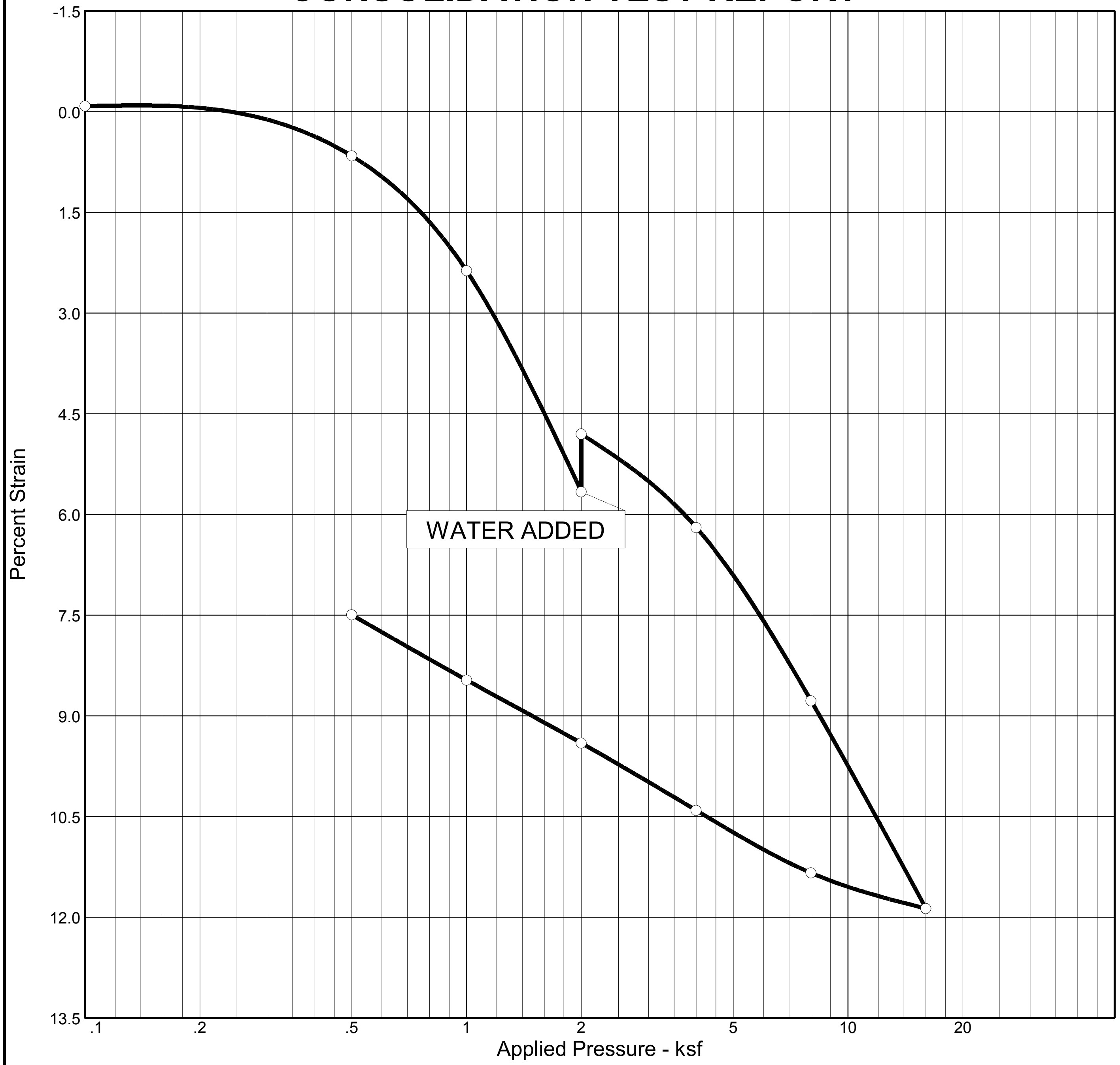
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
93.7 %	21.9 %	102.1	41	26	2.65		4.79	0.17	0.05	3.23	0.9	0.620

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

Source: **Sample No.:** B-2 **Elev./Depth:** 8-9.5'

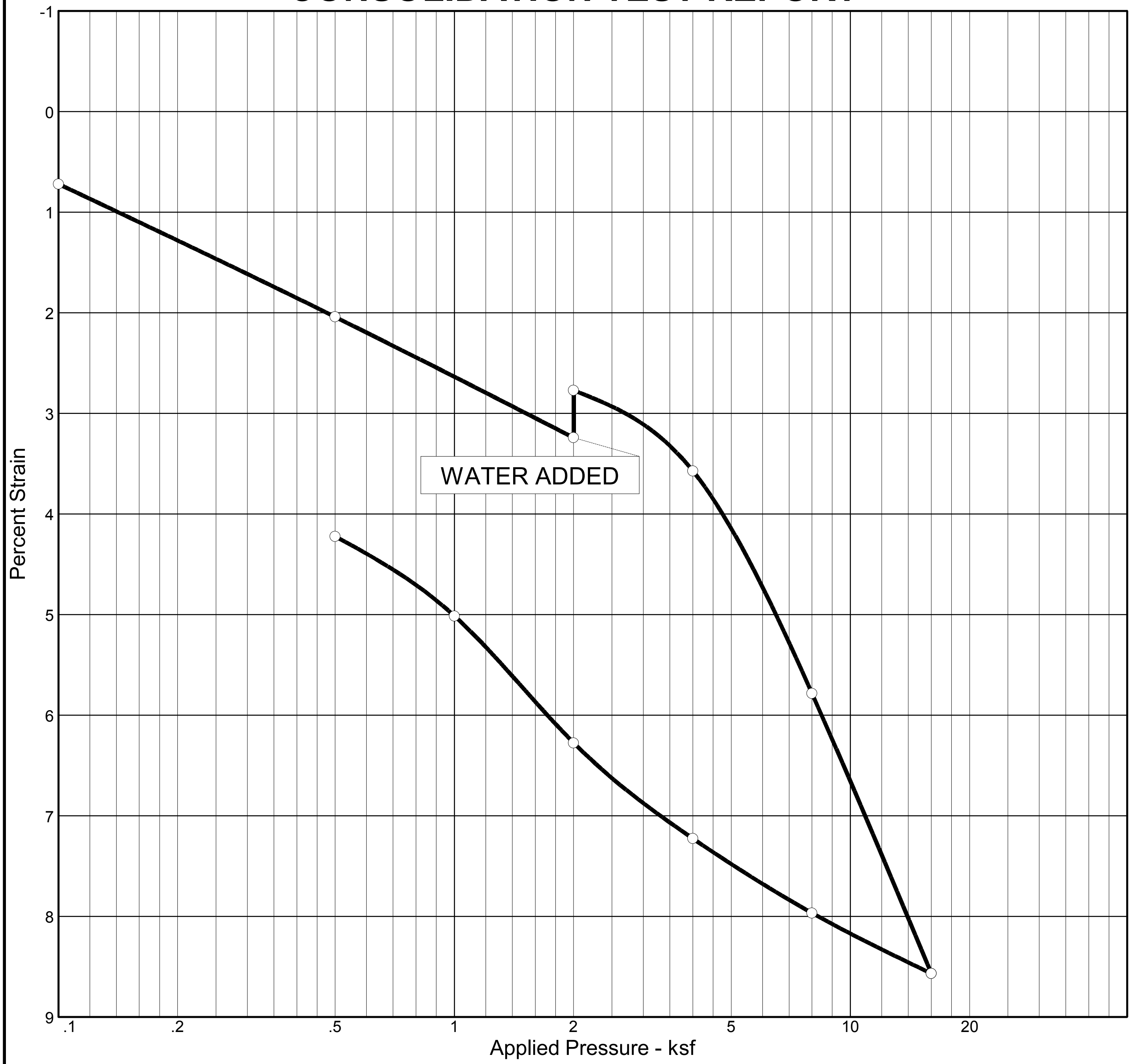
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
98.2 %	24.3 %	99.9			2.65		4.90	0.15	0.05	3.32	0.4	0.656

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. D050S1.01 **Client:** Home Depot USA, Inc.

Project: Proposed Retail Takeover Petaluma

Source: **Sample No.:** B-8 **Elev./Depth:** 10-11.5'

Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Proposed Retail Takeover Petaluma	REPORT DATE: TEST DATE:	3/4/2021 2/18/2021
MTA PROJECT NO.:	D050S1.01		
SAMPLE I.D.:	B-1 @ 2.6-5'		
SAMPLED BY:	AH		
SAMPLE DATE:	1/29/2021	TESTED BY:	TD

MATERIALS DESCRIPTION: Fat clay

% PASSING # 4 SIEVE: 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.8988
Pan + Dry Soil Wt., gm	212.8	Dry Soil Wt., lbs	0.6300
Pan Wt., gm	0.0		
Initial % Moisture Content	17.5	Final % Moisture Content	42.7

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.7401	Ring + Sample Wt., lbs	0.8988
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.7401	Remolded Wt., lbs	0.8988
Remolded Wet Density, pcf	101.8	Remolded Wet Density, pcf	109.5
Remolded Dry Density, pcf	86.6	Remolded Dry Density, pcf	76.8

Initial Volume	Final Volume
0.00727222	0.008207

Expansion Data:	
Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.1785
Expansion, in:	0.1285
Expansion Index	129

Comments: **High 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

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME: Proposed Retail Takeover
Petaluma
REPORT DATE: 3/4/2021
TEST DATE: 2/18/2021
MTA PROJECT NO.: D050S1.01
SAMPLE I.D.: B-3 @ 1.2-3'
SAMPLED BY: AH
SAMPLE DATE: 1/29/2021
TESTED BY: TD

MATERIALS DESCRIPTION: Fat clay with sand

% PASSING # 4 SIEVE 100

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	0.9469
Pan + Dry Soil Wt., gm	223.8	Dry Soil Wt., lbs	0.7618
Pan Wt., gm	0.0		
Initial % Moisture Content	11.7	Final % Moisture Content	24.3

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.8510	Ring + Sample Wt., lbs	0.9469
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.8510	Remolded Wt., lbs	0.9469
Remolded Wet Density, pcf	117.0	Remolded Wet Density, pcf	123.2
Remolded Dry Density, pcf	104.8	Remolded Dry Density, pcf	99.1

Initial Volume	Final Volume
0.00727222	0.007684

Expansion Data:	
Initial Gage Reading, in:	0.0500
Final Gage Reading, in:	0.1066
Expansion, in:	0.0566
Expansion Index	57

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

MOORE TWINING
ASSOCIATES, INC.
Since 1898
California ELAP Certificate #1371

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

February 24, 2021

Work Order #: **HB11015**

Zubair Anwar
MTA Materials Division
2527 Fresno St.
Fresno, CA 93721

RE: Proposed Retail Takeover- Petaluma

Enclosed are the analytical results for samples received by our laboratory on **02/11/21** . For your reference, these analyses have been assigned laboratory work order number **HB11015**.

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

MTA Materials Division	**Project:**	Proposed Retail Takeover- Petaluma	**Reported:**
2527 Fresno St.	**Project Number:**	D05051.01	02/24/2021
Fresno CA, 93721	**Project Manager:**	Zubair Anwar	

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B1@ 2.6-5		HB11015-01	Soil	01/29/21 00:00	02/11/21 11:31
B3 @ 1.2-3		HB11015-02	Soil	01/29/21 00:00	02/11/21 11:31

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

MTA Materials Division
2527 Fresno St.
Fresno CA, 93721

Project: Proposed Retail Takeover- Petaluma
Project Number: D05051.01
Project Manager: Zubair Anwar

Reported:
02/24/2021

B1@ 2.6-5

HB11015-01 (Soil) Sampled: 01/29/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**40**	6.0	mg/kg	3	B1B1815	02/18/21	02/19/21	Cal Test 422
Chloride		**0.004**	0.00060	% by Weight	3	[CALC]	02/19/21	02/19/21	[CALC]
Sulfate as SO4		**0.0025**	0.00060	% by Weight	3	[CALC]	02/19/21	02/19/21	[CALC]
pH		**8.6**	0.30	pH Units	3	B1B1815	02/18/21	02/19/21	Cal Test 643
Sulfate as SO4		**25**	6.0	mg/kg	3	B1B1815	02/18/21	02/19/21	Cal Test 417

B3 @ 1.2-3

HB11015-02 (Soil) Sampled: 01/29/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**37**	6.0	mg/kg	3	B1B1815	02/18/21	02/19/21	Cal Test 422
Chloride		**0.0037**	0.00060	% by Weight	3	[CALC]	02/19/21	02/19/21	[CALC]
Sulfate as SO4		**0.003**	0.00060	% by Weight	3	[CALC]	02/19/21	02/19/21	[CALC]
pH		**7.1**	0.30	pH Units	3	B1B1815	02/18/21	02/19/21	Cal Test 643
Sulfate as SO4		**30**	6.0	mg/kg	3	B1B1815	02/18/21	02/19/21	Cal Test 417

Notes and Definitions

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

MOORE TWINING

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

CHAIN OF CUSTODY / ANALYSIS REQUEST

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

WORK ORDER #: 2HB11015
PAGE 1 **OF** 2

REPORT TO:
ATTENTION: Zubair Answar
COMPANY NAME:
ADDRESS:
PHONE:
EMAIL / FAX:

☐ **INVOICE TO:**
ATTENTION:
COMPANY NAME:
ADDRESS:
PHONE:
EMAIL / FAX:

☐ **REPORT COPY TO:**

REPORTING:
☑ STANDARD FORMAT ☐ PDF
☐ EDT (SWRCB) ☐ EXCEL
☐ GEOTRACKER/COELT (LUFT)
GLOBAL ID:
☐ COUNTY ENVIRONMENTAL HEALTH:
☐ STATE WATER RESOURCES CONTROL BOARD:
☐ OTHER:

SAMPLE INFORMATION
SAMPLED BY (PRINT): Miguel A.
SIGNATURE:

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

TURN AROUND TIME
☑ STANDARD
☐ RUSH, DUE ON:

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

PROJECT INFORMATION
CONTRACT / P.O. NO.:
PROJECT: Proposed Retail Takeover Petaluma
PROJECT NUMBER: D05051.01
PROJECT MANAGER: Zubair Answar

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

ANALYSIS REQUESTED

LAB USE	CLIENT SAMPLE ID	DATE	TIME	TYPE	Corrosion										STATION CODE
1	B1@2.6-5	1/29/21	N/A	SL	✓										
2	B3@1.2-3	1/29/21	N/A	SL	✓										

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Miguel Alcaraz	MTA/MAT	2/11/21	11:31	K McCullough	MTA

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

Moore Twining Associates

Sample Integrity Page 2 of 2 WO# 4B11015 MTA Bottles: [✓ Yes] or No

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

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1-2														
Bacti $Na_2S_2O_3$															
None (P)															
Cr6 Buffer (P) Borate Carbonate Buffer															
HNO_3 (P)															
H_2SO_4 (P)															
NaOH (P)															
NaOH+ZnAc (P)															
Dissolved Oxygen 300ml (P)															
None (AG)															
None (CG) 500ml															
$Na_2S_2O_3$ 250ml (Brown P) 549															
$Na_2S_2O_3$ (AG)															
$Na_2S_2O_3$ (CG)															
Thio/K Citrate															
NH_4Cl (AG) 552															
HCl (AG)															
None (CG) 500ml															
H_3PO_4 (AG)															
Other:															
Plastic Bag	1														
Low Level Hg/Metals Double Bag															
Client Own															
Glass Jar: 125/ 250/ 500															
Soil Tube: Brass/ Steel/ Plastic															
5 g Encore															
Ascorboc Acid (AG) Voa															
1gallon Cubitainer															
Voa w/Sodium Thio + MCAA															

Filter or Split		Container	Preservative	Date/Time/Initials
	S P F			
	S P F			
	S P F			
	S P F			
	S P F			

Labeled by: ____@____ Labels checked by: ____@ 130

FL-SC-0003-06

MOORE TWINING
ASSOCIATES, INC.
Since 1898
California ELAP Certificate #1371

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

March 22, 2021

Work Order #: **HC09011**

Zubair Anwar
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Proposed Retail Takeover, Petaluma

Enclosed are the analytical results for samples received by our laboratory on **03/09/21** . For your reference, these analyses have been assigned laboratory work order number **HC09011**.

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

MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Retail Takeover, Petaluma
Project Number: D05051.01
Project Manager: Zubair Anwar

Reported:
03/22/2021

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B6 @ 1-5		HC09011-01	Soil	02/22/21 00:00	03/09/21 10:54

MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Retail Takeover, Petaluma
Project Number: D05051.01
Project Manager: Zubair Anwar

Reported:
03/22/2021

B6 @ 1-5

HC09011-01 (Soil) Sampled: 02/22/21 00:00

Analyte	Flag	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method
Inorganics									
Chloride		**12**	6.0	mg/kg	3	B1C2102	03/21/21	03/21/21	Cal Test 422
Chloride		**0.0012**	0.00060	% by Weight	3	[CALC]	03/21/21	03/21/21	[CALC]
Sulfate as SO4		**0.0034**	0.00060	% by Weight	3	[CALC]	03/21/21	03/21/21	[CALC]
pH		**8.6**	0.10	pH Units	1	B1C2102	03/21/21	03/21/21	Cal Test 643
Sulfate as SO4		**34**	6.0	mg/kg	3	B1C2102	03/21/21	03/21/21	Cal Test 417

Notes and Definitions

µg/L	micrograms per liter (parts per billion concentration units)
mg/L	milligrams per liter (parts per million concentration units)
mg/kg	milligrams per kilogram (parts per million concentration units)
ND	Analyte NOT DETECTED at or above the reporting limit
RPD	Relative Percent Difference

Analysis of pH, filtration, and residual chlorine is to take place immediately after sampling in the field.
If the test was performed in the laboratory, the hold time was exceeded. **(for aqueous matrices only)**

MOORE TWINING

CHAIN OF CUSTODY / ANALYSIS REQUEST

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

ANALYTICAL CHEMISTRY DIVISION
CALIFORNIA ELAP CERTIFICATION # 1371

WORK ORDER #: HC09011
PAGE 1 **OF** 2

REPORT TO:	☐ INVOICE TO: ☐ REPORT COPY TO:
ATTENTION: Zubair Answar	ATTENTION:
COMPANY NAME:	COMPANY NAME:
ADDRESS:	ADDRESS:
PHONE:	PHONE:
EMAIL / FAX:	EMAIL / FAX:

REPORTING:
☑ STANDARD FORMAT ☐ PDF
☐ EDT (SWRCB) ☐ EXCEL
☐ GEOTRACKER/COELT (LUFT)
GLOBAL ID:
☐ COUNTY ENVIRONMENTAL HEALTH:
☐ STATE WATER RESOURCES CONTROL BOARD:
☐ OTHER:

SAMPLE INFORMATION

SAMPLED BY (PRINT):

SIGNATURE:

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

TURN AROUND TIME
☑ STANDARD
☐ RUSH, DUE ON:

SAMPLE TYPES

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

PROJECT INFORMATION

CONTRACT / P.O. NO.:

PROJECT: Proposed Retail Takeover, Petaluma

PROJECT NUMBER: D0505 1.01

PROJECT MANAGER: Zubair Answar

ANALYSIS REQUESTED

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

LAB USE	CLIENT SAMPLE ID	DATE	TIME	TYPE	Corrosion										STATION CODE
1	B6 @ 1-5	2/22/21	N/A	SL	✓										

COMMENTS / ADDITIONAL INSTRUCTIONS:

RELINQUISHED BY	COMPANY	DATE	TIME	RECEIVED BY	COMPANY
Miguel Alcaraz	MTA/MAT	3/9/21	10:54	K McCullough	MTA

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

FL-SC-0005-04 A

Moore Twining Associates

Sample Integrity Page 2 of 2 WO# HC09011 MTA Bottles: Yes or [✓ No]

COC Info

Was temperature within range? Chemistry ≤6°C Micro <10°C Temp_____°C	Yes No [✓ N/A]	Did all bottle labels agree with COC?	[✓ Yes] No N/A
If samples were taken today, is there evidence that chilling has begun? Recvd_____C°	Yes No [✓ N/A]	Was a sufficient amount of sample received?	[✓ Yes] No N/A
Did all bottles arrive unbroken and intact?	[✓ Yes] No N/A	Were correct containers and preservatives received for the tests requested?	[✓ Yes] No N/A
Do samples have a hold time <72 hours?	Yes [✓ No] N/A		

Were there bubbles in VOA vials? (**Volatiles Only**)	Yes No [✓ N/A]
Was PM notified of discrepancies? PM: By/Time:	Yes No [✓ N/A]

Bottles Received

125ml (A) 250ml (B) 1Liter (C) 40ml VOA (V)	1														
Bacti $Na_2S_2O_3$															
None (P)															
Cr6 Buffer (P) Borate Carbonate Buffer															
HNO_3 (P)															
H_2SO_4 (P)															
NaOH (P)															
NaOH+ZnAc (P)															
Dissolved Oxygen 300ml (P)															
None (AG)															
None (CG) 500ml															
$Na_2S_2O_3$ 250ml (Brown P) 549															
$Na_2S_2O_3$ (AG)															
$Na_2S_2O_3$ (AG)															
Thio/K Citrate															
NH_4Cl (AG) 552															
HCl (AG)															
None (CG) 500ml															
H_3PO_4 (AG)															
Other:															
Plastic Bag	1														
Low Level Hg/Metals Double Bag															
Client Own															
Glass Jar: 125/ 250/ 500															
Soil Tube: Brass/ Steel/ Plastic															
5 g Encore															

Filter or Split

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

Labeled by:_____@_____ Labels checked by:_____@ 1250

FL-SC-0003-06

MOORE TWINING
ASSOCIATES, INC.
Since 1898

Project Name: Proposed Retail Takeover Petaluma
Project Number: D050S1.01

Subject: Minimum Resistivity, ASTM G187
Material Description: Fat clay
Location: B-1 @ 2.6-5'

Report Date: 3/4/2021
Sample Date: 1/29/2021

Sampled By: AH
Tested By: MA
Test Date: 2/18/2021

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
200 mls	4,300
225 mls	3,100
250 mls	1,700
275 mls	910
300 mls	770
325 mls	720
350 mls	490
375 mls	450
400 mls	440
425 mls	440
450 mls	460

Remarks: Min. Resistivity is 440 Ohm-cm

Project Name:	Proposed Retail Takeover Petaluma	Report Date:	3/4/2021
		Sample Date:	1/29/2021
Project Number:	D050S1.01		
		Sampled By:	AH
Subject:	Minimum Resistivity, ASTM G187	Tested By:	MA
Material Description:	Fat clay with sand	Test Date:	2/18/2021
Location:	B-3 @ 1.2-3'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
200 mls	2,900
225 mls	1,400
250 mls	950
275 mls	820
300 mls	710
325 mls	510
350 mls	500
375 mls	500
400 mls	520

Remarks: Min. Resistivity is 500 Ohm-cm

Project Name: Proposed Retail Takeover Petaluma
Project Number: D050S1.01

Subject: Minimum Resistivity, ASTM G187
Material Description:
Location: B-6 @ 1-5'

Report Date: 3/18/2021
Sample Date: 2/22/2021

Sampled By: AV
Tested By: MA
Test Date: 3/11/2021

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
100 mls	9,300
125 mls	8,600
150 mls	6,600
175 mls	3,900
200 mls	1,700
225 mls	970
250 mls	820
275 mls	440
300 mls	410
325 mls	430

Remarks: Min. Resistivity is 410 Ohm-cm