Appendix B
Geological Technical Report

MOORE TWINING ASSOCIATES, INC.

Since 1898

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED STARBUCKS

SOUTHWEST CORNER OF 7th AVENUE AND MAIN STREET

HESPERIA, CALIFORNIA

Project Number: H33201.01

For:

Fountainhead Development
1401 Quail Street, Suite 100
Newport Beach, CA 92660

November 15, 2024

November 15, 2024 H33201.01

Fountainhead Development
1401 Quail Street, Suite 100
Newport Beach, CA 92660

Attention: Ms. Vasanthi Okuma

Subject: **Geotechnical Engineering Investigation**
Proposed Starbucks
SWC 7th Avenue and Main Street
Hesperia, California

Dear Ms. Okuma:

We are pleased to submit this geotechnical engineering investigation report prepared for a proposed Starbucks to be located on the southwest corner of 7th Avenue and Main Street in the City of Hesperia, California.

The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

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

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

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

Sincerely,

MOORE TWINING ASSOCIATES, INC.

Allen H. Harker, CEG
Engineering Geologist
Geotechnical Engineering Division

CERTIFIED ENGINEERING GEOLOGIST
ALLEN H. HARKER
No. 2781
EXP. 7-31-26
STATE OF CALIFORNIA

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. (Moore Twining) prepared this geotechnical engineering investigation report for the proposed Starbucks to be located in Hesperia, California.

The subject site is located on the southwest corner of 7th Avenue and Main Street in Hesperia, California. The area indicated for the proposed Starbucks is a 0.596-acre property which includes an auto sales business and auto repair shop in the north half of the site and pavements from a former auto sales businesses in the south half of the site.

The proposed Starbucks development is planned to include a 1,263 square foot single story building with a drive-thru pickup drive lane. Appurtenant construction is anticipated to include concrete walkways, asphaltic concrete and Portland cement concrete parking and drive areas, underground utilities, and landscape areas.

Moore Twining conducted a previous investigation at the subject site when the Starbucks parcel and adjacent McDonald's parcel (west of the Starbucks parcel) were being considered for development of a Circle K convenience store, car wash and gas station. Near surface infiltrations systems were not deemed to be feasible from Moore Twining's previous February 13, 2019 "Results of Percolation Testing" report. However, deeper poorly graded sand layers were previously encountered at the site and were targeted to conduct deeper percolation tests for consideration of infiltration systems such as dry wells to be used as part of the proposed construction.

On October 23, 2024, five (5) test borings were drilled at the site to depths ranging from 15 to 60 feet below site grades (BSG). The subsurface soils encountered generally consisted of very loose to medium dense silty sands extending to depths of about 1½ to 3½ feet across the site. Below the very loose to loose silty sands, the relative density of the silty sands soils improved to medium dense to dense and extended to depths of about 3½ to 13½ feet BSG. Below the silty sands, medium dense silty, clayey sands; medium dense clayey sands; medium dense to dense poorly graded sands with silt; and medium dense well graded sands with silt were encountered extending to a depth of about 33½ feet BSG which was generally underlain by dense poorly graded sands and dense well graded sands with silt extending to the maximum depth explored of 60 feet BSG.

The surface soils encountered are non-plastic and non-expansive. These soils exhibit low compressibility characteristics, slight collapse potential, and moderate to high shear strength properties. The near surface soils exhibit fair support characteristics for pavements when compacted as engineered fill.

Due to the depth to historical groundwater levels in the vicinity of this site (greater than 450 feet BSG), liquefaction is not considered a concern for the proposed development. However, there is potential for dry seismic settlement to occur during shaking from earthquakes. As part of the analysis, the (N1)60s values of 30 or greater (dense to very dense soils) were not considered to be subject to significant dry seismic settlement in the analyses. Based on the analysis, seismic settlement was estimated to be negligible.

EXECUTIVE SUMMARY

Foundations supported directly on the existing loose native silty sands would be subject to excessive static settlement. In order to reduce the potential for excessive settlement of foundations, over-excavation and compaction of the upper 4 feet of the near surface soils, or to a depth of 12 inches below the bottom of the foundations, or to the depth required to remove existing undocumented fill soils, or to at least 12 inches below subsurface improvements (structures, utilities, etc.) to be removed, whichever is greater, followed by scarification and compaction of an additional 8 inches is recommended in the building pad areas to reduce the total and differential static settlement to 1 inch total and ½ inch differential. An allowable bearing capacity of 2,500 pounds per square foot is recommended for foundation design, for dead-plus-live loads.

The closest active fault is the Ord Mountain Fault zone (part of the North Front Thrust System), which is located about 6½ miles southeast of the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Accordingly, the potential for ground rupture at the site is considered low.

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

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

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

TABLE OF CONTENTS

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED STARBUCKS

SOUTHWEST CORNER OF 7th AVENUE AND MAIN STREET

HESPERIA, CALIFORNIA

Project Number: H33201.01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for a proposed Starbucks to be located on the southwest corner of 7th Avenue and Main Street in Hesperia, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Fountainhead Development to perform this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, site description, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices contain the drawings (Appendix A), the logs of borings and (Appendix B), the results of laboratory tests (Appendix C), the results of percolation tests (Appendix D) and the compaction report, test data and test locations for backfill of the area of removed underground storage tanks with engineered fill (Appendix E).

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

2.0 PURPOSE AND SCOPE OF INVESTIGATION

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

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

2.1.2 Recommendations for 2022 California Building Code seismic coefficients and earthquake spectral response acceleration values;

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

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

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

2.1.6 Recommendations regarding infiltration of storm water;

2.1.7 Recommendations for temporary excavations and trench backfill, and

2.1.8 Conclusions regarding soil corrosion potential.

This report is provided specifically for the Starbucks referenced in the Anticipated Construction section of this report. This investigation did not include a geologic/seismic hazards evaluation, flood plain investigation, compaction tests, environmental investigation, or environmental audit.

2.1.9 **<u>Scope</u>:** Our revised proposal (MTP 24-0548R), dated October 2, 2024, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.1.10 The conceptual site plan SP-8, dated June 12, 2024, prepared by Greenberg Farrow, was reviewed for project information.

2.1.11 A report entitled, "Geotechnical Engineering Investigation, Proposed Circle K Store, Southwest Corner of 7th Avenue and Main Street, Hesperia, California," prepared by Moore Twining, dated January 24, 2019, Moore Twining Project No. G28812.02, was reviewed. This investigation was previously conducted by Moore Twining for a previous Circle K development on the currently planned Starbucks parcel and the adjacent McDonald's parcel on the west side of the Starbucks parcel.

A report entitled, "Supplemental Report of Percolation Testing, Proposed Circle K Store, Southwest Corner of Main Street and 7th Avenue, Hesperia, California," prepared by Moore Twining, dated February 13, 2019, Moore Twining Project Number G28812.02, was reviewed. The percolation testing conducted by Moore Twining in 2019 included shallow percolation testing in the upper 5 feet below site grade on both the currently planned Starbucks and McDonald's parcels.

In addition, a draft report entitled, "Phase I Environmental Site Assessment, Proposed Circle K, 15901 Main Street, Hesperia, California 92345, prepared by Moore Twining's Environmental Division, dated February 13, 2019, Moore Twining Project Number G28812.01, was reviewed.

2.1.12 A visual site reconnaissance and subsurface exploration were conducted.

2.1.13 Satellite images of the site between the years 1994 and 2023 from online sources, were reviewed.

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

2.1.15 Ms. Vasanthi Okuma (Fountainhead Development and Mr. Thomas Hawksworth (C3 Civil Engineering) were consulted during the investigation.

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

2.1.17 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.

3.1.1 Site Description: The site is located at the southwest corner of 7th Avenue and Main Street in Hesperia, California. The north portion of the site was occupied by a Best Buy Auto Sales business, which has an address of 15901 Main Street. The area indicated for the proposed Starbucks development is a 0.596-acre property. The conceptual site plan SP-8, dated June 12, 2024, prepared by Greenberg Farrow, shows a proposed McDonald's fast-food restaurant on the west side of the Starbucks parcel; however, this report only includes a geotechnical engineering investigation for the proposed Starbucks. A site location map is presented on Drawing No. 1 in Appendix A. The site is located at 34.423305 degrees latitude and -117.316065 degrees longitude.

The streets that bound the site are not aligned to true north and are skewed slightly. For the purpose of this report, the assumed north direction is towards Main Street. So, the site is bound to the north by Main Street, to the east by 7th Avenue, to the south by Walnut Street and to the west by the proposed McDonald's parcel with retail shops and a parking lot beyond. The McDonald's parcel adjacent to the west side of the site includes an asphalt concrete paved parking lot in the northern half of the site, and an unpaved dirt lot with a concrete slab-on-grade (about 3,250 square feet) from a previous development in the southern half of the site.

The north half of the Starbucks site is occupied by a retail auto sales and repair business. Also, a canopy is present in the north portion of the site which covered former fuel islands. The existing sales/shop building is located within the building footprint for the proposed Starbucks building and occupies about 1,200 square feet. Equipment surrounded by a chain link fence was noted on the south side of the existing building. A trash enclosure was also noted on the south side of the existing building. Most of the remaining portions of the north half of the site were covered with asphalt concrete pavements in poor condition with large longitudinal and traverse cracking, some areas of raveling, and some patches. Also, underground utility scans identified numerous pipelines throughout the site.

The south half of the site did not include any above grade improvements. However, asphalt concrete pavements covered this area and two small concrete slabs-on-grade (about 160-square-feet and 120-square feet) were located in the western portion of the south half of the site. An exposed pipe was noted as extending vertically from the ground surface on the west side of the 120-square-foot slab-on-grade, and the outline of a trench was noted as extending northeast from the east side of the 120-square-foot slab-on-grade. The exposed pipe on the west side of the 120-square-foot slab-on-grade is believed to be a sewer or septic pipe. An apparent sewer cleanout valve also extended above the ground surface adjacent to the sewer pipe. Another pipe with a steel plate at the top of the pipe extended vertically from the ground surface within the 160-square-foot slab-on-grade. The pavements in the southern half of the Starbucks site were in poor condition with extensive weathering, severe block cracking and weeds growing out of the cracks. A chain link fence surrounded the northern, eastern and southern sides of the southern half of the site with an opening in the fence in the southern portion of the site that provides access to the site. A power pole was noted in the southeast portion of the site with an overhead power line extending southeast of the power pole. A tree was noted along the southern property line in the southwest corner of the site.

3.2 <u>Site History and Previous Studies</u>: As a part of this investigation, a Draft Phase I Environmental Site Assessment Report (Phase I ESA) and on-line aerial images were reviewed regarding the history of the site that are pertinent to this investigation.

The review of historical aerial photographs and city directories, conducted as a part of the Phase I ESA, indicated that the site was occupied by open, undeveloped land since before 1938 until the 1950's. In 1959, a building was present on the site in the north portion. During the 1980's, portions of the southern half of the site were paved, and by at least 1983 until 2004, the northeast portion of the site operated as a gas station. From 2005 to the time of our October 2024 field investigation, Best Buy Auto Sales has operated at the site. At the time of this investigation, the Phase I ESA had identified some records that the underground storage tanks associated with the past fuel facilities had been removed. Three (3) underground storage tanks in the western portion of the fuel canopy and southwest of the fuel canopy were reportedly removed in 1998. Moore Twining's Draft Phase I Environmental Site Assessment prepared for the previous Circle K development indicated that

Advanced Environmental Concepts observed and tested the backfill of two areas for the three (3) underground storage tanks that were removed and summarized their results in a December 1, 1998 report. The results of eleven tests presented in the report prepared by Hi Desert Testing & Inspection for Advanced Environmental Concepts show that the compacted fill met the minimum required 90 percent relative compaction. The initial tests in each area tested placement of fill at depths of about 10 and 11 feet below site grade. Thus, the areas of the excavations made to remove the underground storage tanks were at least about 10 to 11 feet in depth below adjacent site grades. The area of the three (3) removed Underground Storage Tanks appears to be northwest of the proposed Starbucks building (see Drawing No. 2 in Appendix A of this report). The compaction test report, test data and test locations for backfill of the area of the removed underground storage tanks with engineered fill is included in Appendix E of this report.

Aerial images of the site were also reviewed between May 1994 and December 2017. The 1994 and 1995 images of the site appear to show a service station in the northern portion of the site, and an open car sales lot in the southern portion of the site. By 2009, the southern portion of the site was vacant (no cars parked for sale). Between June and December 2017, a small building was removed in the southern half of the site, and two slabs-on-grade (about 160-square-feet and 120-square feet) remained. A sewer or septic pipe also remained adjacent to the west of the 120-square-foot slab-on-grade and an outline of a trench was noted as trending in a northeast direction from the east side of the 120-square-foot slab-on-grade. Another pipe with a steel plate at the top of the pipe extended vertically from the ground surface within the 160-square-foot slab-on-grade. The Draft Phase I ESA report indicated, “The pipes observed during the site reconnaissance indicate that a septic tank may have been associated with the site. As a result, the tank, piping and leach field(s) may be encountered and could impact future development. Additionally, it is unknown whether the historical building foundations located on the site maintained septic systems, and if so, whether they were removed. Costs would be incurred to handle the removal of the tank(s), lines and leach field(s) upon discovery. If the septic system(s) (tanks, piping, leach fields, etc.) is (are) discovered during development, especially in the area of any planned construction, the septic system(s) will need to be removed.” The site appears to be consistent with the current site uses in images for various years after the 2017 aerial image of the site was taken.

Moore Twining’s Geotechnical Engineering Division prepared a report for the site entitled, “Geotechnical Engineering Investigation, Proposed Circle K Store, Southwest Corner of 7th Avenue and Main Street, Hesperia, California,” prepared by Moore Twining, dated January 24, 2019, Moore Twining Project No. G28812.02. The investigation was conducted on both the parcels for the proposed Starbucks and McDonald’s development that was previously planned for development of a Circle K store, car wash and gas station. The investigation included drilling five (5) test borings at the site to depths ranging from 15 to 27 feet below site grades (BSG) in January 2019. The maximum depth proposed for the investigation of 50 feet BSG could not be achieved due to auger refusal on materials that were possibly cemented or cobbles at depths of 25 and 27 feet BSG in two attempts to reach the target maximum depth. The soils encountered consisted of silty sands extending to depths ranging from about 20 to 25 feet BSG. Below the silty sand, poorly graded sands with silt soils were encountered to the maximum depth explored 27 feet BSG. Drilling refusal was encountered at depths of 25 and 27 feet BSG at boring locations B-1 and B-2 due to suspected cobbles. Laboratory testing on the near surface soils indicated the materials were non-plastic, non-expansive, and exhibited moderate compressibility, moderate collapse, and moderate to high shear strength properties. Laboratory testing on the near surface silty sand soils also indicated the near surface soils exhibited good support characteristics for pavements when compacted as engineered fill. Due to the soils exhibiting moderate collapse in the upper 5 feet, the report recommended over-excavation for the proposed Circle K store to a depth of 5 feet below preconstruction site grade, to the depth required to provide at least 2 feet of engineered fill below bottom of footings, to the depth required to remove existing undocumented fill soils and to at least 12 inches below subsurface improvements (structures, utilities, etc.) to be removed, whichever provided the deeper excavation. The Circle K store was recommended to be supported on shallow foundations and designed based on an allowable bearing pressure of 2,500 pounds per square foot for dead-plus-live loads which could be increased by one-third for short duration of seismic loads. Perimeter footings were recommended to extend to a depth of 18 inches below lowest adjacent finished exterior grade, and interior footings were recommended to extend to a minimum depth of at least 12 inches below the bottom of the slab-on-grade. The report recommended the following settlements to be anticipated for design: 1) a total static settlement of 1 inch, 2) a differential static settlement of ½-inch in 40 feet, 3) a total seismic settlement of ¼ inch, and 4) a differential seismic settlement of ¼ inch in 40 feet.

Moore Twining also issued a report for the previous Circle K development entitled, "Supplemental Report of Percolation Testing, Proposed Circle K Store, Southwest Corner of Main Street and 7th Avenue, Hesperia, California," dated February 13, 2019, Moore Twining Project Number G28812.02. An additional boring was drilled in the northeast corner of the site to a depth of 16½ feet BSG, and three (3) percolation test borings were drilled to depths of 3 feet, 4 feet and 5 feet BSG. The percolation tests were conducted within near surface silty sand soils, some of which exhibited cementation. The percolation tests indicated a negligible percolation rate in one of the percolation tests and unfactored infiltration rates of 1.7 inches and 3.0 inches per hour in the other two tests. However, Moore Twining concluded, "Since the borings indicate that the dense cemented soils occur below about 4 to 5 feet across the site and these materials did not have any significant measured infiltration during testing, it does not appear that on-site infiltration of significant stormwater in the near surface soils will be feasible."

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

3.3 **<u>Anticipated Construction</u>:** The latest conceptual site plan SP-8, dated June 12, 2024, prepared by Greenberg Farrow indicates the Starbucks development will include a 1,263 square foot single story Starbucks building and a drive-thru pick up drive lane. Appurtenant construction is indicated to include concrete walkways, asphaltic concrete and Portland cement concrete parking and drive areas, a trash enclosure, underground utilities, and landscaped areas.

It is anticipated that the proposed Starbucks structure will consist of a one-story building including concrete masonry unit wall or wood-framed construction with concrete slab-on-grade floors. It is anticipated that the proposed building will be supported on shallow spread foundation systems. Basements and loading docks are not anticipated as part of the proposed construction.

Based on our experience with past Starbucks projects, it is assumed the that the proposed Starbucks building will have maximum column loads of about 10 kips and maximum wall loads of about 1.5 kips per linear foot for dead-plus-live loads. In the event that the maximum foundation loads exceed those assumed for design, the recommendations of this report may not be applicable and may need to be revised.

Based on the lack of significant slope or grades differences noted across the site, cuts and fills on the order of 1 to 2 feet are anticipated to achieve level pad grades and provide site drainage.

Near surface infiltrations systems were not deemed to be feasible from Moore Twining's previous February 13, 2019 "Results of Percolation Testing" report. However, deeper poorly graded sand layers were previously encountered at the site and were targeted to conduct deeper percolation tests for consideration of infiltration systems such as dry wells to be used as part of the proposed construction.

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 conducting percolation tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by a Moore Twining field engineer on October 23, 2024. The features noted are described in the background information section of this report.

4.1.2 Drilling Test Borings: Prior to drilling, the site was marked for Underground Service Alert for members to mark out the locations of existing public utilities. Also, an underground utility locating service was retained to scan the proposed boring locations to identify potential private on-site underground utilities that could be damaged during drilling. The borings were then offset from marked underground utilities.

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

On October 23, 2024, five (5) test borings were drilled at the site to depths ranging from 15 to 60 feet below site grades (BSG). Boring B-1 was intended to be drilled near the southeast corner of the proposed building footprint. However, due to the presence of an existing building and overhead power line trending southeast from the southeast corner of the existing building, boring B-1 had to be drilled on the east side of the existing building. Boring B-1 could not be drilled on the south side of the existing building as this area was occupied by an equipment storage area and surrounded by chain link fencing. Boring B-2 was drilled to 60 feet BSG within the northern portion of the proposed Starbucks building footprint (and north of the existing building) for evaluation of liquefaction. Boring B-3 was drilled to a depth of about 15 feet BSG within the entrance to the proposed drive-thru pickup drive lane area. Two (2) of the borings (P-1 and P-2) were drilled to depths of about 20 feet BSG to install percolation test pipe in the boreholes and conduct percolation tests. At the direction of Mr. Thomas Hawksworth (C3 Civil Engineering), the percolation tests were drilled in the northern portion of the Starbucks parcel and the northern portion of the adjacent McDonald's parcel. The boring locations are shown on Drawing No. 2 in Appendix A of this report. The borings were drilled with a conventional truck-mounted CME-75 drill rig equipped 8-inch outside diameter (O.D.) hollow-stem augers.

The test borings were drilled under the direction of a Moore Twining Geotechnical Engineer. The soils encountered in the test borings were logged during drilling by a representative of our firm. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

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

Test boring locations were determined with reference to existing property corners and site features shown on the site plan. The locations of the test borings are described on the boring logs in Appendix B of this report. The test borings were backfilled with material excavated during the drilling operations and patched with asphalt concrete cold patch materials.

4.1.3 <u>Soil Sampling</u>: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

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

During the drilling of the test borings, bulk samples of soil were also obtained for laboratory testing. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.1.4 **<u>Percolation Tests</u>:** Two percolation tests was conducted on October 24, 2024. Percolation test borings P-1 and P-2 were drilled to depths of about 20 feet BSG on October 23, 2024. The locations of the percolation tests are shown on Drawing No. 2 in Appendix A of this report.

Percolation tests were conducted at locations P-1 and P-2 and infiltration rates were estimated from the percolation test data.

The percolation testing was conducted in general conformance with San Bernardino County's Section VII.3.8 in Appendix D of their "Technical Guidance Document for Water Quality Management Plans," effective date September 19, 2013, which utilizes the percolation test procedure per Riverside County Department of Environmental Health. The percolation tests included placement of about 2 inches of gravel at the bottom of the hole and installation of percolation test pipe with gravel in the annulus space to keep the pipe stabilized and reduce the potential for washout of the soils on the sides of the holes within the test zone. On the day prior to the testing, about 5 gallons of water was added to each hole. On the day of the percolation tests, per the procedure for deep percolation tests, the percolation tests included presoaking the percolation test holes with at least 40 to 50 gallons of water in P-1 and about 60 gallons of water in P-2 for a period of 2 hours so that the water flow into the hole held constant at a level of at least 5 times the hole's radius above the bottom of the hole. Testing commenced following the presoak. The sandy soil test method was used. This included making two (2) consecutive measurements to show that at least six (6) inches of water seeped away in less than 25 minutes, and the test method indicates to run the test for an additional hour with measurements taken every ten (10) minutes. During the tests, measurements were taken every 10 minutes for an hour at each percolation test location. Measurements were taken with a precision of 0.25 inches or better. The procedure indicates that the drop that occurs during the final reading is to be used to calculate the percolation rate. As required, the field data included the two (2) 25-minute readings and the readings for an additional hour. The head of the water in the test holes was generally about 27 to 28 inches when refilling the water level.

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

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

5.0 FINDINGS AND RESULTS

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

5.1 Subsurface Profile: The following paragraphs describe the subsurface conditions encountered at the boring locations drilled.

The borings were all drilled in existing asphalt concrete pavement areas. The five (5) borings drilled (borings B-1 through B-3 and P-1 and P-2) encountered approximately 2 to 3 inches of asphalt concrete. No aggregate base was encountered underlying any of the asphalt concrete pavements at the locations cored. The asphalt concrete pavement was underlain by silty sand soils that extended to depths ranging from about 1½ to 13½ feet BSG. The silty sands were underlain by interbedded layers of silty, clayey sands; clayey sands; poorly graded sands with silt and well graded sands with silt that extended to depths of about 8½ to 33½ feet BSG. These layers were generally underlain by poorly graded sands and well graded sands with silt extending to the maximum depth explored, about 60 feet BSG.

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

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

Silty Sands: The silty sands encountered were described as very loose to dense, as determined by standard penetration resistance, N-values, ranging from 3 to 32 blows per foot. The moisture content of the silty sands ranged from 4 to 12 percent. Two (2) relatively undisturbed samples revealed dry densities of 113.6 and 116.0 pounds per cubic foot.

A consolidation test conducted on a silty sand sample collected at depths of about 1 to 2½ feet BSG from boring B-2 indicated low compressibility characteristics (about 2.4 percent consolidation under a load of 8 kips per square foot). Upon inundation, the sample exhibited slight swell potential (about 0.1 percent collapse) when wetted under a load of 0.25 kips per square foot. Another consolidation test conducted on a silty sand sample collected at depths of about 5 to 6½ feet BSG from boring B-2 indicated low compressibility characteristics (about 4.0 percent consolidation under a load of 8 kips per square foot). Upon inundation, the sample exhibited slight collapse potential (about 0.4 percent collapse) when wetted under a load of 0.5 kips per square foot. Direct shear tests conducted on silty sand samples collected from depths of about 1 to 2½ feet BSG and 5 to 6½ feet BSG from boring B-2 indicated internal angles of friction of 33 and 41 degrees with cohesion values of 130 and 220 pounds per square foot, respectively.

Silty, Clayey Sands: The silty, clayey sands encountered were described as medium dense, as determined by an SPT equivalent N-value (estimated by driving a California Modified split barrel sampler) of 28 blows per foot. The moisture content of a sample tested was 6.2 percent. One (1) relatively undisturbed sample revealed a dry density of 126.5 pounds per cubic foot. An Atterberg Limits conducted on a silty, clayey sand sample collected from depths of about 3½ to 5 feet BSG from boring B-1 indicated a liquid limit of 21 and a plasticity index of 6.

Clayey Sands: The clayey sands encountered were described as medium dense, as indicated by standard penetration resistance, N-values, ranging from 14 to 30 blows per foot. The moisture content of the samples tested ranged from about 6 to 11 percent. An Atterberg Limit test conducted on a clayey sand sample collected from depths of about 28½ to 30 feet BSG from boring B-2 indicated a liquid limit of 25 and a plasticity index of 8.

Poorly Graded Sands, Poorly Graded Sands with Silt and Well Graded Sands with Silt: The poorly graded sands, poorly graded sands with silt and well graded sands with silt encountered were described as loose to dense as determined by standard penetration resistance, N-values, ranging from 10 to 49 blows per foot. The moisture content of the samples tested ranged from about 4 to 8 percent. One (1) relatively undisturbed sample of poorly graded sand with silt revealed a dry density of 119.2 pounds per cubic foot.

Resistance-Value (R-value) Test: An R-value test conducted on a near surface sample containing a mixture of some silty sand and mostly clayey sand and collected from depths of about 1 to 5 feet BSG from boring B-3 indicated an R-value of 37.

Chemical Tests: Chemical tests performed on a near surface soil sample resulted in a pH value of 7.5; a minimum resistivity value of 3,100 ohms-centimeter; 0.0021 percent by weight concentration of chlorides; and 0.0026 percent by weight concentration of sulfates.

5.3 **Groundwater Conditions:** Groundwater was not encountered in the test borings drilled at the time of our October 2024 field exploration to the maximum depth explored, about 60 feet BSG.

Based on our review of groundwater data published by the Department of Water Resources, a well located about 1½ miles northwest of the site indicates that groundwater has ranged from an elevation of about 2,808 feet in 1981 to an elevation of about 2,767 feet BSG in 2005 for data collected between the years 1981 and 2017. The most recent measurement from this well in 2017 indicated groundwater at an elevation of about 2,778 feet. Considering the subject site has an average elevation of about 3,259 feet above mean sea level (USGS Topographic Data on Google Earth), groundwater at the site is considered to be greater than 450 feet below site grade.

It should be recognized, however, that groundwater elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

5.4 **<u>Results of Percolation Testing</u>:** The infiltration rate estimated from the percolation test data is summarized in Table No. 1 below. The percolation test data is included in Appendix D of this report.

Table No. 1
Results of Percolation Testing

Location and Depth	**Field (Unfactored) Infiltration Rate (Inches per Hour)[1]**	**Subgrade Soil Type**
P-1 at 20.25 feet BSG	3.3	Dense Poorly Graded Sand with Silt
P-2 at 20.2 feet BSG	4.3	Medium Dense Well Graded Sand with Silt

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

It should be noted that the field tests do not consider the long-term effects of subgrade saturation, silt accumulation, groundwater influence, nor vegetation. In general, the infiltration rate of the soils will decrease when the soils are saturated and the reduction in the infiltration rate increases the longer the soils are saturated. Published studies indicate field infiltration rates can significantly overestimate the saturated permeability. In addition, soil bed consolidation, sediment, suspended soils, etc. in the discharge water can result in clogging of the pore spaces in the soil. This clogging effect can also reduce the long-term infiltration rate. Numerous other factors, such as variations in soil type and soil density across the entire area of the system can influence the infiltration rate, both short and long term.

6.0 EVALUATION

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

6.1 Existing Surface and Subsurface Conditions: At the time of our field exploration, the surface of the site was occupied by various pavements, a building, a canopy, and slabs-on-grade, which are to be demolished. It is possible some of the existing slabs on grade in the southern half of the site include buried foundations where a building was removed in 2017. In addition, the sewer or septic pipe extending vertically out of the ground in the area of the slabs-on-grade in the southern half of the site and an outline of a trench trending northeast away from one of these slabs-on-grade suggest that subsurface septic system(s) (tanks, piping, leach fields, etc.) may be present. A power pole exists in the southeastern portion of the site, and an overhead line was noted as trending to the southeast away from the power pole. Abundant weed growth was noted within the cracked pavements in the southern half of the site and also in some of the cracked pavements in the northern half of the site. A chain link fence surrounded an equipment storage area on the south side of the existing building in the northern half of the site. A chain link fence also surrounded the northern, eastern and southern sides of the southern half of the site. Also, a tree was also noted along the fence and southern boundary of the site in the southwest corner of the site.

It is our understanding that the existing improvements will be demolished and removed as part of the site preparation for the proposed Starbucks development. As a part of demolition, it is recommended to remove all existing surface and subsurface improvements. Further, all utilities not required for the new construction should be entirely removed, and not abandoned in-place. Numerous underground utilities were noted at the site, including site light (electric), water lines, sewer lines, etc, that should be identified and removed during demolition and site preparation. As previously noted, a subsurface septic system(s) (tanks, piping, leach fields, etc.) may be present in the southern portion of the site in the area of the concrete slabs-on-grade left in-place. These surface and subsurface features and undocumented fill soils should be entirely removed to expose native, undisturbed soils; and the resulting excavations backfilled as engineered fill to the finished grades. The power pole in the southeastern portion of the site will also need to be removed from the site.

Deep shaft foundations may support the existing canopy in the northern half of the site. If the existing canopy is supported by deep shaft foundations, the portion of foundations that extend below 5 feet below final grade, and that are not within 5 feet of any utility trench, may remain in place. The portion of the foundations above five feet below grade, or within 5 lateral feet of adjacent excavations, should be cutoff and removed. The resultant excavations should be backfilled as engineered fill to final grades.

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

The near surface loose soils encountered in the borings drilled for the proposed Starbucks building are not considered suitable for direct support of proposed structure. In order to reduce the potential for excessive static settlement of foundations and to limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet, it is recommended to support new foundations for the Starbucks structure on engineered fill soils that extend to either: 1) a depth of 4 feet below preconstruction site grade; or 2) to the depth required to provide at least 1 foot of engineered fill below proposed foundations, whichever is greater. In addition, the over-excavation recommended for the proposed Starbucks building will also need to be conducted to remove all surface and subsurface structure such as the existing building, foundations, underground utilities, etc. All undocumented fill soils and soils disturbed from removal of subsurface improvements will also need to be removed during site preparation for the proposed Starbucks building. Provided the building pad areas are prepared in accordance with the recommendations included in this report, a net allowable soil bearing pressure of 2,500 pounds per square foot, for dead-plus-live loads, may be used for design.

The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and weight of the footing may be neglected. The net allowable soil bearing pressure presented was selected using the Terzaghi bearing capacity equations for foundations considering a minimum factor of safety of 3.0 and based on the anticipated static settlements noted in this report.

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

6.3 Seismic Ground Rupture and Design Parameters: The closest active fault is the Ord Mountain Fault zone (part of the North Front Thrust System), which is located about 6½ miles southeast of the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Accordingly, the potential for ground rupture at the site is considered low.

It is our understanding that the 2022 CBC will be used for structural design, and that seismic site coefficients are needed for design.

Based on the 2022 CBC, a Site Class D represents the on-site soil conditions with standard penetration resistance, N-values averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A table providing the recommended seismic coefficients and earthquake spectral response acceleration values for the project site is included in the Foundation Recommendations section of this report. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.550g was determined for the site using the Seismic Design Maps tool provided by the Structural Engineers Association of California (https://seismicmaps.org/).

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

Seismic settlement analyses were conducted based on soil properties from the boring with the deepest advance (B-2) using the computer program LiquefyPro, developed by CivilTech Software. Also, the depth of engineered fill recommended for site preparation was considered in the analysis. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.550g 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 8.2 was applied in the analysis based on the highest earthquake magnitude determined from probabilistic analysis (hazard deaggregation analysis fro the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/), and deterministic analysis using the Building Seismic Safety Council 2014 (BSSC2014) Scenario Catalog from the USGS website for the Earthquake Hazards Program (https://earthquake.usgs.gov/scenarios/catalog/bssc2014/). Soil parameters, such as wet unit weight, standard penetration test, N-values, and fines content were input from the boring data for the soil layers encountered throughout the depths explored.

Due to the depth to historical groundwater in the vicinity of this site (greater than 450 feet BSG), liquefaction is not considered a concern for the proposed development. However, there is potential for dry seismic settlement to occur during shaking from earthquakes. As part of the analysis, the (N1)60s values of 30 or greater (dense to very dense soils) were not considered to be subject to significant dry seismic settlement in the analyses. Based on the analysis, seismic settlement was estimated to be negligible.

6.5 Asphaltic Concrete (AC) Pavements: Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphaltic concrete (AC) pavements. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highway Design Manual. The analysis was based on traffic index values ranging from 5.0 to 7.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

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

A Resistance-Value (R-value) test was conducted on a near surface sample containing a mixture of some silty sand and mostly clayey sand that was collected from boring B-3 which was drilled in the entrance area for the proposed drive-thru pickup drive lane for the Starbucks. The test indicated an R-value result of 37. R-values of 45 and 51 were determined in the area of the bordering proposed McDonald's parcel during our previous January 2019 investigation at the subject site for the previously planned Circle K development. However, the previous samples tested contained all silty sand material and did not contain any clayey sand material, thus resulting in higher R-values. Based on the result of the current testing, and considering potential variation in the near surface soils, an R-value of 35 was used to provide the pavement section thickness recommendations.

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

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

The pavement sections were prepared based on traffic indexes ranging from 6.0 to 8.0. The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements." A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 4-inch layer of aggregate base material (minimum R-value of 78), of 190 psi/in at the top of the aggregate base was used for pavement design.

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

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

Table No. 2
Soil Resistivity and Corrosion Potential Ratings

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

The results of soil sample analyses indicate that the near-surface soils exhibit a “corrosive” corrosion potential to buried metal objects. This is consistent with our previous 2019 test results at the site on the proposed McDonald’s parcel during Moore Twining’s investigation for the previously planned Circle K development. Appropriate corrosion protection should be provided for buried improvements based on the “corrosive” corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

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

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

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

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

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

7.0 CONCLUSIONS

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

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

7.2 The subsurface soils encountered generally consisted of very loose to medium dense silty sands extending to depths of about 1½ to 3½ feet across the site. Below the very loose to loose silty sands, the relative density of the silty sands soils improved to medium dense to dense and extended to depths of about 3½ to 13½ feet BSG. Below the silty sands, medium dense silty, clayey sands; medium dense clayey sands; medium dense to dense poorly graded sands with silt; and medium dense well graded sands with silt were encountered extending to a depth of about 33½ feet BSG which were generally underlain by dense poorly graded sands and dense well graded sands with silt extending to the maximum depth explored of 60 feet BSG.

7.3 Laboratory testing on the near surface soils indicate the materials are non-plastic, non-expansive, and exhibit low compressibility characteristics, slight collapse potential, and moderate to high shear strength properties. The near surface soils exhibit fair support characteristics for pavements when compacted as engineered fill.

7.4 Groundwater was not encountered in the test borings drilled at the time of our October 2024 investigation to the maximum depth explored, about 60 feet BSG. Based on groundwater data published by the Department of Water Resources, the depth to groundwater at the site is considered to be greater than 450 feet below site grade.

7.5 Due to the depth to historical groundwater in the vicinity of this site (greater than 450 feet BSG), liquefaction is not considered a concern for the proposed development. However, there is potential for dry seismic settlement to occur during shaking from earthquakes. As part of the analysis, the (N1)60s values of 30 or greater (dense to very dense soils) were not considered to be subject to significant dry seismic settlement in the analyses. Based on the analysis, seismic settlement was estimated to be negligible.

7.6 The result at percolation test P-1 at 20.25 feet BSG indicated an unfactored infiltration rate of 3.3 inches per hour. The result at percolation test P-2 at 20 feet BSG indicated an unfactored infiltration rate of 4.3 inches per hour. The results indicate that storm water infiltration systems at a depth of 20 feet BSG appear feasible for this site. This report recommends that the lower unfactored infiltration rate of 3.3 inches per hour be considered for use in design for infiltration systems at a depth of 20 feet BSG when including an appropriate factor of safety. Appendix D, Section VII (Technical Guidance Document Appendices) of Technical Guidance Document for Water Quality Management Plans, dated June 7, 2013, prepared by CDM Smith Inc. for the County of San Bernardino Areawide Stormwater Program discusses the factor of safety to be used to be used for design of infiltration facilities. Appendix D, Section VII.4 'Considerations for Infiltration Rate Factor of Safety' indicates, "The factor of safety used to compute the *design infiltration rate* shall not be less than 2.0 but may be higher at the discretion of the design engineer and acceptance of the plan reviewer...."

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

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

7.9 The potential for fault rupture on the site is low.

7.10 It is our understanding that the existing improvements will be demolished as part of the site preparation for the proposed Starbucks development. To provide adequate support for the planned building and pavement improvements, existing surface and subsurface improvements not required for the new construction should be entirely removed, and not abandoned in-place. Numerous underground utilities were noted at the site, including site light (electric), water lines, sewer lines, etc, that should be identified and removed during demolition and site preparation. A subsurface septic system(s) (tanks, piping, leach fields, etc.) may also be present in the southern portion of the site in the area of the concrete slabs-on-grade left in-place. These surface and subsurface features and undocumented fill soils should be entirely removed to expose native, undisturbed soils; and the resulting excavations backfilled as engineered fill to the finished grades.

In order to reduce the potential for excessive static settlement of foundations and to limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet, it is recommended to support new foundations for the Starbucks structure on engineered fill soils that extend to either: 1) a depth of 4 feet below preconstruction site grade; or 2) to the depth required to provide at least 1 foot of engineered fill below proposed foundations, whichever is greater.

7.11 If the existing canopy is supported by deep shaft foundations, the portion of foundations that extend below 5 feet below final grade, and that are not within 5 feet of any utility trench, may remain in place. The portion of the foundations above five feet below grade, or within 5 lateral feet of adjacent excavations, should be cutoff and removed. The resultant excavations should be backfilled as engineered fill to final grades.

8.0 RECOMMENDATIONS

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

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

8.1 General

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

8.1.2 When the actual foundation loads are known, this information should be provided to Moore Twining for review to confirm the recommendations for site preparation are appropriate. In the event the foundation loads are different than assumed, the recommendations in this report may need to be revised.

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

8.1.4 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 owner in writing prior to bidding the project that the data provided in this report is not sufficient to bid the project. This notification should be specific and explain in detail as to what data are not sufficient.

8.2 Site Grading and Drainage

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

8.2.2 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from the proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

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

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

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

8.2.6 Stormwater systems that allow wetting of the soils should not be placed directly adjacent to structures or foundations. On a preliminary basis, these types of features should be setback at least 20 feet from foundations. The result at percolation test P-1 at 20.25 feet BSG indicated an unfactored infiltration rate of 3.3 inches per hour. The result at percolation test P-2 at 20 feet BSG indicated an unfactored infiltration rate of 4.3 inches per hour. The results indicate that storm water infiltration systems at a depth of 20 feet BSG appear feasible for this site for infiltration systems such as deeper dry wells. This report recommends that the lower unfactored infiltration rate of 3.3 inches per hour be considered for use in design for infiltration systems at a depth of 20 feet BSG when including an appropriate factor of safety. Shallow infiltration systems should not be considered based on the results of previous percolation testing conducted by Moore Twining at the site in 2019 that identified cemented soils and unfavorable infiltration rates in the near surface soils. Appendix D, Section VII (Technical Guidance Document Appendices) of Technical Guidance Document for Water Quality Management Plans, dated June 7, 2013, prepared by CDM Smith Inc. for the County of San Bernardino Areawide Stormwater Program discusses the factor of safety to be used for design of infiltration facilities. Appendix D, Section VII.4 'Considerations for Infiltration Rate Factor of Safety' indicates, "The factor of safety used to compute the *design infiltration rate* shall not be less than 2.0 but may be higher at the discretion of the design engineer and acceptance of the plan reviewer...."

8.3 <u>Site Preparation</u>

8.3.1 Stripping should be conducted in all areas of existing landscaping to remove surface vegetation and root systems (if any). The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. A tree occupied the southwest corner of the site. Tree roots that are encountered during site grading should be excavated to remove all roots larger than ¼-inch or accumulation of organics greater than 3 percent by dry weight.

8.3.2 As part of the site preparation, existing surface and subsurface improvements should be completely removed. Existing subsurface improvements and associated backfill soils should be excavated to at least 12 inches below the improvements removed, to the depth required to remove all disturbed soils, and all fill materials, whichever requires the deeper excavation. Underground utilities to be removed should not be capped and abandoned or crushed and buried in-place. Instead, underground utilities not scheduled to remain should be fully removed from the site along with the associated trench backfill soils that should be assumed to extend at a 1 horizonal to 1 vertical gradient extending from the bottom of the utility to the ground surface. Excavated soils associated with removal of utilities and other subsurface improvements should be moisture conditioned and compacted as engineered fill as recommended in this report.

8.3.3 For the deep shaft foundations that may support the existing canopy, the portion of foundations that extend below 5 feet below final grade, and that are not within 5 feet of any utility trench, may remain in place. The portion of the foundations above five feet below grade, or within 5 lateral feet of adjacent excavations, should be cutoff and removed. The resultant excavations should be backfilled as engineered fill to final grades.

8.3.4 The fill soils used to backfill the USTs (removed in 1998 from the western portion and southwest side of the existing canopy) were documented to be compacted as engineered fill. When the area of this certified fill is exposed, Moore Twining should observe it and probe it to determine if any unsuitable or loose soils are exposed during the over-excavations for the proposed building and other site improvements. If any unsuitable or loose soils are exposed, these soils should be removed, moisture conditioned as necessary and compacted as engineered fill.

8.3.5 After stripping and removal of existing surface and subsurface improvements, the building pad areas for the proposed Starbucks and over-build zone should be over-excavated to the depths required to meet all of the following requirements, whichever requires the deeper excavation:

1) to at least 1 foot below the bottom of footings,
2) to at least 4 feet below preconstruction site grades,
3) to the depth required to remove existing undocumented fill soils, and
4) to at least 12 inches below the subsurface improvements (structures, utilities, etc.) to be removed.

The horizontal limits of over-excavation should include the footprint of the building, all foundations, all concrete walkways adjacent to the structures, and a minimum of 5 feet beyond these features, whichever is greater. Upon review of the Contractor's survey data (regarding the vertical and horizontal limits of the over-excavation) and approval of the over-excavation by Moore Twining, the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill.

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

8.3.7 Following stripping and removal of surface and subsurface improvements, areas to receive miscellaneous lightly loaded foundations, such as site walls, retaining walls or screen walls for trash enclosures, should be over-excavated to a minimum of 1 foot below foundations, to a depth of at least 4 feet below preconstruction site grades, to the depth required to remove undocumented fills, or to at least 12 inches below subsurface improvements (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 edges of the foundations or up to improvements to remain, whichever occurs first. The bottom of the over-excavation should be scarified to a depth of at least 8 inches, moisture conditioned and compacted as engineered fill.

8.3.8 Following stripping and removal of surface and subsurface improvements, areas to receive new pavements, exterior slabs on grade outside the building pad preparation limits and areas to receive fill outside the building pad preparation limits should be over-excavated to a depth of 12 inches below pre-construction pavement grades, to the depth required to remove undocumented fill soils, to a depth of 12 inches below the bottom of the new aggregate base section, to at least 12 inches below subsurface improvements (utilities, etc.) to be removed, and to the depth required to remove all disturbed soils, whichever is greater. The exposed surface after over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill. The limits of scarification for pavement areas and exterior slabs should extend at least 3 feet beyond the edge of these improvements or up to improvements to remain, whichever occurs first. The upper 12 inches of the subgrade soils beneath the pavement areas should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.3.9 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill. Refer to Section 8.4.5 of this report for the moisture content range and minimum percent relative compaction recommendations for engineered fill.

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

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

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

8.3.13 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal.

8.4 Engineered Fill

8.4.1 The on-site near surface soils encountered are predominantly silty sands; silty, clayey sands; and clayey sands. The on-site soils will be suitable for use as engineered fill below the recommended aggregate base section, provided they are free of organics (less than 3 percent by weight and no roots larger than ¼ inch in diameter), irreducible material greater than 3 inches, have an expansion index of less than 20 and the moisture content of the soil is within optimum to three (3) percent above optimum moisture content at the time of placement. This report recommends that interior and exterior slabs-on-grade be underlain by at least 4 inches of aggregate base. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.4.2 If materials larger than 3 inches are encountered in the excavated material, the oversize rock should be removed prior to use as engineered fill (mar require hand picking).

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

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

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 15
Organics	Less than 3 percent by weight
R-Value	Minimum 35*
Sulfates	< 0.05 percent by weight
Min. Resistivity	>5,000 ohms-cm

* for pavement areas only

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. In lieu of sampling and testing aggregate base materials (or bedding sand) from virgin sand and gravel sources, a letter stating that the aggregate base (or bedding sand) comprises materials entirely from natural (virgin) sources and that the aggregate base (or bedding sand) is non-contaminated may be provided by the Contractor. Moore Twining will sample and test the material after the environmental certification submittal is approved to verify that the proposed material complies with the geotechnical engineering recommendations of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested for the geotechnical properties.

8.4.5 Native and imported engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum moisture content and three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of fill and subgrade compacted in pavement areas should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

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

Table No. 4

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

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

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

8.5 <u>Shallow Spread Foundations</u>

8.5.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on the estimated settlements. The following static settlements should be anticipated for design: 1) a total static settlement of 1 inch; and 2) a differential static settlement of ½-inch in 40 feet.

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

8.5.3 Perimeter foundations should have a minimum depth of 18 inches below the lowest adjacent finished exterior ground surface. Interior footings should have a minimum depth of at least 12 inches below the bottom of the slab-on-grade. All footings should have a minimum width of 15 inches, regardless of load.

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

8.5.5 The following seismic factors were developed using online data obtained from the Ground Motion Parameter Calculator provided by the Structural Engineers Association of California website (https://seismicmaps.org/) based upon a latitude of 34.423305 degrees and a longitude of -117.316065 degrees and a Site Class D. The data provided in Table No. 5 are based upon the procedures of the 2022 California Building Code and were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 5 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

Table No. 5
Seismic Factors

Seismic Factor	2022 CBC Value*
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.550g
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-10 (PGA)	0.500g
Spectral Response At Short Period (0.2 Second), Ss	1.415
Spectral Response At 1-Second Period, S_1	0.547
Site Coefficient (based on Spectral Response At Short Period), Fa	1.0

Seismic Factor	2022 CBC Value*
Site Coefficient (based on spectral response at 1-second period) Fv	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.415
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note
Five percent damped design spectral response accelerations for short period, S_{DS}	0.944
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	See Note

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

*The above data is subject to the disclaimers listed in the website https://seismicmaps.org/

8.5.6 All loose soils should be removed from foundation excavations and the excavations should be maintained at near optimum moisture content by periodic wetting. Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement.

8.5.7 Structural loads for lightly loaded (less than 1.5 kips per lineal foot) miscellaneous foundations (such as screen walls for the proposed trash enclosures) may be supported engineered fills prepared in accordance with the recommendations included in the Site Preparation section of this report. The lightly loaded foundations should extend to a minimum depth of 12 inches below the lowest adjacent grade and a minimum width of 12 inches, regardless of load. Footings for miscellaneous lightly loaded foundations may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

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

8.5.9 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot. The upper 6 inches of subgrade in landscaped areas should be neglected in determining the total passive resistance.

8.6 Interior Slabs-on-Grade

8.6.1 Interior slabs-on-grade should be supported over 4 inches of non-recycled aggregate base over engineered fill extending to the depth recommended below foundations in the Site Preparation section of this report.

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

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

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

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

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

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

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

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

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

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

8.6.11 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

8.7 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic. They are intended for pedestrian traffic areas.

8.7.1 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.

8.7.2 Exterior slabs within the building pad preparation limits and exterior slabs outside the building pad preparation limits should be supported on 4 inches of aggregate base overlying subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report.

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

8.7.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the aggregate base. This should reduce the potential for infiltration of water into the aggregate base below exterior slabs.

8.7.5 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.

8.8 Asphaltic Concrete (AC) Pavements

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

8.8.2 The following pavement sections are based on an R-value of 35 and traffic index values ranging from 5.0 to 7.0. A minimum of 3 inches of asphalt concrete is recommended below for the pavement sections. It should be noted that if pavements are constructed prior to construction of the buildings, the traffic index value should account for construction traffic. The actual traffic index values applicable to the site should be determined by the project civil engineer.

Table No. 6
Two-Layer Asphalt Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	4.5	12
5.5	3.0	6.0	12
6.0	3.5	6.5	12
6.5	3.5	8.0	12
7.0	4.0	8.5	12
7.5	4.0	9.5	12
8.0	4.5	10.0	12

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

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

8.8.4 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.

8.8.5 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.

8.8.6 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.

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

8.8.8 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.

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

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

8.9 Portland Cement Concrete (PCC) Pavements

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

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

8.9.2 The following preliminary Portland cement concrete pavement sections have been prepared for Traffic Indices Ranging from 6.0 to 8.0. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic.

Table No. 7
Portland Cement Concrete Pavements

Traffic Index	Average Daily Truck Traffic (ADTT)	PCC thickness (inches)	Aggregate Base (inches)	Compacted Subgrade (inches)
6.0	2.0	6.0	4.0	12.0
7.0	7.3	6.0	4.0	12.0
8.0	22.2	6.5	4.0	12.0

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

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

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

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

8.9.6 Specifications for the concrete mixtures used in the PCC pavement to reduce the effects of excessive shrinkage (such as curling and excessive shrinkage at joints), including maximum water requirements for the concrete mix, allowable shrinkage limits, curing methods, etc. should be provided by the designer/detailer of the PCC slabs. In addition, as noted in Section 8.9.5, contraction joint requirements should be detailed by the designer/detailer of the PCC pavement to maintain stability. The minimum PCC thickness noted in this report assumes aggregate interlock occurs at contraction joints. However, curling and excessive shrinkage can disengage aggregate interlock and allow greater pavement deflection at free edges.

8.9.7 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.

8.9.8 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.

8.9.9 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.

8.9.10 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.

8.9.11 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.

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

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

8.10 Slopes, Shoring and Temporary Excavations

8.10.1 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.

8.10.2 Due to the low cohesion of the onsite soils, temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 2:1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be shored.

8.10.3 In no case should excavations extend below a 2H to 1V zone below existing roadways, 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.

8.10.4 All soils disturbed as part of the shoring removal shall be over-excavated and compacted as engineered fill. In addition, all cavities and void space resulting from the shoring removal activity shall be backfilled with a cementitious grout under pressure to backfill the voids created by removal of the shoring. All voids resulting from removal of shoring shall be backfilled.

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

8.11 Utility Trenches

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

8.11.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted to a minimum of 92 percent relative compaction, except the upper 12 inches of trench backfill in pavement areas should be compacted to a minimum of 95 percent relative compaction. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.11.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in table below (minimum manufacturer requirements), or to a minimum of 24 inches, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 8
Minimum Trench Widths for HDPE Pipe with
Sand Bedding Initial Backfill

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

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

8.11.5 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned to between optimum and three (3) percent above the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, except the upper 12 inches of trench backfill in pavement areas should be compacted to a minimum of 95 percent relative compaction. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

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

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

8.11.8 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.

8.11.9 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.

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

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

8.12 Corrosion Protection

8.12.1 Based on National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 1 and the analytical results of sample analyses indicate the one sample tested had a resistivity value of 3,100 ohms-centimeter. This is consistent with data for two samples that had a resistivity values of 4,269 and 4,402 ohms-centimeter that were previously tested in 2019 on the proposed adjacent McDonald's parcel during Moore Twining's investigation for the previously planned Circle K development. Based on the resistivity values, the soils exhibit a "corrosive" corrosion potential. Therefore, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.12.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils of 0.0026 percent by dry weight. 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 of 0.52 or less is recommended for slabs on grade as recommended in the "Interior Slab on Grade" section of this report.

8.12.3 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, pavements 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 afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least $3,000,000 and review this report. After their review, the firm should, in writing, state that they understand 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 Section 3.3, Anticipated Construction. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

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

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

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

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

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

Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Allen H. Harker, CEG
Certified Engineering Geologist

CERTIFIED ENGINEERING GEOLOGIST
ALLEN H. HARKER
No. 2781
EXP. 7-31-26
STATE OF CALIFORNIA

Harry D. Moore, RGE
President

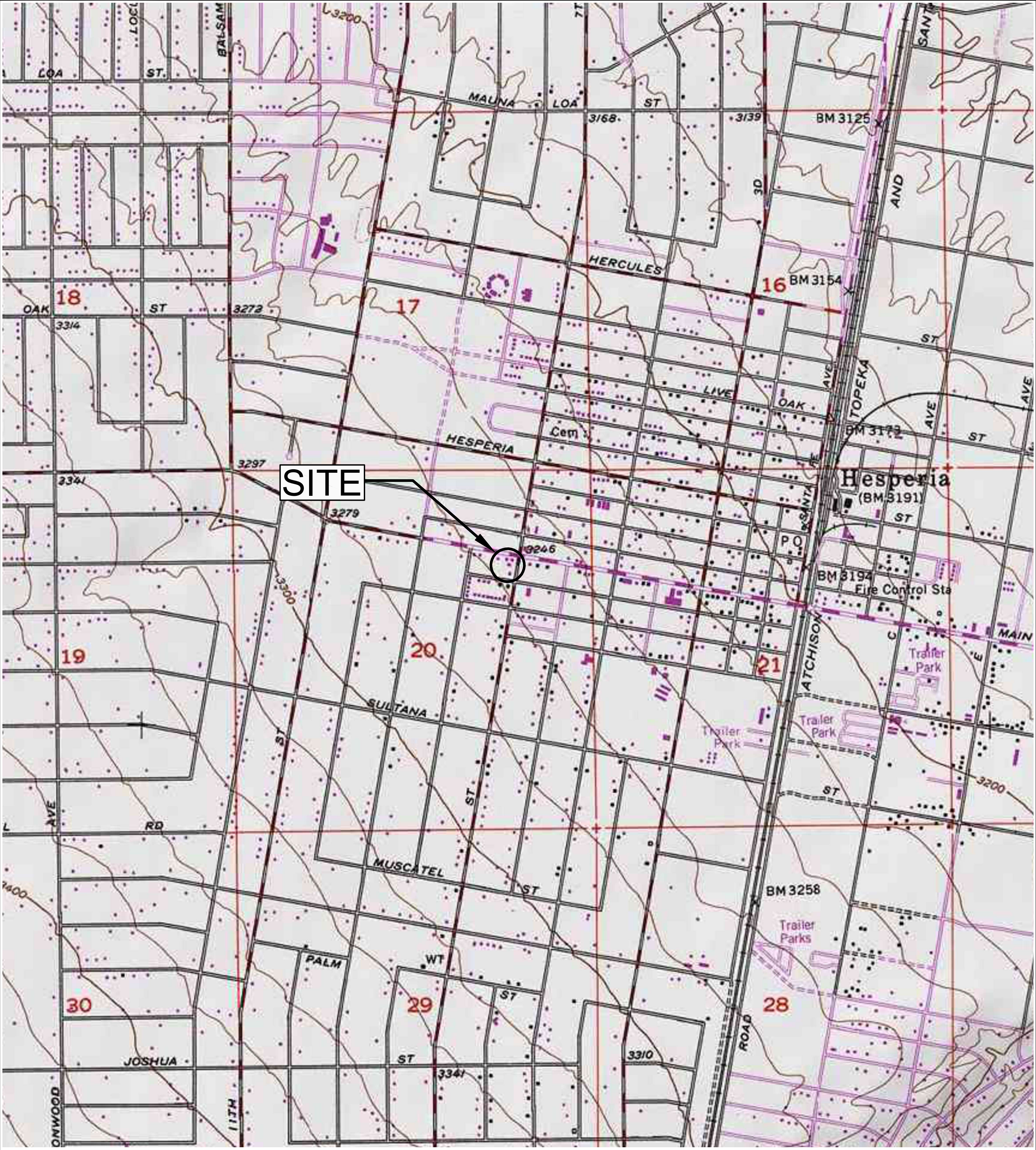
PROFESSIONAL ENGINEER
HARRY D. MOORE
GE 2069
EXP. 09/30/2025
GEOTECHNICAL
STATE OF CALIFORNIA

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map

Drawing No. 2 - Test Boring Location Map

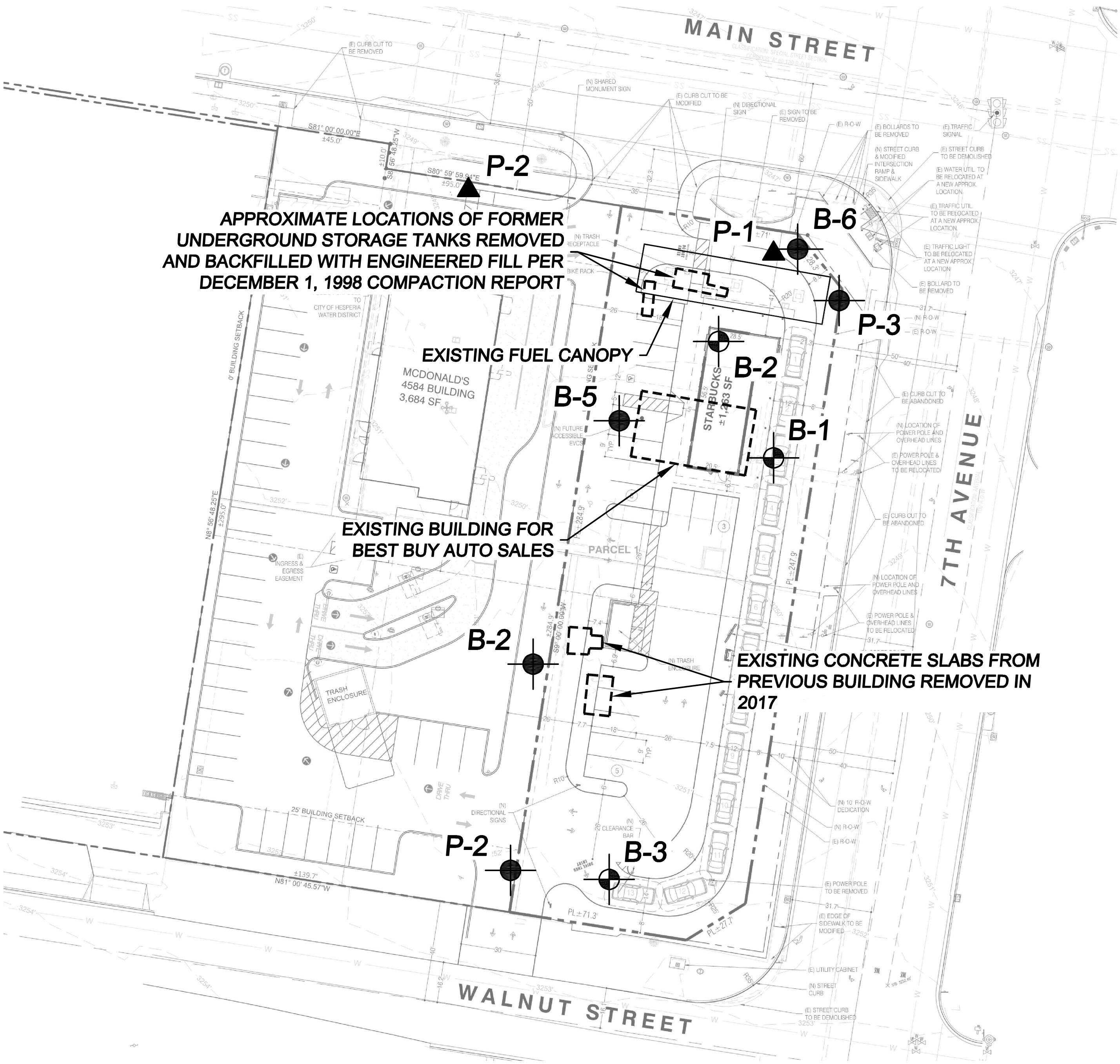


SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
HESPERIA, CALIFORNIA QUADRANGLE 1978, PHOTOREVISED 1980

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP PROPOSED STARBUCKS SOUTHWEST CORNER OF MAIN STREET AND 7TH AVENUE HESPERIA, CALIFORNIA	FILE NO.: 33201-01-01 DRAWN BY: RM PROJECT NO. H33201.01	DATE: 11/08/2024 APPROVED BY: DRAWING NO. 1

MOORE TWINING ASSOCIATES, INC.



APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

APPROXIMATE LOCATIONS OF PREVIOUS BORINGS DRILLED BY MOORE TWINING IN 2018 WITHIN OR NEAR THE STARBUCKS PARCEL

N

0 60

APPROXIMATE SCALE IN FEET

TEST BORING AND PERCOLATION TEST BORING LOCATION MAP PROPOSED STARBUCKS SOUTHWEST CORNER OF MAIN STREET AND 7TH AVENUE HESPERIA, CALIFORNIA	FILE NO. 33201-01-02	DATE DRAWN: 11/08/2024	MOORE TWINING ASSOCIATES, INC.
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. H33201.01	DRAWING NO. 2	

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of borings. These logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

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

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

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-1

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 2.5 inches			
	1/6 2/6 2/6	SM	SILTY SAND; loose, moist, fine to medium grained, brown, trace gravel		4	5.0
	10/6 18/6 25/6	SC-SM	SILTY, CLAYEY SAND; medium dense, moist, fine to coarse grained, brown, with a little fine gravel	From 3.5-5': DD = 126.5 pcf Gravel = 5.2% Sand = 74.4% -200 = 20.4% LL = 21 PI = 6	43	6.2
5	20/6 20/6 21/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, moist, fine to coarse grained, brown, with a little fine gravel	From 5-6.5': DD = 119.2 pcf	41	6.0
10	5/6 5/6 5/6		At 8.5 feet - Loose, light brown	From 8.5-10': Gravel = 5.4% Sand = 84.1% -200 = 10.5% LL = Non-viscous PI = Non-plastic	10	
15	8/6 10/6 10/6	SP	POORLY GRADED SAND; medium dense, moist, fine to coarse grained, light brown, trace fine gravel		20	
			Bottom of Boring B-1 at 15 feet			
20						
25						

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-2

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 2 inches			
	5/6 5/6 6/6	SM	SILTY SAND; loose, damp, fine to medium grained, brown, trace subangular gravel	From 0.2-5': pH = 7.5 SR = 3,100 ohm-cm Cl = 0.0021% SS = 0.0026%	11	3.6
	3/6 2/6 3/6				5	
5	7/6 13/6 20/6		Medium dense, moist Fine to coarse grained, with trace clay	From 1-2.5': DD = 113.6 pcf ø = 33° c = 130 psf	33	6.0 7.1
10	6/6 7/6 6/6			From 5-6.5': DD = 116.0 pcf ø = 41° c = 220 psf	13	
15	5/6 9/6 18/6	SC	CLAYEY SAND; medium dense, moist, fine to medium grained, brown, trace coarse gravel		27	6.5
20	13/6 16/6 18/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, moist, fine to coarse grained, brown		34	
25	17/6 17/6 20/6				37	7.8
	10/6 15/6	SC	CLAYEY SAND; medium dense,	From 28.5-30': Gravel = 5.1%	25	10.9

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: B-2

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30	10/6		moist, fine to coarse grained, brown, with a little fine gravel	Sand = 65.2% -200 = 29.7% LL = 25 PI = 8		
	11/6 15/6 17/6	SP	POORLY GRADED SAND; dense, moist, fine to coarse grained, light brown		32	6.8
35						
	12/6 16/6 22/6				38	
40						
	7/6 14/6 18/6	SM	SILTY SAND; dense, moist, fine grained, brown		32	12.1
		SW-SM	WELL GRADED SAND WITH SILT; dense, damp, fine to coarse grained, brown, with some fine gravel			4.4
45						
	11/6 19/6 20/6			From 48.5-50': Gravel = 11.6% Sand = 79.7% -200 = 8.7%	39	
50						
	15/6 19/6 30/6		Increase in coarse sand content		49	
55						
	17/6 18/6				43	

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
60	25/6		Bottom of Boring B-2 at 60 feet			
65						
70						
75						
80						
85						

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 3 inches			
	5/6 8/6 8/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown	From 1-5': R-value = 37	16	7.8
	13/6 15/6 15/6	SC	At 1.5 feet - CLAYEY SAND; medium dense, moist, fine to coarse grained, brown, trace fine gravel At 3.5 feet - Weakly cemented		30	
5						
	6/6 9/6 11/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, moist, fine to coarse grained, brown, trace fine gravel		20	
10						
	8/6 12/6 13/6	SP	POORLY GRADED SAND; medium dense, moist, fine to coarse grained, light brown		25	
15			Bottom of Boring B-3 at 15 feet			
20						
25						

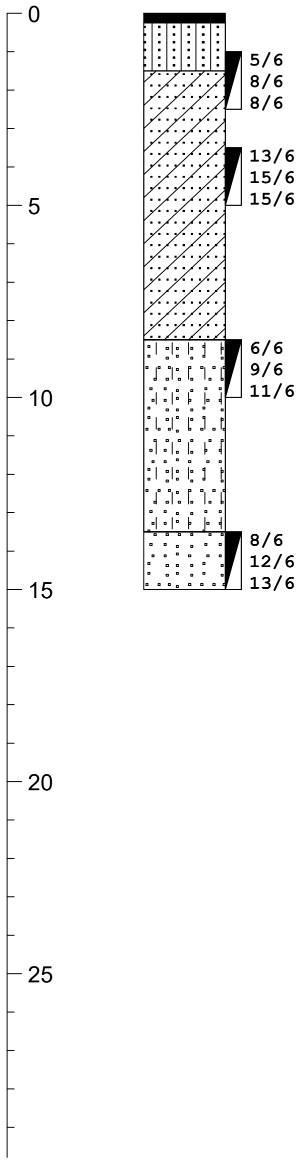


Figure Number 3

MOORE TWINING ASSOCIATES, INC.
Since 1898

Test Boring: P-1

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

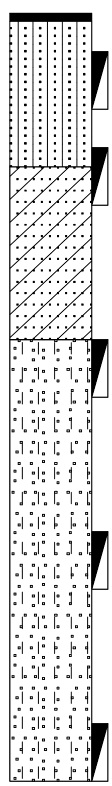
Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 2.5 inches			
	1/6 1/6 2/6	SM	SILTY SAND; very loose, damp, fine to coarse grained, brown, trace gravel		3	4.2
	3/6 7/6 7/6		Medium dense		14	8.1
5		SC	CLAYEY SAND; medium dense, moist, fine to coarse grained, brown, trace gravel			
10	3/6 7/6 7/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, moist, fine to coarse grained, brown, with trace fine gravel		14	5.5
15	6/6 6/6 9/6				15	
	13/6 14/6 18/6		Dense, slight decrease in fines content	From 18.5-20': Gravel = 3.9% Sand = 84.9% -200 = 11.2% LL = Non-viscous PI = Non-plastic	32	5.3
20			Bottom of Percolation Test Boring P-1 at 20 feet			
25						

Figure Number 4

MOORE TWINING ASSOCIATES, INC.

Since 1898

Test Boring: P-2

Project: Proposed Starbucks in Hesperia

Project Number: H33201.01

Drilled By: 2R Drilling

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: A.V.

Date: October 23, 2024

Elevation: N/A

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphalt Concrete = 3 inches			
	11/6 11/6 11/6	SM	SILTY SAND; medium dense, moist, fine to medium grained, brown		22	6.0
	14/6 17/6 17/6	SC	At 2 feet - CLAYEY SAND; medium dense, moist, fine to coarse grained, brown, trace fine gravel At 3.5 feet - Dense, with weak to moderate cementation		34	6.1
5						
	3/6 7/6 11/6	SP-SM	POORLY GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, brown, trace fine gravel and clay		18	3.5
10						
	9/6 10/6 12/6				22	
15						
	9/6 13/6 16/6	SW-SM	WELL GRADED SAND WITH SILT; medium dense, damp, fine to coarse grained, brown, with fine a little fine gravel	From 18.5-20': Gravel = 8.1% Sand = 82.0% -200 = 9.9%	29	4.1
20			Bottom of Percolation Test Boring P-2 at 20 feet			
25						

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Asphalt Concrete
	SM: Silty sand
	SC-SM: Silty, Clayey Sand
	SP-SM: Poorly graded sand with silt
	SP: Poorly graded sand
	SC: Clayey sand
	SW-SM: Well graded sand with silt

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

Notes:

1. Exploratory borings were drilled on 10/23/24 using a CME 75 drill rig equpped with 8" outside diameter hollow stem augers.

2. Groundwater was not encountered during drilling of the borings.

3. Boring locations were measured or paced from existing site 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. Abbreviations used are:

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

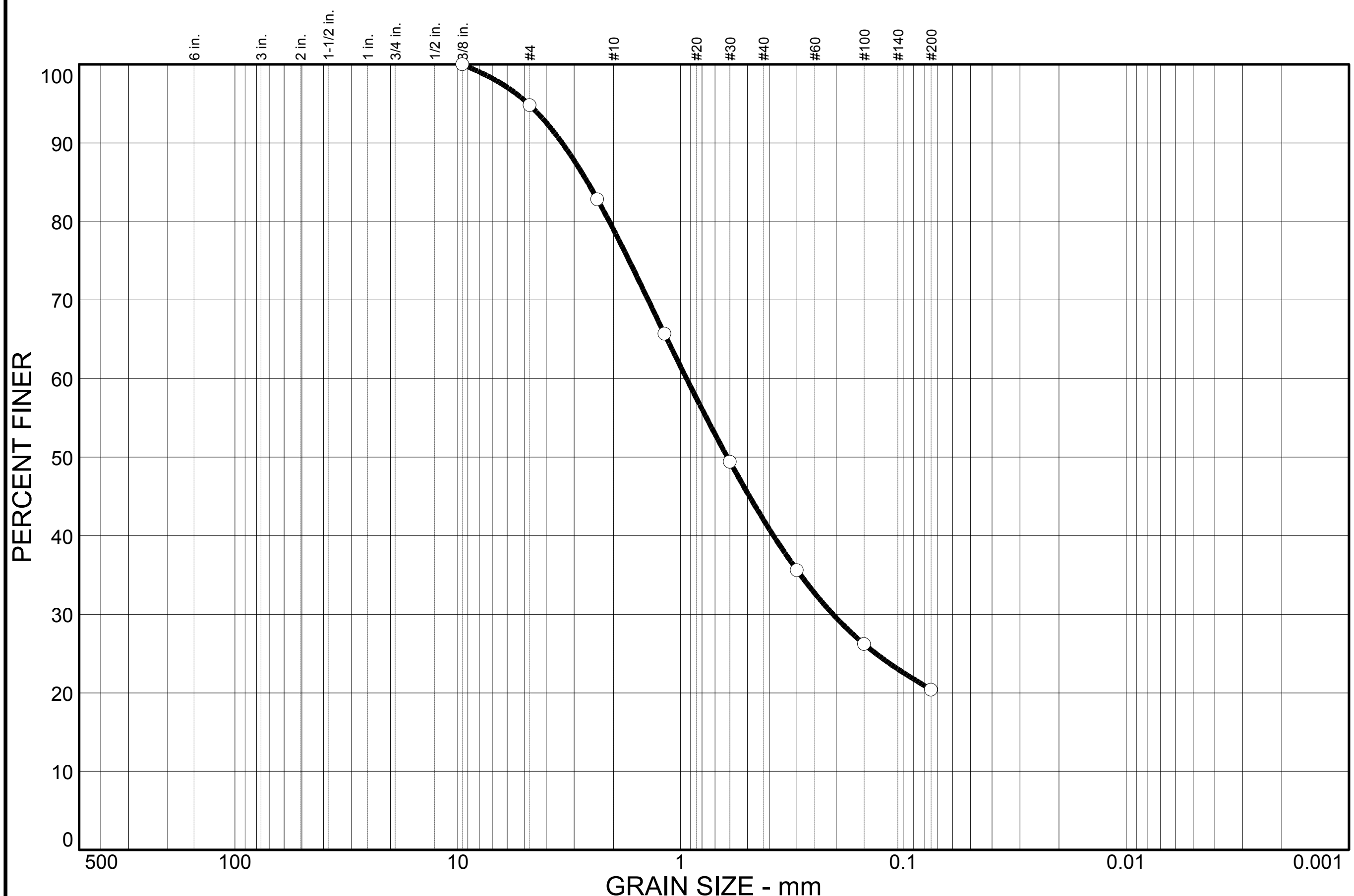
APPENDIX C

RESULTS OF LABORATORY TESTS

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

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

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.2	15.8	36.9	21.7	20.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	94.8		
#8	82.8		
#16	65.7		
#30	49.4		
#50	35.6		
#100	26.2		
#200	20.4		

* (no specification provided)

Material Description

Silty, clayey sand

Atterberg Limits

PL= 15 LL= 21 PI= 6

Coefficients

D_{85}= 2.61 D_{60}= 0.940 D_{50}= 0.616
D_{30}= 0.207 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC-SM AASHTO=

Remarks

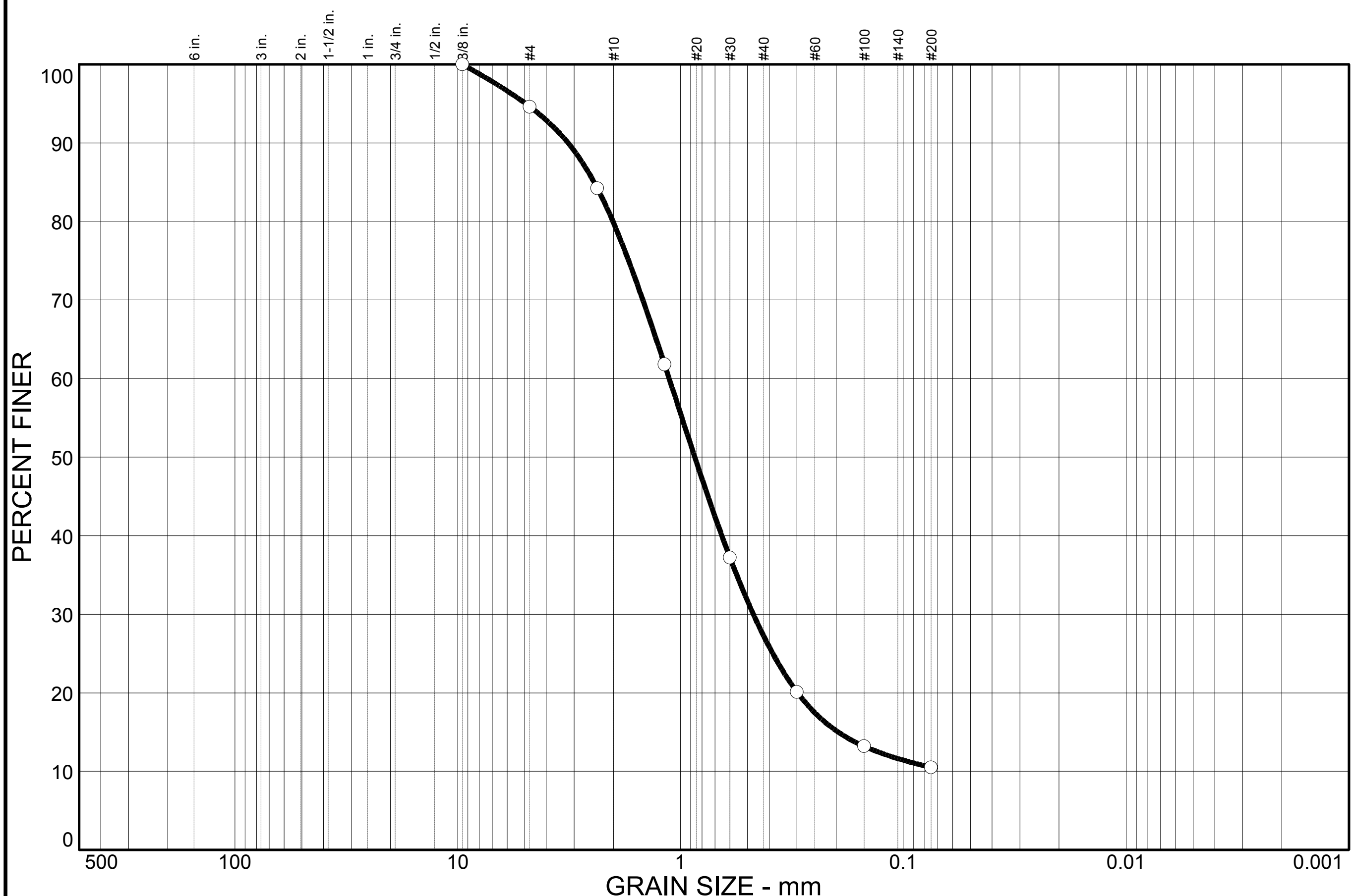
Sample No.: B-1 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 3.5-5'

Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks
Project No: H33201.01 **Figure** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.4	14.8	52.5	16.8	10.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	94.6		
#8	84.2		
#16	61.8		
#30	37.2		
#50	20.1		
#100	13.2		
#200	10.5		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 2.44 D_{60}= 1.12 D_{50}= 0.863
D_{30}= 0.471 D_{15}= 0.196 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 8.5-10'

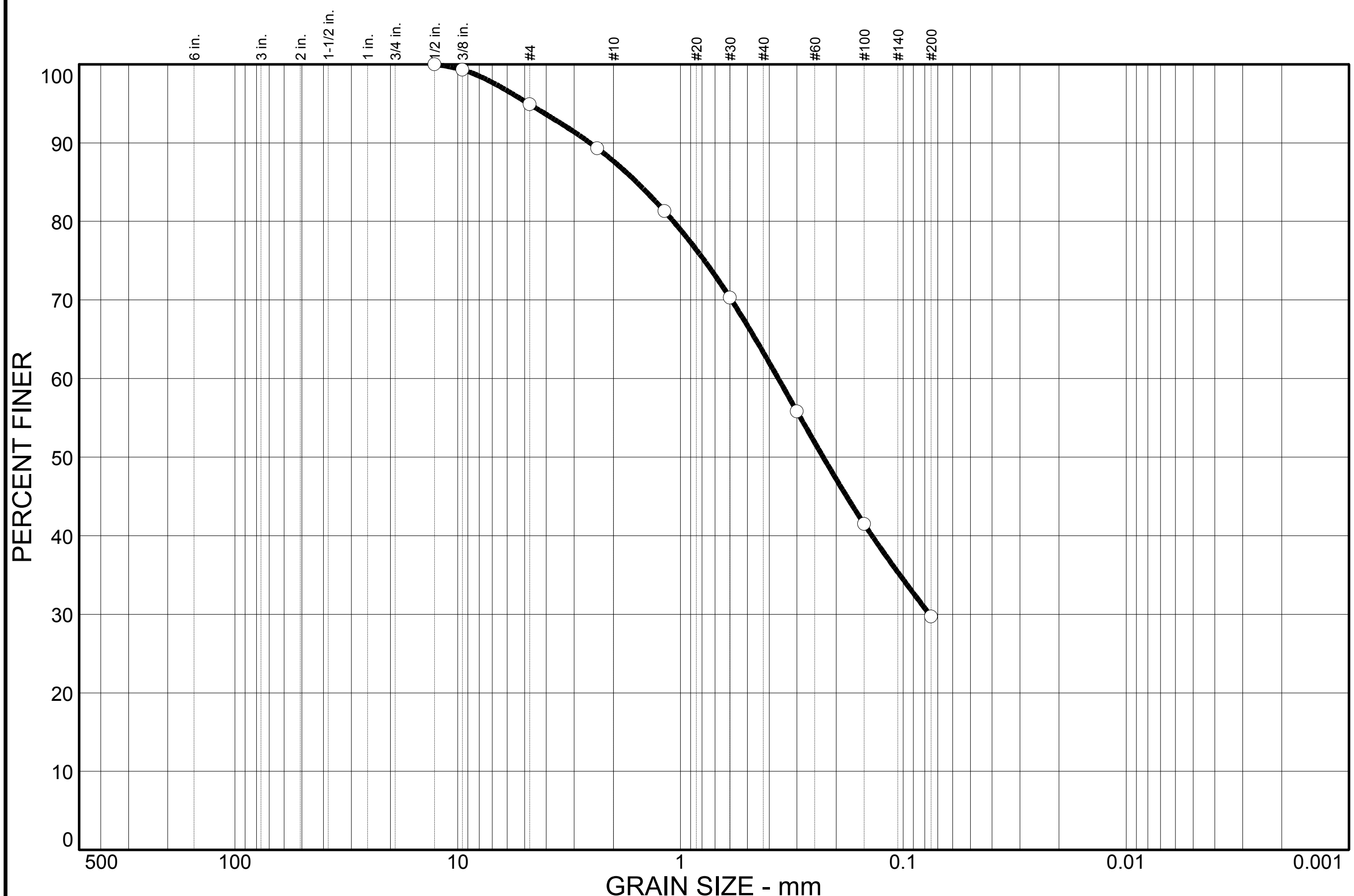
Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks

Project No: H33201.01 **Figure** 2

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.1	7.2	24.3	33.7	29.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.3		
#4	94.9		
#8	89.3		
#16	81.3		
#30	70.3		
#50	55.8		
#100	41.5		
#200	29.7		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= 17 LL= 25 PI= 8

Coefficients

D_{85}= 1.57 D_{60}= 0.364 D_{50}= 0.229
D_{30}= 0.0764 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

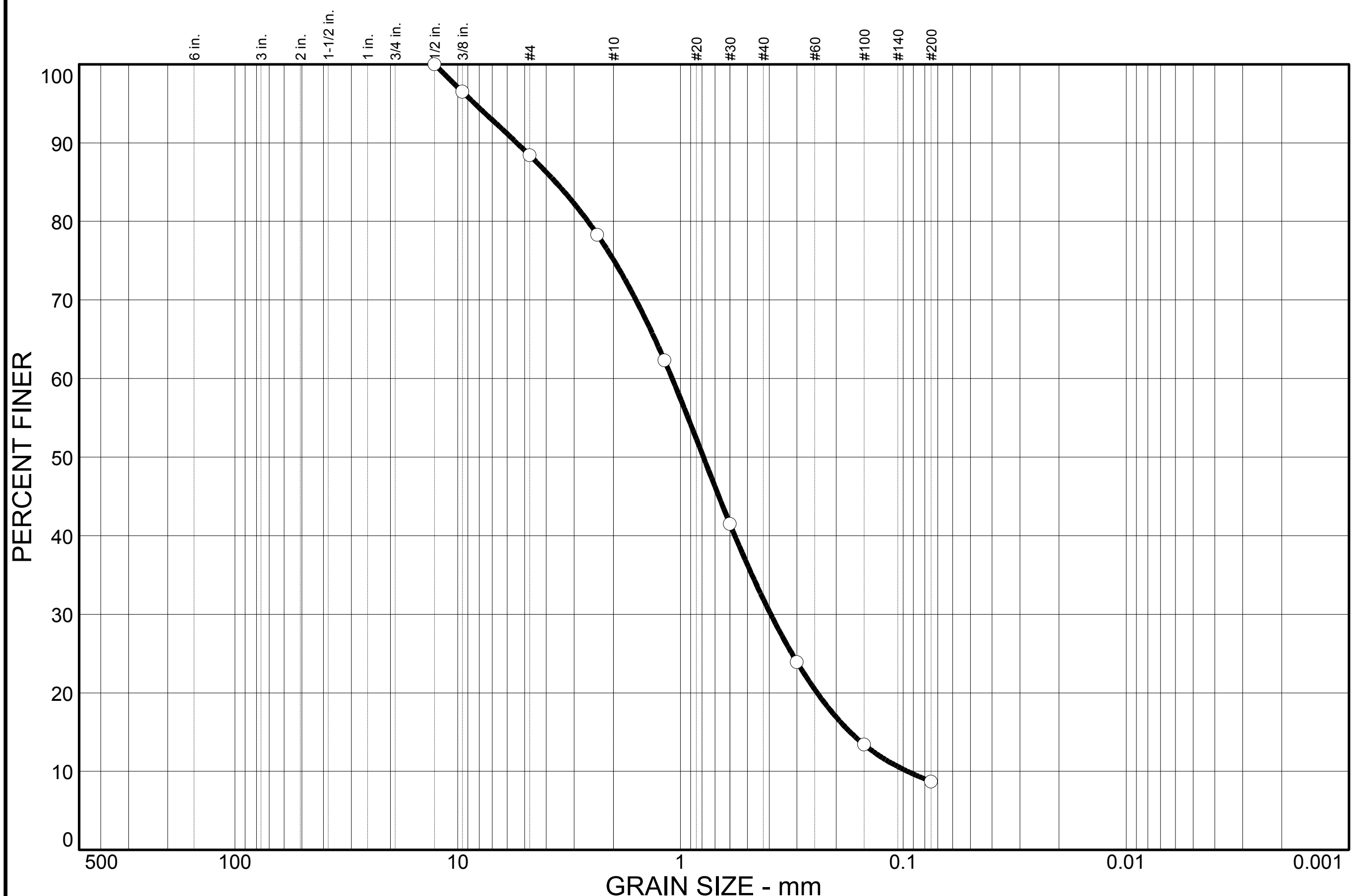
Sample No.: B-2 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 28.5-30'

Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks
Project No: H33201.01 **Figure** 3

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	11.6	13.3	43.2	23.2	8.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	96.5		
#4	88.4		
#8	78.3		
#16	62.3		
#30	41.5		
#50	23.9		
#100	13.4		
#200	8.7		

* (no specification provided)

Material Description

Well-graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.63 D_{60}= 1.09 D_{50}= 0.789
D_{30}= 0.394 D_{15}= 0.173 D_{10}= 0.0957
C_u= 11.40 C_c= 1.49

Classification

USCS= SW-SM AASHTO=

Remarks

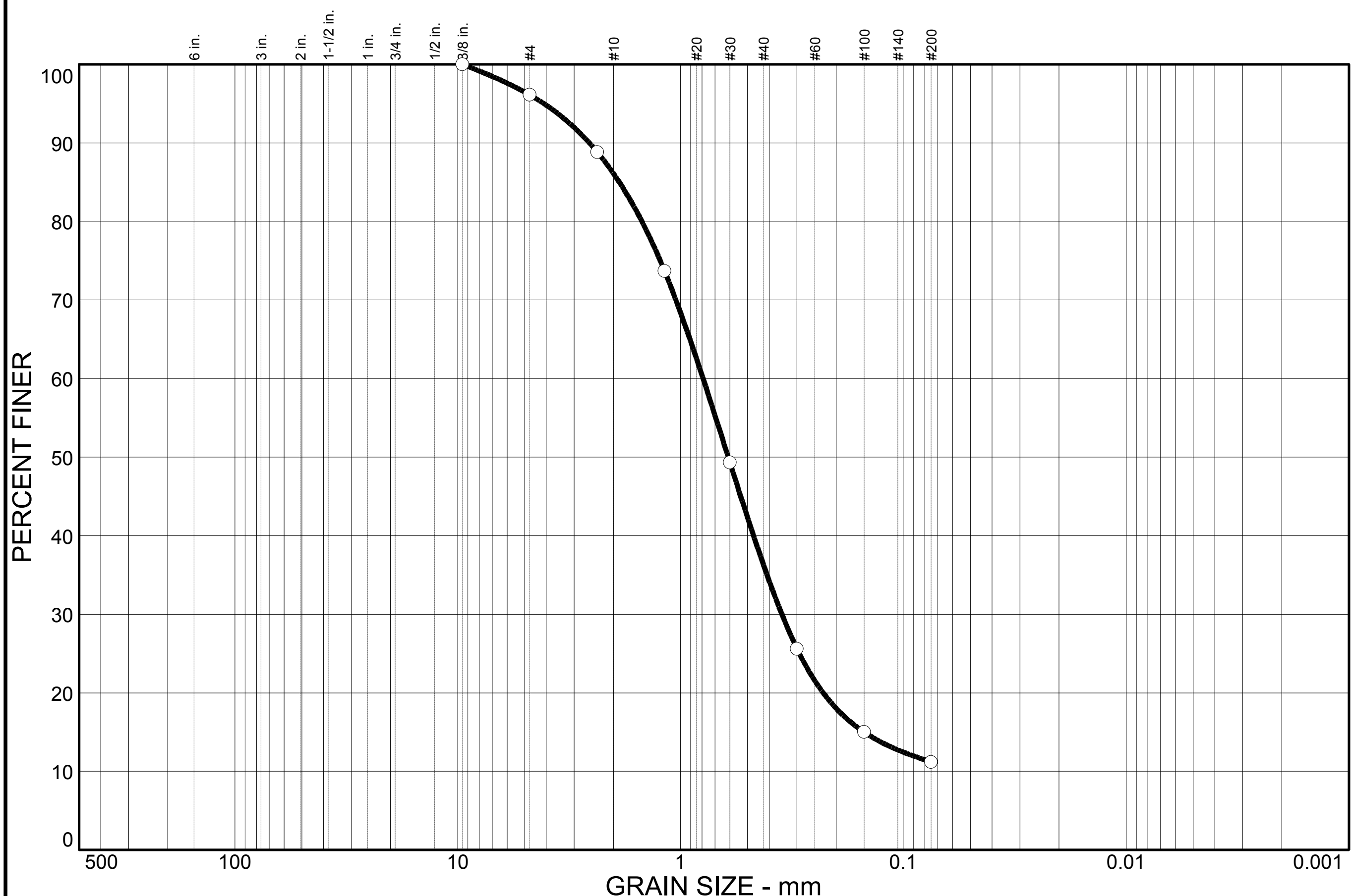
Sample No.: B-2 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 48.5-50'

Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks
Project No: H33201.01 **Figure** 4

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	3.9	10.0	49.8	25.1	11.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	96.1		
#8	88.8		
#16	73.7		
#30	49.3		
#50	25.6		
#100	15.0		
#200	11.2		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 1.88 D_{60}= 0.792 D_{50}= 0.611
D_{30}= 0.351 D_{15}= 0.150 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: P-1 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 18.5-20'

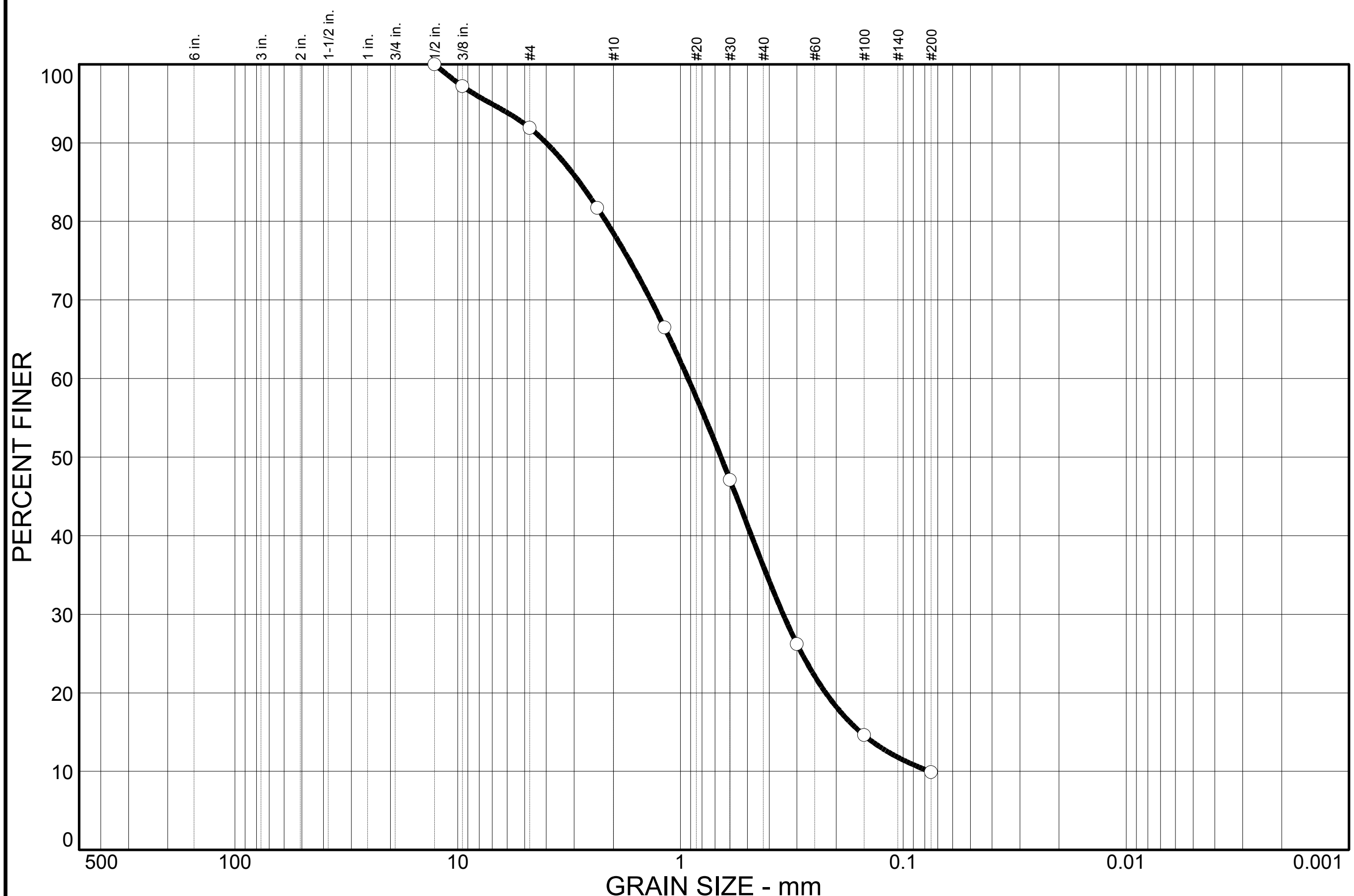
Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks

Project No: H33201.01 **Figure** 5

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	8.1	13.4	42.4	26.2	9.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	97.2		
#4	91.9		
#8	81.7		
#16	66.5		
#30	47.1		
#50	26.2		
#100	14.6		
#200	9.9		

* (no specification provided)

Material Description

Well-graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 2.85 D_{60}= 0.925 D_{50}= 0.658
D_{30}= 0.346 D_{15}= 0.156 D_{10}= 0.0765
C_u= 12.09 C_c= 1.69

Classification

USCS= SW-SM AASHTO=

Remarks

Sample No.: P-2 **Source of Sample:** **Date:** 10/23/24
Location: **Elev./Depth:** 18.5-20'

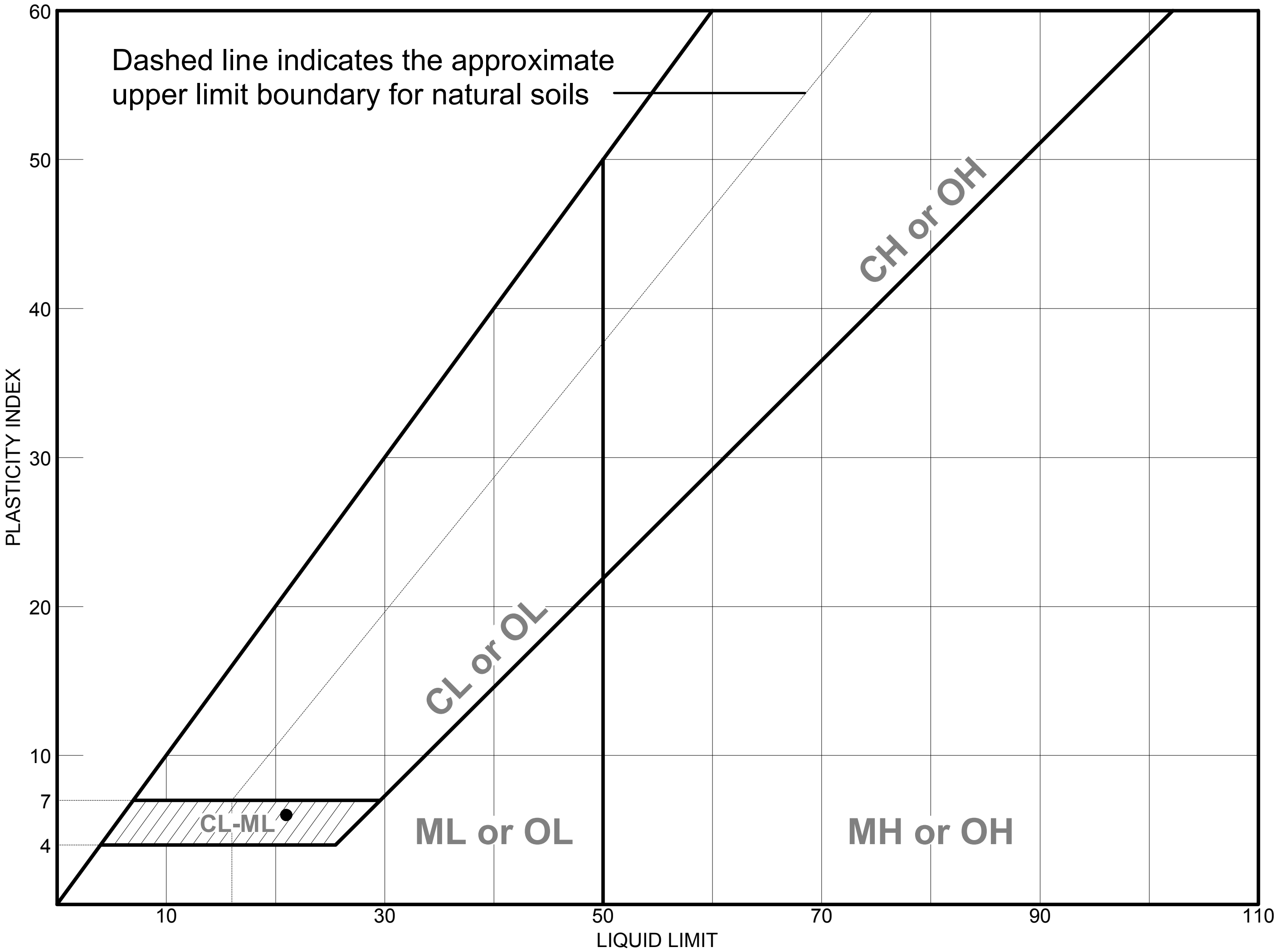
Moore Twining Associates, Inc.

Fresno, CA

Client: Fountainhead Development
Project: Proposed Starbucks

Project No: H33201.01 **Figure** 6

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silty, clayey sand	21	15	6	42.1	20.4	SC-SM

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

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

Remarks:

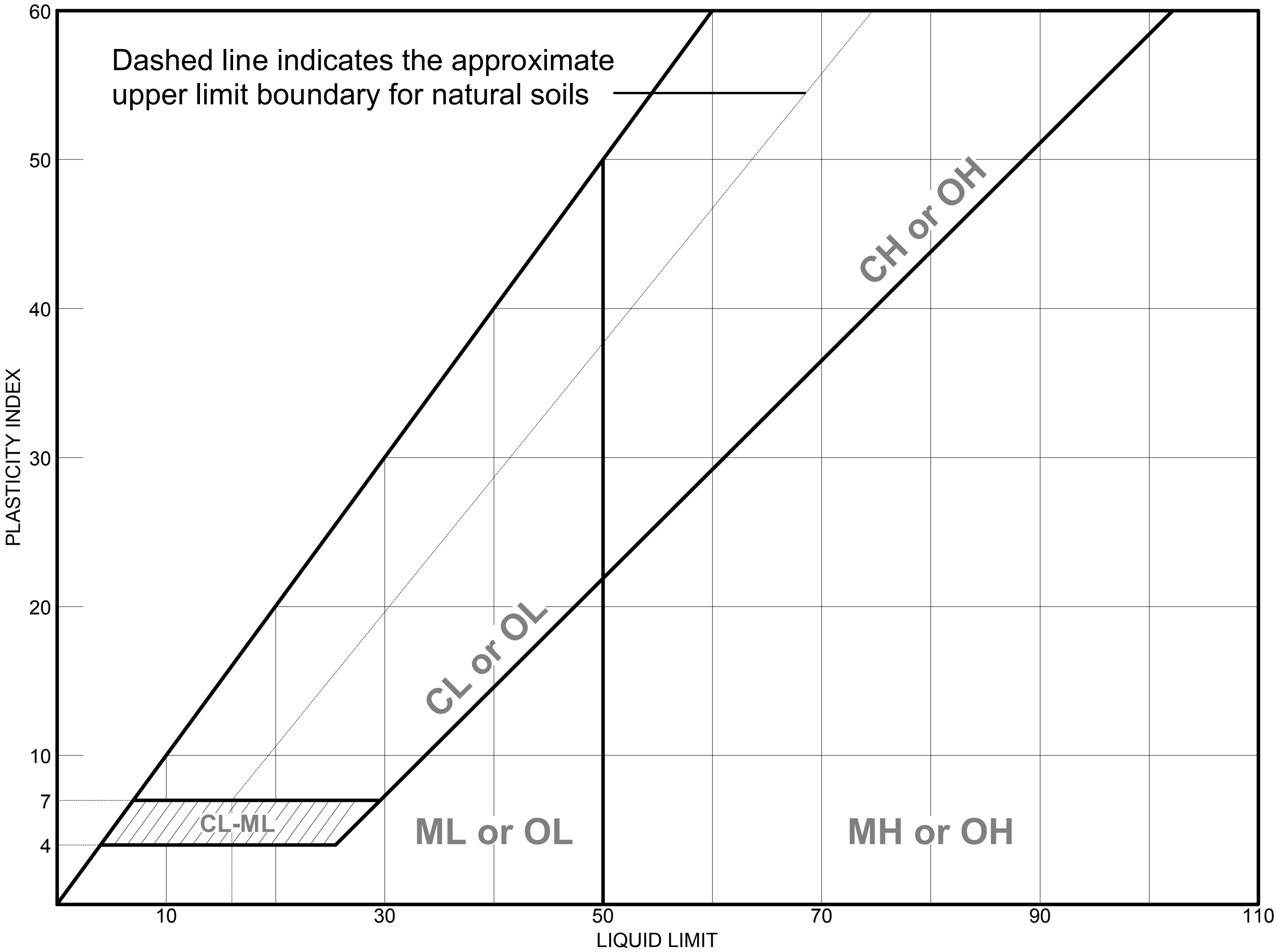
●

Moore Twining Associates, Inc.

Fresno, CA

Figure 7

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly graded sand with silt	NV	NP	NP	27.3	10.5	SP-SM

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

● **Source:** **Sample No.:** B-1 **Elev./Depth:** 8.5-10'

Remarks:

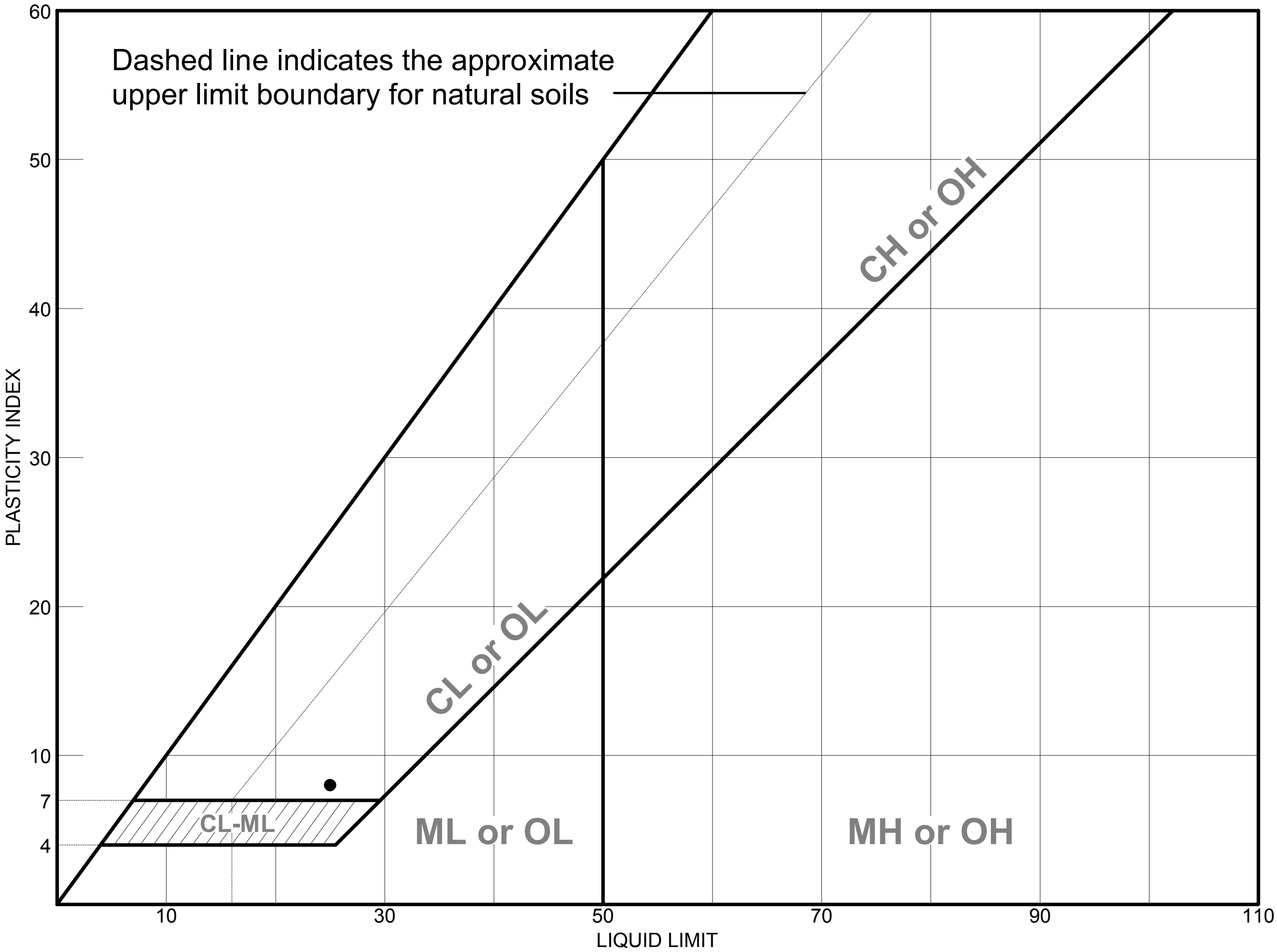
●

Moore Twining Associates, Inc.

Fresno, CA

Figure 8

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Clayey sand	25	17	8	63.3	29.7	SC

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

● **Source:** **Sample No.:** B-2 **Elev./Depth:** 28.5-30'

Moore Twining Associates, Inc.

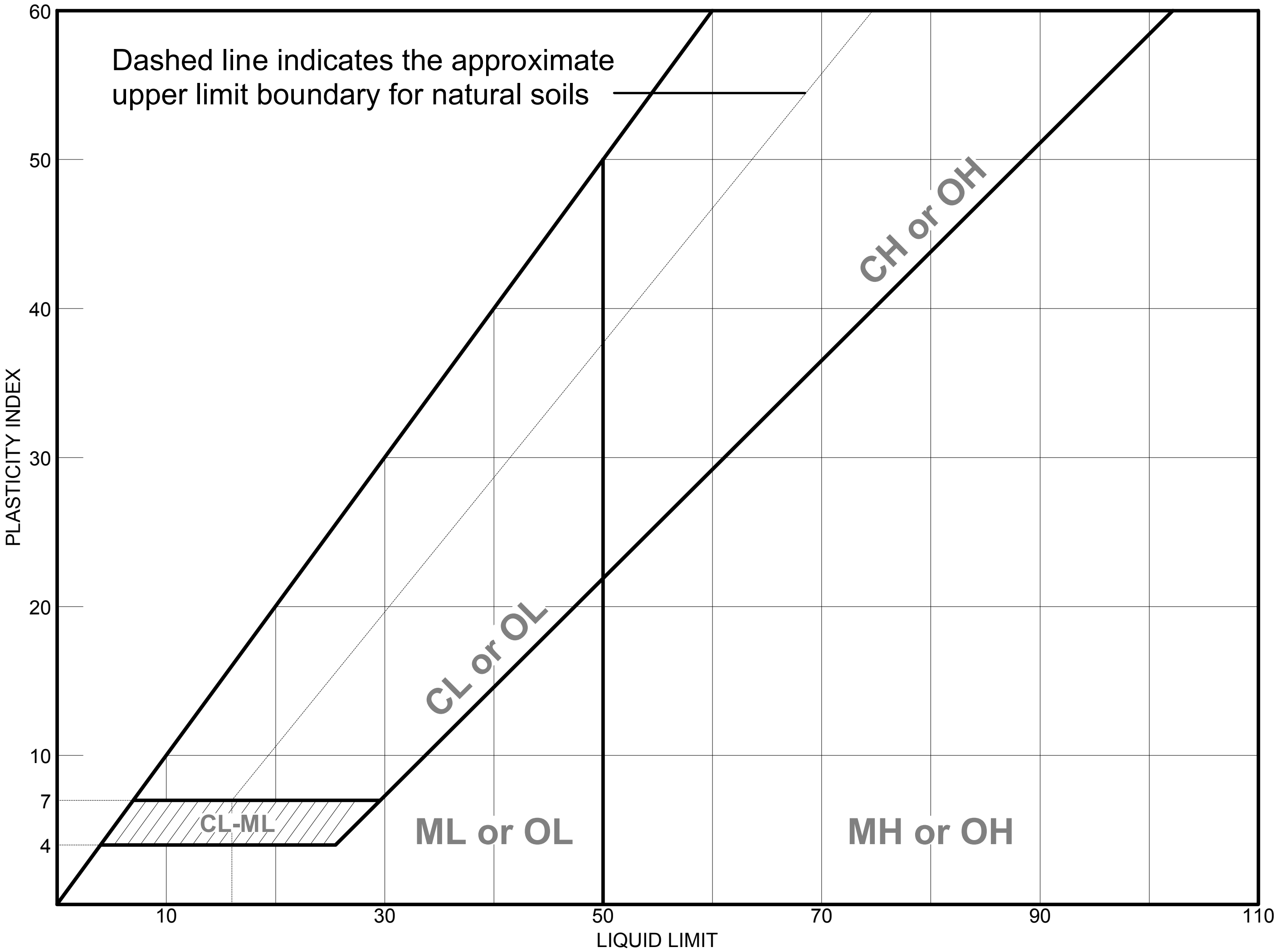
Fresno, CA

Remarks:

●

Figure 9

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly graded sand with silt	NV	NP	NP	36.3	11.2	SP-SM

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

● **Source:** **Sample No.:** P-1 **Elev./Depth:** 18.5-20'

Remarks:

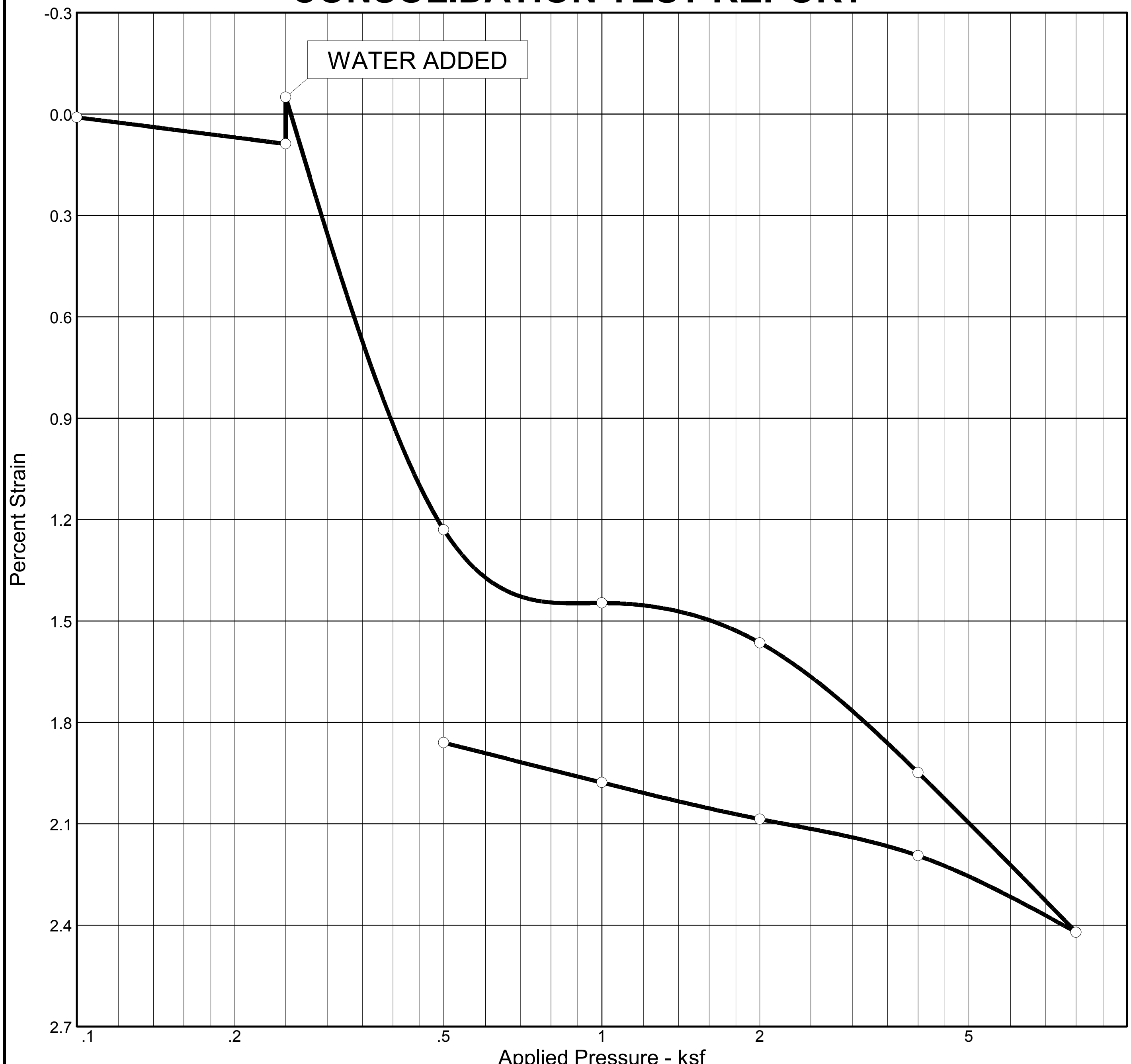
●

Moore Twining Associates, Inc.

Fresno, CA

Figure 10

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.											
31.0 %	5.3 %	113.8			2.65		3.05	0.02	0.01	0.27	0.1	0.454

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand		

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

Source: **Sample No.:** B-2 **Elev./Depth:** 1-2.5'

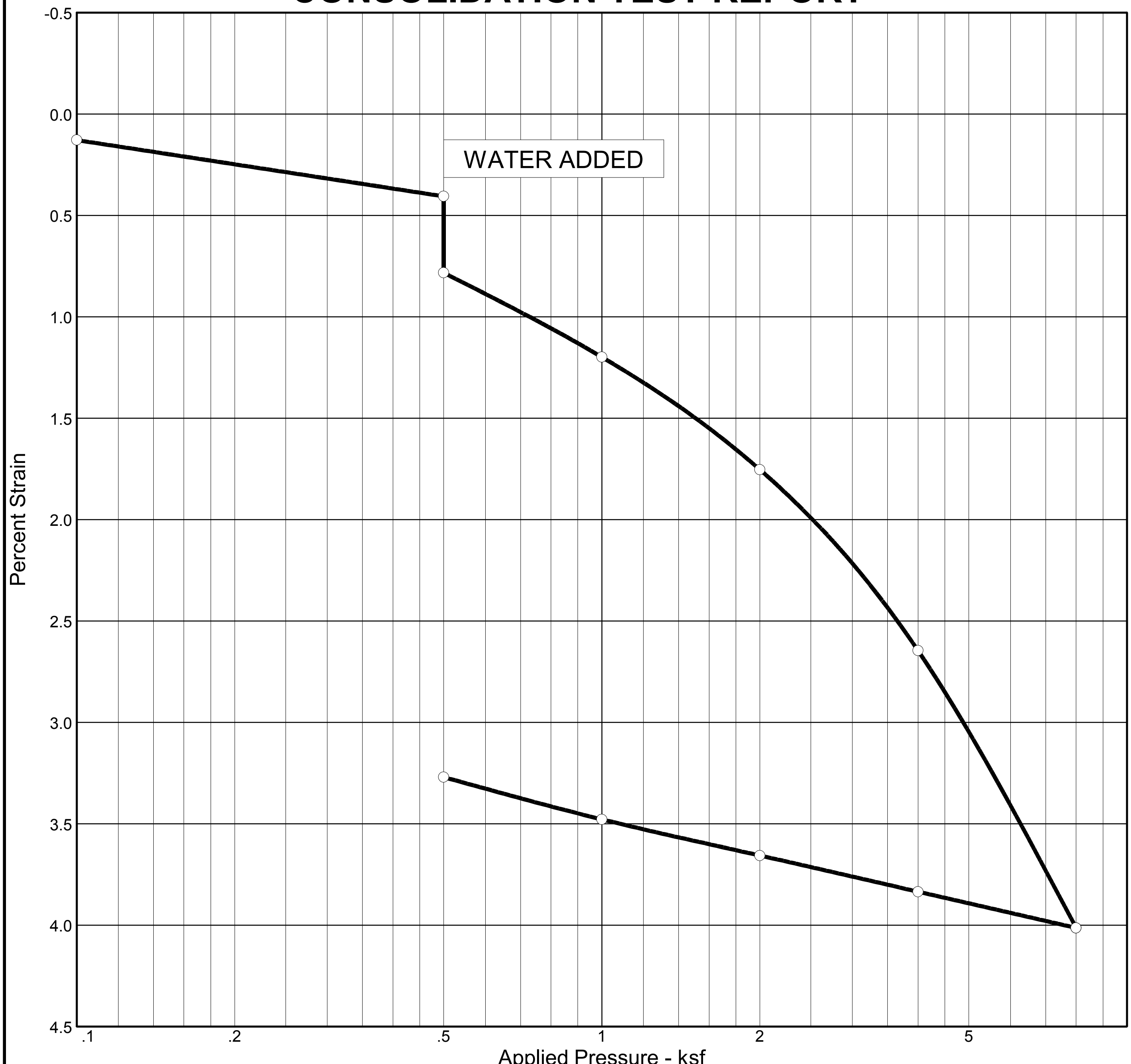
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure 11

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
66.2 %	8.6 %	122.9			2.65		3.54	0.06	0.01		0.4	0.346

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand		

Project No. H33201.01 **Client:** Fountainhead Development

Project: Proposed Starbucks

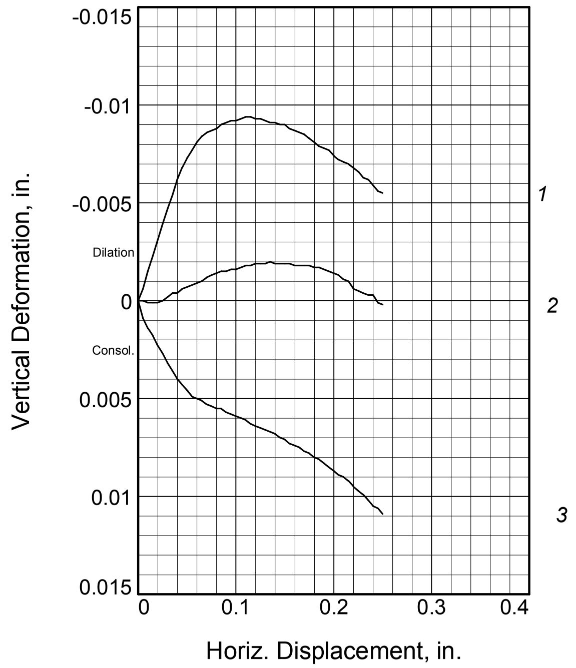
Source: **Sample No.:** B-2 **Elev./Depth:** 5-6.5'

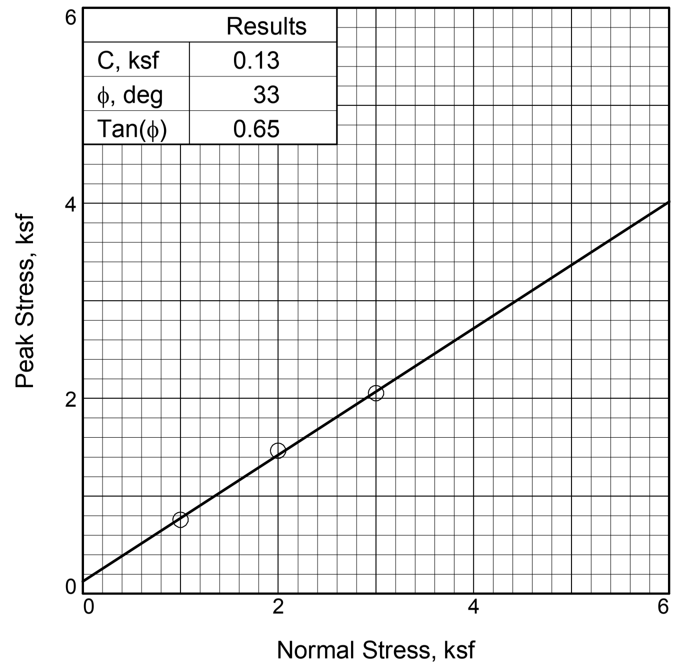
Moore Twining Associates, Inc.

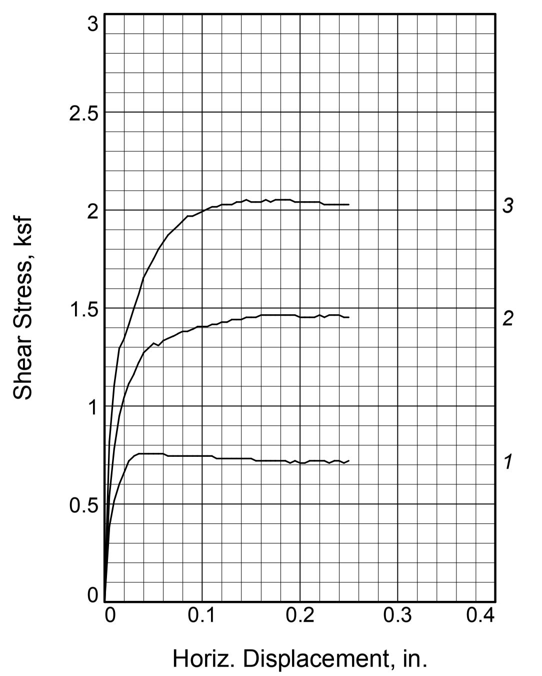
Fresno, CA

Remarks:

Figure 12







	Sample No.	1	2	3
Initial	Water Content, %	7.9	5.2	6.4
	Dry Density, pcf	115.4	117.6	115.8
	Saturation, %	48.4	33.7	39.5
	Void Ratio	0.4333	0.4072	0.4292
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	15.6	14.7	15.4
	Dry Density, pcf	115.6	117.9	116.1
	Saturation, %	96.0	96.5	96.2
	Void Ratio	0.4311	0.4033	0.4250
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	0.76	1.46	2.05
	Displacement, in.	0.04	0.16	0.14
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	1.4	6.6	6.0

Sample Type:
Description: Silty sand

Specific Gravity= 2.65
Remarks:

Figure 13

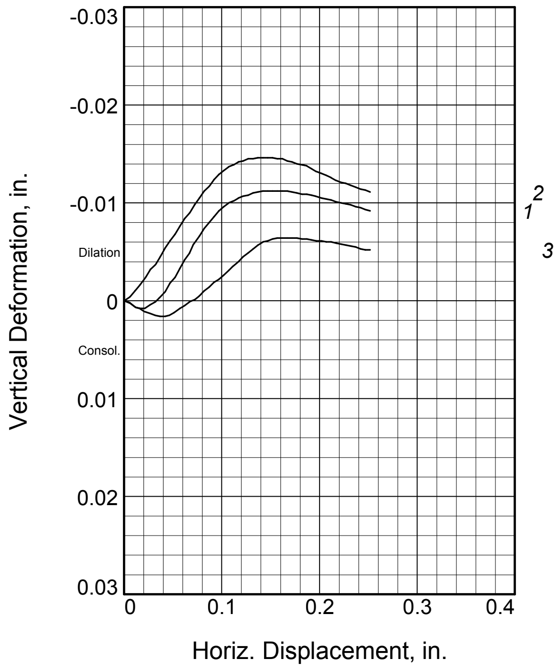
Client: Fountainhead Development

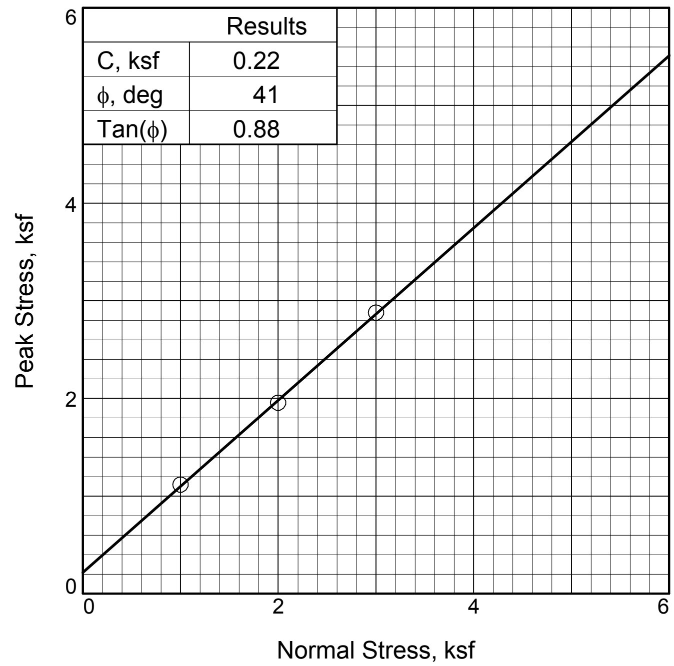
Project: Proposed Starbucks

Sample Number: B-2 **Depth:** 1-2.5'

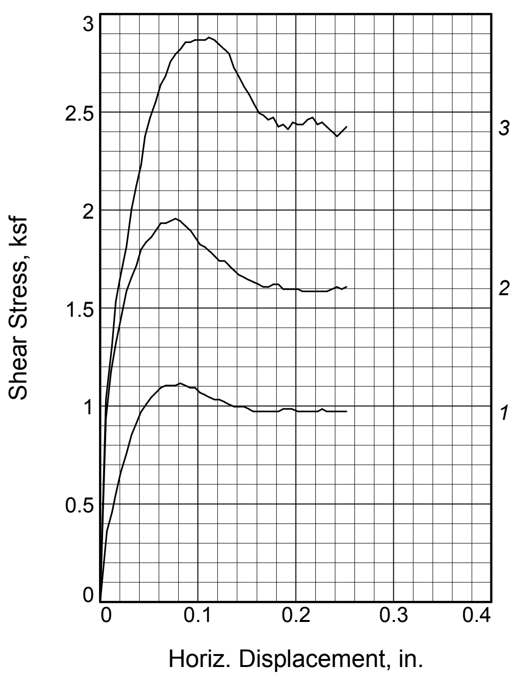
Proj. No.: H33201.01 **Date Sampled:** 10/23/24

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





Results	
C, ksf	0.22
ϕ, deg	41
Tan(ϕ)	0.88



	Sample No.	1	2	3
Initial	Water Content, %	10.3	9.3	8.3
	Dry Density, pcf	110.9	117.6	114.4
	Saturation, %	55.7	60.3	49.2
	Void Ratio	0.4924	0.4069	0.4458
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.1	14.8	15.9
	Dry Density, pcf	111.0	117.8	114.7
	Saturation, %	98.0	96.9	95.4
	Void Ratio	0.4907	0.4038	0.4417
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Peak Stress, ksf	1.12	1.96	2.88
	Displacement, in.	0.08	0.08	0.11
	Ultimate Stress, ksf			
	Displacement, in.			
	Strain at peak, %	3.4	3.2	4.6

Sample Type:

Description: Silty sand

Specific Gravity= 2.65

Remarks:

Figure 14

Client: Fountainhead Development

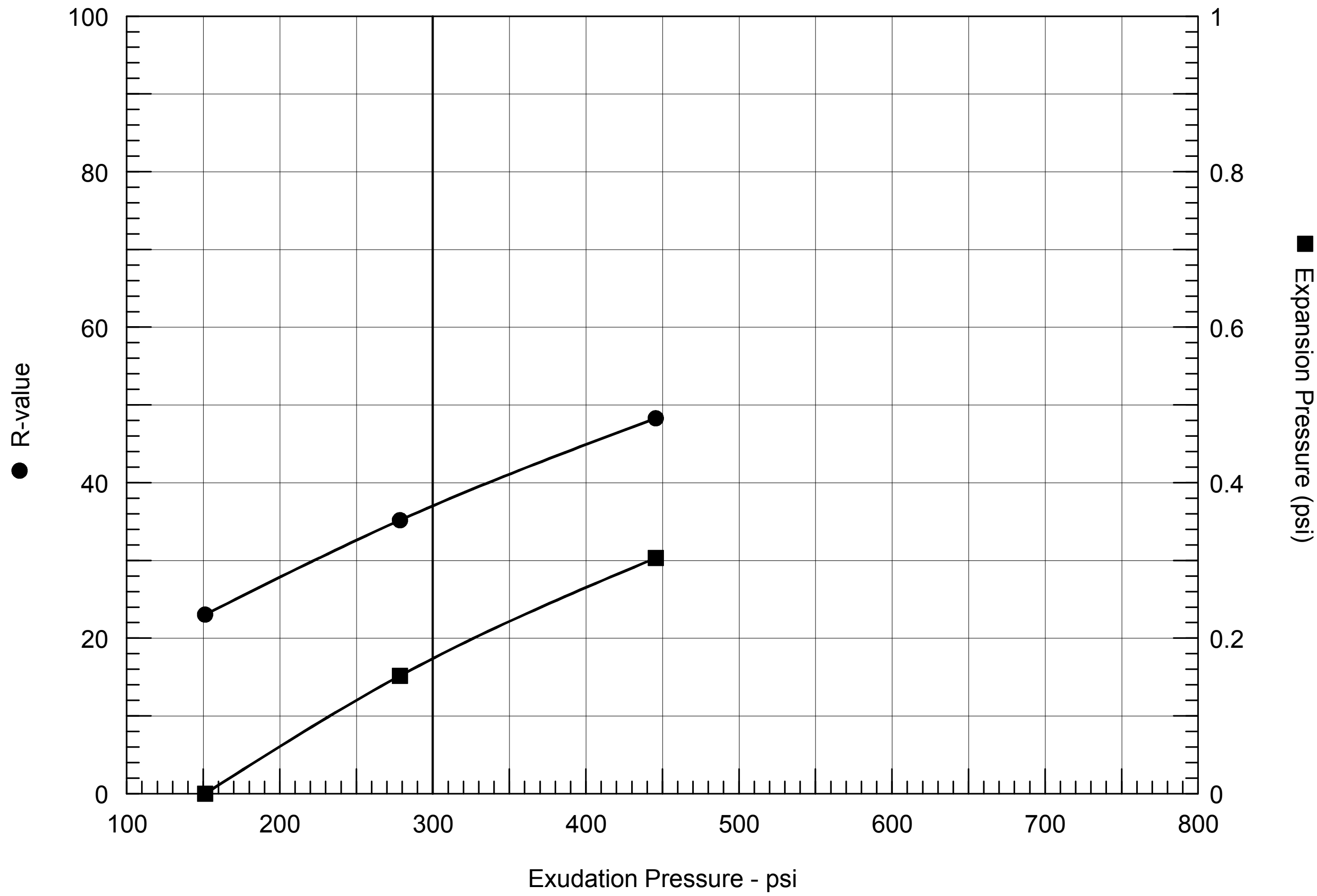
Project: Proposed Starbucks

Sample Number: B-2 **Depth:** 5-6.5'

Proj. No.: H33201.01 **Date Sampled:** 10/23/24

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

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	127.7	11.6	0.30	71	2.44	446	50	48
2	200	125.3	12.5	0.15	93	2.51	279	35	35
3	100	122.0	13.5	0.00	115	2.58	151	22	23

Test Results	Material Description
R-value at 300 psi exudation pressure = 37 **Exp. pressure at 300 psi exudation pressure =** 0.17 psi	Mix of silty sand and clayey sand

Project No.: H33201.01

Project: Proposed Starbucks

Sample Number: B-3 **Depth:** 1-5'

Date: 11/8/2024

Tested by: CG

Checked by: MS

Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure 15

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

November 07, 2024

Work Order #: **KJ29014**

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

RE: Proposed Starbucks

Enclosed are the analytical results for samples received by our laboratory on **10/29/24** . For your reference, these analyses have been assigned laboratory work order number **KJ29014**.

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

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

Sincerely,

Moore Twining Associates, Inc.

Lauren Cox
Client Services Representative

Figure 16

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Proposed Starbucks **Project Number:** H33201.01 **Project Manager:** Allen Harker	**Reported:** 11/07/2024

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-2 @ 0.2-5		KJ29014-01	Soil	10/23/24 00:00	10/29/24 11:00

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

MTA Geotechnical Division 2527 Fresno Street Fresno CA, 93721	**Project:** Proposed Starbucks **Project Number:** H33201.01 **Project Manager:** Allen Harker	**Reported:** 11/07/2024

B-2 @ 0.2-5

KJ29014-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	**0.0021**	0.00060	% by Weight	[CALC]	11/02/24	11/02/24	[CALC]	
Chloride	**21**	6.0	mg/kg	B4J3113	10/31/24	11/02/24	Cal Test 422	
pH	**7.5**	0.10	pH Units	B4J3113	10/31/24	11/04/24	Cal Test 643 M	
Sulfate as SO4	**0.0026**	0.00060	% by Weight	[CALC]	11/02/24	11/02/24	[CALC]	
Sulfate as SO4	**26**	6.0	mg/kg	B4J3113	10/31/24	11/02/24	Cal Test 417	

Notes and Definitions

DUP1	A high RPD was observed between a sample and this sample's duplicate.
PREP	Modified preparation by pulverizing sample to pass #40 sieve and soaked for a minimum of 12 hours using a minimum dilution ratio of 1:10
ND	Analyte NOT DETECTED at or above the reporting limit
mg/kg	milligrams per kilogram (parts per million concentration units)

MOORE TWINING ASSOCIATES, INC.
Since 1898

Project Name:	Proposed Starbucks	Report Date:	11/7/2024
		Sample Date:	10/23/2024
Project Number:	H33201.01		
		Sampled By:	AV
Subject:	Minimum Resistivity, ASTM G187	Tested By:	RS
Material Description:	Silty sand	Test Date:	11/4/2024
Location:	B-2 @ 0.2-5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
25 mls	9,000
50 mls	6,000
75 mls	4,400
100 mls	3,400
125 mls	3,100
150 mls	3,200

Remarks: Min. Resistivity is 3,100 Ohm-cm

Figure 17

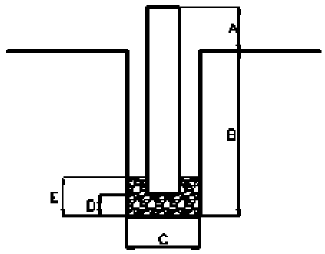
www.mooretwining.com
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST
No. P-1

Project:	Proposed Starbucks	**Project No.**	H33201.01
Location:	SWC of Main Street and 7th Avenue, Hesperia, CA	**Test Date:**	10/24/2024
Coordinates:			



A. Top of Pipe Above Ground	0	Inches
B. Depth of Hole	243	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	5	Inches
E. Total Gravel Layer Thickness	60	Inches
F. Pipe Length	238	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: 40-50 gallons of water for required 2-hour presoak
Water was constantly filled up to about 2.45 feet
from bottom of hole on 10/24/24

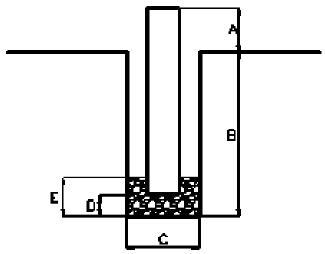
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	**10/24/2024**	**8:40:00**	**17.9**				
	10/24/2024	**8:41:25**	**18.4**	**1.42**	**6**	**0.6**	**7.2**
2	**10/24/2024**	**8:41:25**	**18.4**				
	10/24/2024	**8:43:50**	**18.9**	**2.42**	**6**	**1.0**	**5.4**
Refill 3	**10/24/2024**	**8:45:30**	**17.9**				
Begin Test	**10/24/2024**	**8:55:30**	**19.28**	**10.00**	**16.56**	**1.5**	**3.5**
Refill 4	**10/24/2024**	**8:57:30**	**17.9**				
	10/24/2024	**9:07:30**	**19.27**	**10.00**	**16.44**	**1.6**	**3.5**
Refill 5	**10/24/2024**	**9:09:25**	**17.9**				
	10/24/2024	**9:19:25**	**19.27**	**10.00**	**16.44**	**1.6**	**3.5**
Refill 6	**10/24/2024**	**9:21:25**	**17.9**				
	10/24/2024	**9:31:25**	**19.25**	**10.00**	**16.2**	**1.6**	**3.4**
Refill 7	**10/24/2024**	**9:32:55**	**17.9**				
	10/24/2024	**9:42:55**	**19.23**	**10.00**	**15.96**	**1.6**	**3.3**
Refill 8	**10/24/2024**	**9:44:20**	**17.9**				
	10/24/2024	**9:54:20**	**19.22**	**10.00**	**15.84**	**1.6**	**3.3**
Refill 9	**10/24/2024**	**9:56:30**	**17.9**				
	10/24/2024	**10:06:30**	**19.22**	**10.00**	**15.84**	**1.6**	**3.3**

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project:	Proposed Starbucks	**Project No.**	H33201.01
Location:	SWC of Main Street and 7th Avenue, Hesperia, CA	**Test Date:**	10/24/2024
Coordinates:			



A. Top of Pipe Above Ground	1	Inches
B. Depth of Hole	242	Inches
C. Diameter of Hole	8	Inches
D. Depth of Gravel Below Pipe	5	Inches
E. Total Gravel Layer Thickness	60	Inches
F. Pipe Length	238	Inches
G. Pipe Diameter	2	Inches

Pre-saturated: 60 gallons of water for required 2-hour presoak
Checked Water was constantly filled up to about 2.4 feet from bottom of hole on 10/24/24
Gravel Correction Factor: 2.6

Trial	Date	Time	Depth To Water* (feet)	Time Interval (min)	Water Drop (inches)	Unfactored Percolation Rate, (minutes per inch)	Unfactored Infiltration Rate, (Inches per hour)
1	10/24/2024	11:40:00	17.85				
	10/24/2024	11:41:45	18.35	1.75	6	0.7	5.7
2	10/24/2024	11:41:45	18.35				
	10/24/2024	11:44:05	18.85	2.33	6	1.0	5.4
Refill 3	10/24/2024	11:45:50	17.9				
Begin Test	10/24/2024	11:55:50	19.5	10.00	19.2	1.3	4.3
Refill 4	10/24/2024	11:57:30	17.8				
	10/24/2024	12:07:30	19.41	10.00	19.32	1.3	4.1
Refill 5	10/24/2024	12:09:30	17.9				
	10/24/2024	12:19:30	19.5	10.00	19.2	1.3	4.3
Refill 6	10/24/2024	12:21:15	17.9				
	10/24/2024	12:31:15	19.51	10.00	19.32	1.3	4.3
Refill 7	10/24/2024	12:33:00	17.9				
	10/24/2024	12:43:00	19.5	10.00	19.2	1.3	4.3
Refill 8	10/24/2024	12:44:30	17.9				
	10/24/2024	12:54:30	19.5	10.00	19.2	1.3	4.3

* Depth to water measured from top of pipe

APPENDIX E

COMPACTION TEST REPORT, TEST DATA AND TEST LOCATIONS FOR BACKFILL OF THE AREA OF REMOVED UNDERGROUND STORAGE TANKS WITH ENGINEERED FILL

This appendix contains the compaction test report, test data and test locations, prepared by Hi Desert Testing & Inspection, dated December 1, 1998, for backfill of the removed underground storage tanks with engineered fill in the vicinity of the former fuel canopy. The area of the removed Underground Storage Tanks are also shown on Drawing No. 2 in Appendix A of this report.

Dan Goodwin,
Hi Desert
Testing & Inspection

December 1, 1998
HDT&I P.N. 81041
Report No. 1

ADVANCED ENVIRONMENTAL CONCEPTS, INC.
4400 Ashe Road #206
Bakersfield, CA 93313
(805)831-1646

Attention: Mr. Jonathan L. Buck
Reference: 15901 Main Street, Hesperia, California.

Gentlemen:

In accordance with your request, a representative of this office observed backfilling of two gas tank excavations at the referrenced site, and performed random representative testing of compacted backfill. Samples of the soils were delivered to our laboratory where maximum density and optimum moisture were determined.

Results of our inspections and testing indicates backfill compaction complies with minimum requirements. Results are shown on the attached sheet.

REGISTERED PROFESSIONAL ENGINEER
JERRY L. MILES
No. 42593
Exp 3/31/2000
CIVIL
STATE OF CALIFORNIA

Respectively submitted,
HI DESERT TESTING & INSPECTION

Dan D. Goodwin

Jerry L. Miles, P.E.
RCE 42593

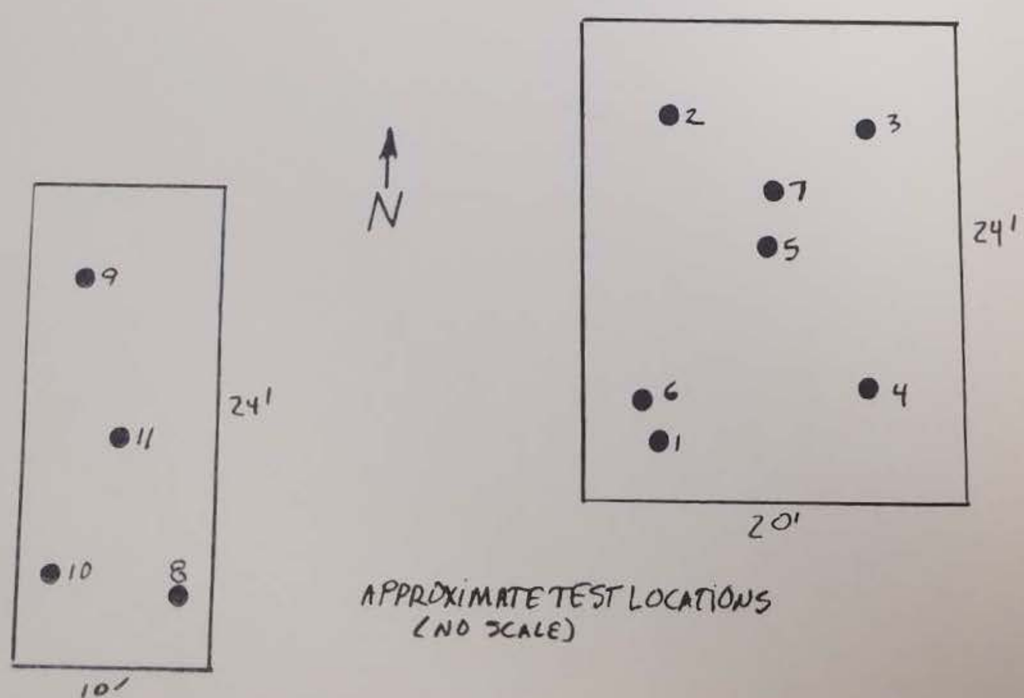
TEST RESULTS

ASTM D 1557-91 TEST METHODS FOR LABORATORY COMPACTION CHARACTERISTICS OF SOIL USING MODIFIED EFFORT

SAMPLE NO.	MAXIMUM DENSITY, P.C.F.	OPTIMUM MOISTURE CONTENT, %
1	126.5	11.0
2	124.0	9.5

ASTM D 2922-91 TEST METHOD FOR DENSITY AND UNIT WEIGHT OF SOIL IN PLACE BY NUCLEAR METHOD.

Test No.	Depth From Finished Grade Ft.	Dry Density, P.C.F.	Maximum Density, P.C.F.	Relative Density, %	Required R.D., %
1	11.0	119.6	126.5	94.5	90
2	9.0	119.8	126.5	94.7	90
3	7.0	125.4	126.5	99.1	90
4	6.0	120.4	126.5	95.2	90
5	4.0	112.8	124.0	91.0	90
6	2.0	121.0	124.0	97.6	90
7	0.5	122.7	124.0	99.0	90
8	10.0	119.9	124.0	96.7	90
9	8.0	121.4	124.0	97.9	90
10	6.0	120.7	124.0	97.3	90
11	0.5	117.1	124.0	94.4	90



APPROXIMATE TEST LOCATIONS
(NO SCALE)

DG/cg