MOORE TWINING ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED CIRCLE K

NORTHWEST CORNER OF TWENTYNINE PALMS HIGHWAY

AND AVALON AVENUE

YUCCA VALLEY, CALIFORNIA

Project Number: G28851.02

For:

Circle K Stores - West Coast Division
255 E. Rincon Street, Suite 100
Corona, CA 92879

August 30, 2024

www.mooretwining.com

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

MOORE TWINING ASSOCIATES, INC.

August 30, 2024 G28851.02

Ms. Sandra Agraz
Circle K Stores - West Coast Division
255 E. Rincon Street, Suite 100
Corona, CA 92879

Subject: **Geotechnical Engineering Investigation**
Proposed Circle K
Northwest Corner of Twentynine Palms Highway and Avalon Avenue
Yucca Valley, California

Dear Ms. Agraz:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed Circle K to be located at the northwest corner of Twentynine Palms Highway and Avalon Avenue in Yucca Valley, 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 those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement; however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

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

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

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

Sincerely,

MOORE TWINING ASSOCIATES, INC.

Allen H. Harker, CEG
Certified Engineering Geologist
Geotechnical Engineering Division

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. (Moore Twining) prepared this geotechnical engineering investigation report for the proposed Circle K Store development to be located at the northwest corner of Twentynine Palms Highway and Avalon Avenue in Yucca Valley, California.

Based on our review of the referenced site plan, the proposed development is anticipated to include a Circle K store building (5,200 square feet) in the northern portion of the site, a fueling island with canopy is planned southeast of the Circle K store, and an underground storage tank area is planned southeast of the fuel canopy. Appurtenant construction is anticipated to include concrete walkways, asphaltic concrete and Portland cement concrete parking and drive areas, a trash enclosure, underground utilities, and landscaped areas.

At the time of our field exploration, the ground surface sloped gently down to the north and was generally covered by soil and some scattered trash, Creosote bushes and scattered Yucca Trees (shown and referred to on the ALTA/NSPS Land Title Survey Map as Yucca Trees which are similar but slightly different than Joshua Trees). Overhead power lines cross the site in many areas. One set of overhead power lines trends across the site parallel to the northern part of the proposed fuel canopy. Just beyond the east end of the proposed fuel canopy is a power pole and utility box, and the overhead power lines diverge from this power pole in three directions to the north, northeast and southeast. An underground electric line was marked as trending east between the utility box on the east side of the proposed fuel canopy to another utility box near Avalon Avenue.

The project site is located in an Alquist-Priolo Earthquake Fault Zone for the Eureka Peak Fault. The Fault Activity Map of California, prepared by the California Geological Survey, shows an active fault as being about 200 feet southwest of the Circle K site. The eastern portion of the Earthquake Fault Zone for this fault overlaps the western portion of the Circle K parcel. This short fault segment is unnamed but indicated by the California Geological Survey on the Fault Activity Map of California to be part of the Eureka Peak Fault Zone. This short fault segment of the Eureka Peak Fault Zone was informally named the "Avalon Avenue Splay" (CDMG, 1993a). The eastern edge of the Earthquake Fault Zone for the "Avalon Avenue Splay" of the Eureka Peak Fault Zone is roughly parallel with the western edge of the proposed Circle K store. Thus, a fault investigation needed to be conducted.

A fault trench investigation was conducted in July and August 2024 across the proposed Circle K building footprint, and the fault trench was logged by geologists from Hilltop Geotechnical, Inc. (Hilltop) who were retained by Moore Twining. The City of Yucca Valley hired a thirty party reviewer (Mr. Matt Cohrt, Certified Engineering Geologist with Sladden Engineering) to meet Moore Twining at the site to review the fault trench findings and later review Hilltop's Fault Trench investigation report (included in Appendix E of this report). Thus, on August 1, 2024, Mr. Allen Harker, Certified Engineering Geologist with Moore Twining, met with Mr. Cohrt with Sladden Engineering to review the findings of the fault trench that was logged by the geologists from Hilltop. An unbroken layered sequence of Quaternary alluvial materials was observed with laterally continuous contacts. The trench also identified units of younger alluvium (Qa) overlying units of older alluvium (Qoa). Mr. Harker and Mr. Cohrt concurred with the geologists from Hilltop that there were no features indicative of active surface faulting within the exploratory fault trenches excavated.

EXECUTIVE SUMMARY (continued)

Between August 6 and August 23, 2024, the fault trench was backfilled as certified, compacted engineered fill, and on August 24, 2024, the fault trench area was graded to restore the area to the existing drainage pattern at the site (sloping gently down to the north/northwest).

Between August 16, 2024 and August 18, 2024, ten (10) test borings (B-1 through B-9 and P-2) were drilled to depths ranging from about 10 to 50 feet BSG. Percolation test borings B-2/P-1 and B-5/P-3 were utilized to drill 10-foot borings for proposed drive-entry areas and also conduct percolation tests. Percolation test P-2 was drilled to a depth of 40 feet BSG.

Based on our review of the Geologic Map of the Joshua Tree Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-516, Scale 1:62,500, dated 1967, prepared by Thomas W. Dibblee, the site is mapped as being underlain by Pleistocene and Recent (Holocene-age) alluvium (Qa). On the opposite side (southeast side) of Twentynine Palms Highway, across the street from the site, the ground surface is mapped as being underlain by Pleistocene age older alluvium (Qoa). Both the younger alluvium (Qa) and older alluvium (Qoa) were encountered below the subject site during the fault trench investigation and borings drilled for the field exploration.

The soils encountered in the borings drilled for this investigation generally consisted of silty sands extending from the ground surface to the maximum depth explored, about 50 feet BSG. It should be noted that the upper 2½ to 3½ feet of soils encountered in borings B-6 and B-7 in the Circle K building pad area were loose. Borings B-6 and B-7 were drilled in the northern portion of the building footprint, north of the fault trench and in soils that were undisturbed from the fault trench activities.

Fill soils were encountered in one boring (B-1) that was drilled at the location of the proposed underground storage tanks. Samples collected at depths of 0 to 1½ feet and 3½ to 5 feet in boring B-1 both had cloth material in the samples. The next sample collected from depths of 8½ to 10 feet BSG did not encounter any cloth or show any other indications that the soil was fill material. Thus, the log for boring B-1 is shown as having about 8½ feet of fill encountered at the site.

The near surface granular soils encountered are non-plastic and have a very low expansion potential, low compressibility characteristics and negligible collapse potential, high shear strength characteristics and excellent support characteristics for pavements when compacted as engineered fill.

Although the on-site soils have a very low expansion potential, this report recommends that interior and exterior concrete slabs-on-grade and Portland cement concrete pavements be underlain by 4 inches of aggregate base for constructability purposes.

Groundwater was not encountered in the borings to the maximum depth explored, about 50 feet BSG.

Based on our review of groundwater data from wells on the Department of Water Resources website, a well located about ½ mile west of the Circle K site indicates groundwater has ranged from about 127 feet deep in 2012 to about 324 feet in 2008 for dates measured between the years 1980 and 2012.

EXECUTIVE SUMMARY (continued)

Due to the depth of groundwater being greater than 50 feet, the potential for liquefaction to occur is considered low. However, there is a 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. The seismic settlements were estimated to be about 1½ inches total and ¾ inches differential over a horizontal distance of 40 feet.

In the event the estimated seismic settlement is considered an impact to the structure design, a supplemental investigation using Cone Penetration Tests (CPTs) may be desirable to allow for a more detailed analysis of seismic settlement. CPTs provide nearly continuous soil penetration resistance information as compared with the typical sampling interval (about 5 feet) for soil borings. Therefore, the CPTs provide more data for use in refining the estimated seismic settlement.

In the event the differential seismic settlement is considered excessive for design of a conventional spread foundation system, alternative methods of site preparation or foundation design could be considered for the project. Based on our experience with other projects considering the size of the building, structural mitigation such as an interconnected gradebeam foundation system would be anticipated to be the most cost effective approach, if special mitigation is required. This report assumes the foundations can be designed for support of shallow spread foundations. In the event conventional shallow spread foundations cannot be designed to tolerate the anticipated seismic settlements, Moore Twining should be contacted to provide alternative recommendations.

The results at percolation test B-2/P-1 at a depth of 10 feet BSG indicated an unfactored infiltration rate of 9.1 inches per hour. The result at percolation test P-2 at a depth of 40 feet BSG indicated an unfactored infiltration rate of 11.2 inches per hour. The results at percolation test B-5/P-3 at a depth of 10 feet BSG indicated an unfactored infiltration rate of 5.8 inches per hour. The results indicate that storm water infiltration systems appear feasible for this site. This report recommends that the lower unfactored infiltration rate of 5.8 inches per hour be considered for use in design for infiltration systems when including an appropriate factor of safety.

To remove the upper loose soils and provide a uniform blanket of engineered fill across the building pad and limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet horizontally, this report recommends that the entire building pad be over-excavated to a depth of at least 4 feet below preconstruction site grades and to the depth required to provide at least 2 feet of engineered fill below bottom of footings, whichever is greater.

The results of soil sample analyses indicate that the near-surface soils exhibit a “moderately corrosive” corrosion potential to buried metal objects.

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).

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

TABLE OF CONTENTS

Page

TABLE OF CONTENTS

Page

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED CIRCLE K

NORTHWEST CORNER OF TWENTYNINE PALMS HIGHWAY

AND AVALON AVENUE

YUCCA VALLEY, CALIFORNIA

Project Number: G28851.02

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for a proposed Circle K store and fueling station to be located at the northwest corner of Twentynine Palms Highway and Avalon Avenue in Yucca Valley, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Circle K Stores, Inc. to perform this geotechnical engineering investigation.

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

The Geotechnical Engineering Division of Moore Twining 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 traffic loads;

2.1.2 Recommendations for 2022 California Building Code mapped 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 thickness and construction of new asphalt concrete (AC) and Portland cement concrete (PCC) pavements;

2.1.6 Recommendations for temporary excavations and trench backfill; and

2.1.7 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed Circle K improvements 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.

2.2 Scope: Our Professional Services Agreement and Work Order, dated January 18, 2024, for the geotechnical engineering investigation, and the Professional Services Agreement and Work Order, dated March 15, 2024, for the fault trench investigation, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 The Site Plan Exhibit CSP 6.0W, prepared by Greenberg Farrow, dated October 16, 2023, was reviewed for general project information.

2.2.2 The Circle K Appendix A - Standard Scope of Work, December 3, 2019, was reviewed.

2.2.3 Boring permit WP0039485 was obtained from San Bernardino County Public Health Environmental Health Services.

2.2.4 Satellite images of the site between the years 1989 and 2023 from online sources, were reviewed. In addition, historical aerial photographs of the site from various years between 1948 and 2016 were reviewed from a February 3, 2023 report entitled: “Phase I Environmental Site Assessment, Proposed Circle K Store - Imperial, Northwest Corner of Twentynine Palms Highway and Avalon Avenue, Yucca Valley, California 92284,” prepared by Moore Twining’s Environmental Division.

2.2.5 A report entitled, "Fault Investigation, Proposed Retail Center, NWC Avalon Avenue and Palisade Drive, APN 601-201-41, Yucca Valley, California," dated January 20, 2017, prepared by Sladden Engineering, was provided by Mr. Matt Cohrt (Sladden Engineering) who was the third party reviewer that was hired by the City of Yucca Valley to review the fault trench investigation that was performed at the Circle K project site by Hilltop Engineering Inc. Sladden Engineering had previously conducted a fault trench investigation across the street from the proposed Circle K building site, on the opposite side of Twentynine Palms Highway, and was trenched within the same Earthquake Fault Zone (formerly referred to as the Alquist-Priolo Special Studies Zone) where a portion of the Eureka Peak Fault is mapped and informally named the "Avalon Avenue Splay" (CDMG, 1993a). This report was reviewed and discussed in the "Site History and Previous Studies" section of this report.

2.2.6 An ALTA/NSPS Land Title Survey Map, prepared by Duryea & Associates, Inc., dated April 9, 2024, was reviewed for utility locations and topographic elevations at the site.

2.2.7 A fault trench investigation was conducted, and the fault trench was backfilled and compacted as engineered fill. The fault trench investigation report is included in Appendix E of this report. The compaction test report for backfill of the fault trench is included in Appendix F of this report. A visual site reconnaissance and subsurface geotechnical exploration were conducted.

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

2.2.9 Ms. Christine Harris (Sevan Multi-Site Solutions) was consulted during the investigation.

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

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

3.0 BACKGROUND INFORMATION

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

3.1 Site Description: The Circle K site is located at the northwest corner of Twentynine Palms Highway (also known as State Highway 62) and Avalon Avenue in Yucca Valley, San Bernardino, California. Based on our review of the Site Plan Exhibit CSP 6.0W, prepared by Greenberg Farrow, dated October 16, 2023, the Circle K net site area is 1.765 acres. Including the 0.229-acre right-of-way dedication, the 0.313-acre right-of-way vacation and 4.679-acre remaining parcel area, the total Circle K gross site area is 6.986 acres. The geotechnical engineering investigation discussed in this report was conducted within the proposed property lines for the Circle K development shown on the Site Plan Exhibit CSP 6.0W and also included a boring on the remaining parcel area for a driveway entrance to the site from Diadem Drive located northwest of the Circle K project area. A site location map is presented on Drawing No. 1 in Appendix A of this report. The locations of the borings drilled for the proposed Circle K development discussed in this report are shown on Drawing No. 2 in Appendix A of this report.

The area of the proposed Circle K development discussed in this report is bound to the north by vacant land and Paxton Road beyond, to the northeast by Avalon Avenue, to the southeast by Twentynine Palms Highway (also known as State Highway 62), to the southwest by vacant land and a storage facility beyond, and to the northwest by Diadem Drive.

At the time of our field exploration, the ground surface sloped gently down to the north and northwest and was generally covered by soil, some scattered trash, Creosote bushes and scattered Yucca or Joshua Trees (shown and referred to on the ALTA/NSPS Land Title Survey Map as Yucca Trees which are similar but slightly different than Joshua Trees). For the purpose of this report, these trees are referred to herein after as Yucca Trees. Overhead power lines cross the site in many areas. One set of overhead power lines trends across the site parallel to the northern part of the proposed fuel canopy. Just beyond the east end of the proposed fuel canopy is a power pole and utility box, and the overhead power lines diverge from this power pole in three directions to the north, northeast and southeast. Utility boxes were noted between Avalon Avenue and the Circle K northeastern property line. This is worth noting as an underground electric line was marked as trending east between the utility box on the east side of the proposed fuel canopy to another utility box near Avalon Avenue. The ALTA/NSPS Land Title Survey Map shows an underground water line trending north to south in the eastern portion of the Circle K site within the assumed Right-of-Way Vacation area. However, this water line was not marked by any members of Underground Service Alert during our field exploration; thus, it may be a privately owned line.

The ALTA/NSPS Land Title Survey Map indicates the site ranges in elevation from about 3,210 feet in the northern corner of the site to about 3,221 feet in the southern corner of the site. The area of the proposed driveway that extends to Diadem Drive ranges in elevation from about 3,212 feet to 3,215 feet.

3.2 Site History and Previous Studies: For site history information, satellite images of the site between the years 1996 and 2024 from online sources, were reviewed. In addition, historical aerial photographs of the site from various years between 1948 and 2016 were reviewed from a February 3, 2023 report entitled: "Phase I Environmental Site Assessment, Proposed Circle K Store - Imperial, Northwest Corner of Twentynine Palms Highway and Avalon Avenue, Yucca Valley, California 92284," prepared by Moore Twining's Environmental Division.

The earliest aerial photo of the site from 1948 shows the site as vacant land with Twentynine Palms Highway bordering the southeast side of the site. The site is shown as being covered by soil and scattered bushes and Yucca Trees. The 1952 aerial image of the site appears similar; however, Twentynine Palms Highway appears to have been widened, and Paxton Road located north of the site appears to be an unpaved dirt road. The next available 1970 aerial photograph of the site still shows the site as being vacant land, and other roads bordering the overall Circle K site (that includes the remaining parcel) are now all shown as being paved, including Avalon Avenue on the east side of the site, Paxton Road to the north and Diadem Drive to the northwest and west. The 1975, 1983, 1989, 1995, 2005, 2009, 2012 and 2016 aerial photographs all show the site as still being vacant land with areas surrounding the site being developed over time. Based on our review of the historical aerial images and satellite images from online sources, sometime between the 1995 and 2003, the configuration of Avalon Avenue on the east side of the site was changed from trending north to south in a straight line to curving and trending northwest to southeast on the northeast side of the Circle K site.

A report entitled, "Fault Investigation, Proposed Retail Center, NWC Avalon Avenue and Palisade Drive, APN 601-201-41, Yucca Valley, California," dated January 20, 2017, prepared by Sladden Engineering, was reviewed.

This report was provided by Mr. Matt Cohrt (Sladden Engineering) who was the third party reviewer that was hired by the City of Yucca Valley to review the fault trench investigation that was performed at the Circle K project site by Hilltop Engineering Inc. Sladden Engineering had previously conducted a fault trench investigation across the street from the proposed Circle K building site, on the opposite side of Twentynine Palms Highway, and was trenched within the same Earthquake Fault Zone (formerly referred to as the Alquist-Priolo Special Studies Zone) where a portion of the Eureka Peak Fault is mapped and informally named the "Avalon Avenue Splay" (CDMG, 1993a). A summary of Sladden Engineering's fault trench study is included below.

Sladden Engineering's area studied was on the west side of Avalon Avenue and southeast side of Twentynine Palms Highway. The site is roughly an ellipsoid-shaped site with the longer length of the site trending north to south. During Sladden's site reconnaissance, they indicated the following, "*Based on the site reconnaissance and literature review, the site is judged to be underlain entirely by alluvium. No evidence of faulting or secondary seismic effects (e.g. lurching, lateral spreading, subsidence, etc.) was observed during our field reconnaissance.*" The report discussed the regional geologic setting and regional faulting, indicating that the regional faulting generally includes north to northwest trending faults that have generated damaging earthquakes such as the June 28, 1992 Moment Magnitude (M_W) 7.3 Landers Earthquake. The report also discusses the Pinto Mountain Fault Zone which the California Geologic Survey maps as a dashed line (concealed fault) trending roughly parallel with Paxton Road at the north end of the Circle K remaining parcel area. Where the Pinto Mountain fault is mapped parallel with Paxton Road at the north end of the Circle K remaining parcel area, the Pinto Mountain fault is not shown as being included within an Earthquake Fault Zone (formerly known as Alquist Priolo Special Studies Zone).

Sladden's fault investigation report indicated, "The closest known active fault to the proposed development is the informally named "Avalon Avenue Splay". The report describes the "Avalon Avenue Splay" as follows: "*A well defined zone of cracks, located near Avalon Avenue and Highway 62, were originally mapped by Rasmussen & Associates (1992) subsequent to the June 28, 1992 Landers Earthquake and later zoned by the State of California (Treiman, 1993). Treiman (1993) indicates that although this short northwest trending fracutre set is isolated and is not clearly related to any of the better defined 1992 fault rupture, the distinct left-stepping pattern is strongly suggestive of a right-lateral shear. The Avalon Avenue Splay is mapped to the west of the project site and the site is located within the Avalon Avenue Splay fault zone."*

The Sladden Engineering report indicated that their fault trench investigation for the "Avalon Avenue Splay" was conducted between December 12 and December 15, 2016. A total of three (3) trenches were excavated. The first trench trended N16E and had a length of 347 feet. Two other shorter step out trenches were excavated on the west side of the central portion of the first trench. The second trench trended N60E and had a length of 40 feet. The third trench was located northwest of the second trench, trended N36E and had a length of 76 feet. The second and third shorter step out trenches were noted in the report to have been excavated to address features report by PSI (2006). The trenches were all excavated to depths of about 10.5 to 11 feet below site grade.

The three (3) exploratory trenches were noted to encounter "*a sequence of materials that were classified as Quaternary alluvium (Qal) and Quaternary age older alluvium (Qoa)*." The Quaternary alluvium (Qal) included two silty sand units that were logged Qal and Qal2. The Quaternary older alluvium (Qoa) included seven (7) gravelly sand to silty sand units that were logged as Qoa through Qoa7. One of the Qoa silty sand units was noted to include some silt and clay beds. A unit of silty sand fill soils was also identified in the vicinity of Trench T-3 where trench backfill was encountered. The report indicated, "*No suitable materials for radiometric dating were encountered*

within the exploratory fault trench excavations on which to base an absolute age determination." However, the report indicated that "*Sladden retained the services of Mr. John Helms, a Soil Geochronologist and a State of California Certified Engineering Geologist to describe the exposed soil stratigraphy and to determine the relative age dates for some of the soils encountered at the site.*" The January 3, 2017 Soil Stratigraphy Study attached to Sladden's January 20, 2018 Fault Investigation report indicated that Quaternary alluvium (Qal) units ranged from 2,000 to 8,000 years old, and the underlying Quaternary older alluvium (Qoa) units ranged from 13,200 to 90,000 years old.

Sladden's Fault Investigation report concluded, "...***features indicative of active surface faulting were not encountered within the exploratory fault trenches excavated***." Sladden's study excavated a single exploratory trench (T-1) extending from near the southern property boundary through the eastern portion of the mapped fault zone (the western half of the mapped fault zone extends southwest of the property they studied). Sladden's report also concluded, "Evidence of faulting was not found to exist within the fault trenches excavated. An unbroken layered sequence of Quaternary alluvial materials was observed with laterally continuous contacts." The report also concluded, "*Because it remains unknown if faulting exists to the southwest of the property, a structural setback line delineating a restrict use zone (RUZ) should be offset 50 feet from the southern end of Trench T-1 and oriented parallel to the State of California established fault zone."* The approximate RUZ was shown on one of the figures in Sladden's Fault Investigation report. The report further indicated, "*Structures intended for habitable use should not be constructed within the RUZ. The RUZs should be incorporated into the design and construction of the project*."

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 Anticipated Construction: Based on our review of the referenced site plan, the proposed development is anticipated to include a Circle K store building (5,200 square feet) in the northern portion of the site, a fueling island with canopy is planned southeast of the Circle K store, and an underground storage tank area is planned southeast of the fuel canopy. Appurtenant construction is anticipated to include concrete walkways, asphalt concrete and Portland cement concrete parking and drive areas, a trash enclosure, underground utilities, and landscaped areas.

It is anticipated that the proposed Circle K structure will consist of a one-story building including wood or concrete masonry unit wall construction with concrete slab-on-grade floors. It is anticipated that the proposed building will be supported on shallow foundation systems. The fuel canopy is anticipated to be supported on a cast-in-drilled-hole foundation. Basements and loading docks are not anticipated as part of the proposed construction. In addition, storm water infiltration systems are being considered as part of the proposed construction.

It is anticipated that the Circle K structure may have maximum column loads of about 40 kips, maximum perimeter wall loads of 3.5 kips per linear foot and a floor load of 175 pounds per square foot.

Based on our review of the ALTA/NSPS Land Title Survey Map, prepared by Duryea & Associates, Inc., dated April 9, 2024, the existing site grades in the area of the Circle K building footprint gently fall to the north and northwest and range from about 3,214½ feet in the northeast corner to about 3,216½ feet in the southwest corner. Thus, cuts and fills of about 1 to 2 feet are generally anticipated to achieve planned site grades.

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 fault trench investigation, site reconnaissance, drilling test borings, conducting standard penetration tests, soil sampling and percolation testing.

4.1.1 Fault Trench Investigation: The project site is located in an Alquist-Priolo Earthquake Fault Zone for the Eureka Peak Fault. The Fault Activity Map of California, prepared by the California Geological Survey, shows an active fault as being about 200 feet southwest of the Circle K site. The eastern portion of the Earthquake Fault Zone for this fault overlaps the western portion of the Circle K parcel. This short fault segment is unnamed but indicated by the California Geological Survey on the Fault Activity Map of California to be part of the Eureka Peak Fault Zone. This short fault segment of the Eureka Peak Fault Zone was informally named the "Avalon Avenue Splay" (CDMG, 1993a). The eastern edge of the Earthquake Fault Zone for the "Avalon Avenue Splay" fault trace of the Eureka Peak Fault Zone is roughly parallel with the western edge of the proposed Circle K store. Thus, a fault investigation is required.

Moore Twining Associates, Inc. retained Hilltop Engineering, Inc. to conduct a fault trench investigation across the proposed Circle K building footprint in July and August 2024 and prepare a fault trench investigation report and compaction report for backfill of the fault trench. This section provides a brief description of the fault trench activities. More detailed descriptions of the fault trench investigation are provided in the fault trench investigation report, prepared by Hilltop Geotechnical, Inc. (Hilltop) and included in Appendix E of this report. The fault trench was excavated between July 22, 2024 and July 29, 2024 and then logged by a Professional Geologist and Certified Engineering Geologist from Hilltop during and shortly after the completion of the excavation of the fault trench. The fault trench was excavated to a depth of about 12 feet below surrounding site grades. The trench was excavated with a series of stair-steps on the northwest and southeast sides of the trench that included four (4) separate 3-foot vertical cuts with benches in between the cuts that were each about 4 feet in width. The overall fault trench was approximately 30 feet in width. The fault trench was excavated throughout the inside of the proposed Circle K

building footprint and extended roughly parallel with the 113-foot long length of the proposed Circle K building and roughly perpendicular to the trend of the Earthquake Fault Zone for the "Avalon Avenue Splay" fault trace of the Eureka Peak Fault Zone. The fault trench extended at least 50 feet beyond each end of the proposed Circle K building footprint (at least 213 feet in total length). When shadowing the fault trench in a southeastward direction toward the proposed fuel canopy, the fault trench also extended at least 20 feet beyond the foundations at the outer ends of the proposed fuel canopy which was deemed to be acceptable by the third party reviewer hired by the City of Yucca Valley when discussing the fault trench findings with Moore Twining at the site on August 1, 2024. To extend the fault trench at least 50 feet beyond each end of the long length of the proposed Circle K building, measurements were made from building corner stakes that were staked by professional surveyors hired by Circle K Stores, Inc.

At station 200+00 in the eastern portion of the trench, a Yucca Tree was in the way of extending the fault trench further to the northeast. Thus, the fault trench was stepped to the southeast near Station 200+00, and a separate short trench (stair-stepped in the same manner as the longer trench) was excavated parallel to the longer trench in order to excavate at least 213 feet of trench and extend the trench at least 50 feet beyond each long length of the proposed Circle K building footprint. The short trench and the long trench overlapped each other by approximately 5 feet to ensure that the logging of the soil strata within the trench was continuous. The City of Yucca Valley hired a thirty party reviewer (Mr. Matt Cohrt, Certified Engineering Geologist with Sladden Engineering) to meet Moore Twining at the site to review the fault trench findings and fault trench investigation report. Thus, on August 1, 2024, Mr. Allen Harker, Certified Engineering Geologist with Moore Twining, met with Mr. Cohrt with Sladden Engineering to review the findings of the fault trench that was logged by the geologists from Hilltop. An unbroken layered sequence of Quaternary alluvial materials was observed with laterally continuous contacts. The trench also identified units of younger alluvium (Qa) overlying units of older alluvium (Qoa). The fault trench report by Hilltop indicated that in the building pad area, the younger alluvium (Qa) extended from the ground surface to about 1 to 6 feet below site grade (BSG). Mr. Harker and Mr. Cohrt concurred with the geologists from Hilltop that there were no features indicative of active surface faulting within the exploratory fault trenches excavated.

Between August 6 and August 23, 2024, the fault trench was backfilled as certified, compacted engineered fill, and on August 24, 2024, the fault trench area was graded to restore the area to the existing drainage pattern at the site (sloping gently down to the north/northwest). The soils excavated from the trench were moisture conditioned and generally placed in 8-inch lifts and compacted using a sheepsfoot wheel roller attached to a John Deer 85G backhoe and also by the large rubber-tire wheels of the backhoe. A larger John Deer 544 front loader was later used for compaction of the fill soils. A representative of Hilltop was on-site during the entire duration of the backfill of the fault trench to observe and test the backfill that was placed. Two samples representative of the soils excavated from the fault trench were tested by Hilltop to obtain the

maximum density and optimum moisture contents to use for testing the relative compaction of the backfill. The higher of the two maximum dry densities (126.5 pounds per cubic foot) was used for the duration of the testing of the backfill. The backfill was tested to achieve a minimum of 90 percent relative compaction and the tests were all slightly below to slightly above the optimum moisture content of the soil. In general, a minimum of two tests were taken each day during the backfill procedures. The testing also generally included a minimum of 3 tests across the length of the tench for every 2 feet of fill placed vertically. A Moore Twining soils technician also visited the site periodically for a few hours during of the initial days of backfilling the trench on August 6, August 9 and August 14, 2024; and a Moore Twining Area Supervisor visited the site on the final day of backfilling on August 23, 2024. Hilltop's compaction report is included in Appendix F of this report.

4.1.2 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by a Moore Twining engineering geologist on August 1, 2024 and by a staff engineer of Moore Twining during the field exploration between August 16 and 18, 2024. The features noted are described in the background information section of this report.

4.1.3 Drilling Test Borings: Prior to drilling, the site was marked for Underground Service Alert for members to mark out the locations of member utilities. In addition, a boring permit WP0039485 was obtained from San Bernardino County Public Health Environmental Health Services.

The depths and locations of the test borings were selected based on the client provided criteria in the Circle K Appendix A - Standard Scope of Work, December 3, 2019, the size of the structures, type of construction, estimated depths of influence of the anticipated foundation loads, and the subsurface soil conditions encountered at the site.

Between August 16 and 18, 2024, ten (10) test borings (borings B-1 through B-9 and percolation test boring P-2), were drilled to depths ranging from about 10 to 50 feet BSG. Percolation test borings B-2/P-1 and B-5/P-3 were utilized to drill 10-foot borings for proposed drive-entry areas and also conduct percolation tests. Percolation test P-2 was drilled to a depth of 40 feet BSG. Percolation test borings P-2 was originally planned within a proposed landscaped area between the proposed fuel canopy and Avalon Avenue. However, due to the markings of an underground electric line and surrounding overhead power lines, the boring could not be drilled in a safe location and was moved to between the proposed fuel canopy and Twentynine Palms Highway as shown on Drawing No. 2 in Appendix A of this report. It should also be noted that the fault trench, excavated across the southern and central portions of the building pad area was still open at the time of our field exploration and not entirely backfilled. It was also desired to drill the borings for the building footprint where soils had not been disturbed by the fault trenching activities. Thus, borings B-6 and B-7 were drilled in the northern portion of the building footprint near the northeast and northwest

building corners as 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 with 6⅝-inch and 8-inch outside diameter (O.D.) hollow-stem augers. The approximate locations of the test borings and percolation test borings drilled for the proposed Circle K development are shown on Drawing No. 2 in Appendix A of this report.

During the drilling of the test borings, bulk samples of soil were obtained for laboratory testing. The test borings were logged during drilling by a Moore Twining staff engineer The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

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

Test boring locations were determined with reference to existing site features shown on the site plan. In accordance with the requirements of our boring permit with San Bernardino County Public Health Environmental Health Services, the borings were all backfilled with neat cement at the completion of the drilling and percolation testing activities. The neat cement backfill was then topped with soil cuttings in the upper few feet; thus, some settlement should be anticipated at the boring locations. The excess soil cuttings from the borings were spread near the boring locations.

4.1.4 Soil Sampling: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

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

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

4.1.5 Percolation Tests: Percolation testing was conducted on August 18, 2024. Percolation test borings B-2/P-1 and B-5/P-3 were both drilled to a depth of about 10 feet BSG and percolation test boring P-2 was drilled to a depth of about 40 feet BSG on August 17, 2024.

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

On the day prior to the percolation tests, the percolation test holes were presoaked with water. The presaturation water in the percolation test holes had completely seeped into the soil by the next morning. The percolation tests were then conducted in accordance with the percolation test method criteria in 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. Prior to starting the percolation tests, the holes were then refilled with about 22 to 32 inches of water to conduct the percolation tests so that they were filled with the minimum amount of required water (at least 20 inches) which was at least five times the radius of the holes. In accordance with the percolation test method criteria, two measurements were then made that confirmed that at least 6 inches of water had seeped into the soil in less than 25 minutes for the sandy soils method. Measurements were then take for an additional hour. While the test method discusses taking measurements every 10 minutes, the measurements had to be taken every 2 minutes due to the relatively rapid drop in water level. The holes were refilled with at least 20 inches of water prior to the one hour of measurements and the water level was also refilled to the same level after each 2 minute reading during the one hour of measurements. No collapse of the boreholes occurred during the percolation tests. The percolation rate had stabilized during the final measurements for each percolation test. Per the sandy soil test method, the final reading is used to calculate the percolation rate.

After the percolation tests were completed, the percolation test pipe was removed from the percolation test holes, and the holes were backfilled with neat cement and topped with soil cuttings in the upper few feet; thus, some settlement should be anticipated at the percolation test boring locations.

4.2 Laboratory Testing: 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 field exploration and laboratory testing are summarized in the following subsections.

5.1 Site Geology: Based on our review of the Geologic Map of the Joshua Tree Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-516, Scale 1:62,500, dated 1967, prepared by Thomas W. Dibblee, the site is mapped as being underlain by Pleistocene and Recent (Holocene-age) alluvium (Qa). On the opposite side (southeast side) of Twentynine Palms Highway, across the street from the site, the ground surface is mapped as being underlain by Pleistocene age older alluvium (Qoa). Both the younger alluvium (Qa) and underlying older alluvium (Qoa) were encountered below the subject site during the fault trench investigation and borings drilled for the field exploration. The fault trench report by Hilltop indicated that in the building pad area, the younger alluvium (Qa) extended from the ground surface to about 1 to 6 feet below site grades (BSG).

5.2 Soil Profile: The soils encountered in the borings drilled for this investigation generally consisted of silty sands with varying amounts of gravel extending from the ground surface to the maximum depth explored, about 50 feet BSG. Some near surface or deeper layers of poorly graded sand with silt and well graded sand with silt (both with varying amounts of gravel) were also encountered. It should be noted that the upper 2½ to 3½ feet of soils encountered in borings B-6 and B-7 in the Circle K building pad area were loose. Borings B-6 and B-7 were drilled in the northern portion of the building footprint, north of the fault trench and in soils that were undisturbed from the fault trench activities.

Fill soils were encountered in one boring (B-1) that was drilled at the location of the proposed underground storage tanks. Samples collected at depths of 0 to 1½ feet and 3½ to 5 feet in boring B-1 both had cloth material in the samples. The next sample collected from depths of 8½ to 10 feet BSG and did not encounter any cloth or show any other indications that the soil was fill material. Thus, the fill noted on the log for boring B-1 is shown as extending from the ground surface to a depth of about 8½ 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.

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

Silty Sands [SM]: The silty sands encountered had varying amounts of gravel and were described as loose to very dense, as indicated by standard penetration resistance, N-values, ranging from 4 to greater than 50 blows per foot. The moisture content of the samples tested ranged from about 0.2 to 5.4 percent. Seven (7) relatively undisturbed samples revealed dry densities of 120.6, 115.3, 116.0, 109.8, 116.2, 115.7 and 115.1 pounds per cubic foot. An Atterberg Limits test conducted on a sample collected from depths of 0 to 3½ feet from boring B-6 (building pad boring) indicated that the sample was non-plastic. A consolidation test conducted on a sample collected from depths of about 2 to 3½ feet BSG from boring B-6 (proposed building pad area) indicated the sample exhibited low compressibility characteristics (about 2.7 percent consolidation under a load of 8 kips per square foot). Upon inundation, the sample exhibited negligible collapse potential (0.0 percent collapse when wetted under a load of 0.5 kips per square foot). A consolidation test conducted on a sample collected from depths of about 5 to 6½ feet BSG from boring B-6 (proposed building pad area) indicated the sample exhibited low compressibility characteristics (about 1.6 percent consolidation under a load of 8 kips per square foot). Upon inundation, the sample exhibited negligible collapse potential (0.0 percent collapse when wetted under a load of 0.5 kips per square foot). A consolidation test conducted on a sample collected from depths of about 1½ to 3 feet BSG from boring B-7 (proposed building pad area) indicated the sample exhibited low compressibility characteristics (about 2.1 percent consolidation under a load of 8 kips per square foot). Upon inundation, the sample exhibited negligible collapse potential (about 0.0 percent collapse when wetted under a load of 0.5 kips per square foot). A direct shear test conducted on a sample collected from depths of 5 to 6½ feet BSG from boring B-4 (proposed fuel canopy area) indicated an internal angle of friction of 38 degrees and 370 pounds per square foot of cohesion. A direct shear test conducted on the a sample collected from depths of 1½ to 3 from boring B-7 (proposed building pad area) indicated an internal angle of friction of 42 degrees and 240 pounds per square foot of cohesion.

Poorly Graded Sands with Silt [SP-SM] and Well Graded Sands with Silt [SW-SM]: The poorly graded sands with silt and well graded sands with silt encountered (both with varying amounts of gravel) were loose to dense, as indicated by standard penetration resistance, N-values, ranging from 5 to 35 blows per foot. The moisture of a near surface poorly graded sand with silt sample was about 0.5 percent. One (1) relatively undisturbed sample revealed a dry density of 113.0 pounds per cubic foot.

Expansion Index Tests: An expansion index test was conducted on a near surface silty sand sample collected at depths of about 0 to 3½ feet BSG from boring B-6 (proposed Circle K building area) which indicated an expansion index value of 0.

Maximum Density/Optimum Moisture Determination: A maximum density/optimum moisture determination test conducted on a sample collected from boring B-8 at a depth of 0 to 3½ feet BSG indicated a maximum dry density of 127.7 pounds per cubic foot at an optimum moisture content of 7.3 percent.

R-value Test: The results of two (2) R-value tests conducted on the near surface silty sand samples indicated R-values of 74 and 72.

Chemical Tests: Chemical tests performed on a near surface silty sand sample collected from depths of 0 to 3½ feet BSG from boring B-6 (building pad area) indicated a pH value of 8.1; a minimum resistivity value of 9,200 ohms-centimeter; 0.0033 percent by weight concentration of sulfate; and 0.0018 percent by weight concentration of chloride.

5.4 Groundwater Conditions: Groundwater was not encountered in the borings to the maximum depth explored, about 50 feet BSG.

Based on our review of groundwater data from wells on the Department of Water Resources website, a well located about ½ mile west of the Circle K site indicates groundwater has ranged from about 127 feet in 2012 to about 324 feet in 2008 for dates measured between the years 1980 and 2012.

The referenced January 20, 2017 fault investigation report by Sladden Engineering described in Section 3.2 of this report indicated, "Groundwater depths available through DWR (2017) report groundwater at depths greater than 100 feet in the site vicinity."

Based on our review of the California State Water Resources Control Board Geotracker website, no monitoring well reports were found on the website with groundwater data in the vicinity of the subject site.

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

5.5 Results of Percolation Testing: The infiltration rate estimated from the percolation test data is summarized in Table No. 1 below. The percolation test data are 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
B-2/P-1 at 10 feet BSG	9.1	Silty Sand
P-2 at 40 feet BSG	11.2	Poorly Graded Sand with Silt
B-5/P-3 at 10 feet BSG	5.8	Silty Sand

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

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

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 site was generally covered by soil and some scattered trash, Creosote bushes and scattered Yucca Trees. Overhead power lines crossed the site in many areas.

One set of overhead power lines trends across the site parallel to the northern part of the proposed fuel canopy. It is anticipated that this set of overhead power lines will need to be relocated.

Just beyond the east end of the proposed fuel canopy is a power pole and utility box, and the overhead power lines diverge from this power pole in three directions to the north, northeast and southeast. Utility boxes were noted between Avalon Avenue and the Circle K northeastern property line. This is worth noting as an underground electric line was marked as trending east between the utility box on the east side of the proposed fuel canopy to another utility box near Avalon Avenue. It is anticipated that this underground electric line will need to be removed and relocated during site preparation. The ALTA/NSPS Land Title Survey Map shows an underground water line trending north to south in the eastern portion of the Circle K site within what is shown on the Circle K site plan as the "Assumed R-O-W Vacation" area. However, this water line was not marked by any members of Underground Service Alert during our field exploration; thus, it may be a privately owned line. Underground utilities to be removed should not be capped or crushed and buried in-place. Where underground utilities are encountered in the project area and scheduled to be removed, they should be fully removed including all utility trench backfill. Contractors should assume that the utility trench backfill extends from below the utility to the ground surface at a 1 horizontal to1 vertical gradient.

The soils at the site typically have very low moisture contents and will require significant moisture conditioning and mixing of soils to ensure a uniform moisture content prior to placement and compaction of soils as engineered fill.

One of the borings (B-1) where the underground fuel storage tanks are planned encountered fill soils with cloth in the soil samples collected at the surface and between the depths of 3½ to 5 feet BSG. Native soils with no cloth were encountered at the next sample depth at 8½ feet BSG. Thus, fill soils with some buried debris (cloth and possibly other items) are expected to be encountered as extending to depths of about 5 to 8½ feet BSG in the area of the proposed underground fuel storage tanks. No fill soils were encountered in any of the other borings. There was some scattered trash along the ground surface in the area of boring B-1. However, the ground surface did not appear to be disturbed

in the area of boring B-1. Thus, the limits of the fill extending beyond the location of boring B-1 are unknown. When soils are excavated during site preparation, any debris such as the cloth encountered in boring B-1 should be removed from the site and not mixed with soils to be reused as engineered fill. During the excavation for the underground fuel storage tanks, if it appears that the fill soils with cloth debris or any other debris extend horizontally beyond the limits of the proposed underground fuel storage tanks, then additional removals will be required to remove the fill soils in other areas of planned improvements that may be sensitive to settlement.

However, as previously noted in this report, a fault trench was excavated through the central and southern portion of the building pad area and extended 50 feet beyond the western and eastern ends of the proposed building footprint. The fault trench was excavated because the Circle K site is located in the eastern portion of a mapped Earthquake Fault Zone for the "Avalon Avenue Splay" fault trace of the Eureka Peak Fault Zone. At the time of our field exploration, the fault trench was being backfilled with full-time observation and density testing of the backfill to ensure that the backfill was moisture conditioned and compacted to at least 90 percent relative compaction. A compaction report was also provided by Hilltop that certified the fill as engineered fill. Borings for the building footprint (borings B-6 and B-7) were drilled where native undisturbed soils existed on the north side of the fault trench. Loose near surface soils were encountered in these borings in the northern portion of the building pad area in the upper 2½ to 3½ feet BSG. In order to provide a uniform blanket of engineered fill across the building pad, this report recommends that the entire building pad be over-excavated to at least 4 feet below preconstruction site grades and to the depth required to provide at least 2 feet of engineered fill below bottom of footings, whichever is greater.

As part of site grading, existing vegetation will need to be stripped to the depth required to remove root structures and excess organics. Roots greater than ¼-inch in diameter should be removed from the soils to be reused as engineered fill during site preparation. Roots greater ¼-inch in diameter and stripped grasses and organics will not be considered suitable for reuse as engineered fill.

The site has several Yucca Trees (similar looking to Joshua Trees). It is anticipated that the Yucca Trees may need to be moved out of the project area and replanted so that they are saved. Circle K should consult with the City of Yucca Valley and a "desert native plant specialist" regarding the requirements for relocation or restoration of native California desert vegetation at the project site.

6.2 Expansive Soils: In evaluation of the potential for expansive soils at the site, expansion index testing was performed on representative samples of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The expansion index testing was performed in accordance with ASTM D4829 and the results are included in Appendix C of this report. The result of the expansion index testing indicated that the near surface silty sand soils tested in the proposed Circle K area are non-plastic and have a very low expansion potential based on the result of expansion index (EI) testing (EI of 0). Although the on-site soils have a very low expansion potential, this report recommends to that interior and exterior concrete slabs-on-grade and Portland cement concrete pavements be underlain by 4 inches of aggregate base for constructability purposes.

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

The near surface soils tested have low compressibility characteristics and negligible collapse potential but the upper soils were loose to depths of about 2½ to 3½ feet BSG in the borings drilled in the proposed Circle K building footprint. Thus, in order to provide a uniform blanket of engineered fill across the building pad and limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet, this report recommends that the entire building pad be over-excavated to at least 4 feet below preconstruction site grades and to the depth required to provide at least 2 feet of engineered fill below bottom of footings, whichever is greater. Provided the building pads are prepared in accordance with the recommendations presented in this report, a net allowable soil bearing pressure of 3,000 pounds per square foot, for dead-plus-live loads, may be used for design.

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

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

6.4 Seismic Ground Rupture and Design Parameters: The project site is located in an Alquist-Priolo Earthquake Fault Zone for the informally named "Avalon Avenue Splay" fault trace within the Eureka Peak Fault Zone. The Fault Activity Map of California, prepared by the California Geological Survey, shows a short segment of the Eureka Peak Fault Zone as being about 200 feet southwest of the Circle K site. The eastern edge of the Earthquake Fault Zone for the "Avalon Avenue Splay" fault trace of the Eureka Peak Fault Zone is roughly parallel with the western edge of the proposed Circle K store. Thus, a fault investigation was conducted that is briefly discussed in Section 4.1.1 of this report, and the fault trench investigation report is included in Appendix E of this report. The fault trench was excavated throughout the central and southern portions of the building pad and extended 50 feet beyond each end of the long length of the proposed

Circle K building footprint (which was roughly perpendicular to the mapped orientation of the "Avalon Avenue Splay" fault trace for the Eureka Peak Fault Zone. The fault trench revealed no features indicative of active surface faulting within the exploratory fault trenches excavated. Accordingly, the potential for ground rupture associated with any active faulting within the proposed building pad area and extending 50 feet beyond the long length of the building pad area is considered low. When shadowing the fault trench in a southeastward direction toward the proposed fuel canopy, the fault trench also extended at least 20 feet beyond the foundations at the outer ends of the proposed fuel canopy which was deemed to be acceptable by the third party reviewer hired by the City of Yucca Valley. Thus, the potential for ground rupture associated with any active faulting within the proposed fuel canopy area and extending at least about 20 feet beyond the foundations at the outer ends of the proposed fuel canopy area is also 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. The site is in a very active seismic region. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 1.055g was determined for the site using the Ground Motion Parameter Calculator provided by the Structural Engineers Association of California website (https://seismicmaps.org/).

6.5 <u>Liquefaction and Seismic Settlement</u>: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing can result. Fine, well sorted, loose sand, shallow groundwater conditions, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction. One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils; however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by building pad boring B-6 (drilled to 50 feet BSG) in accordance with the NCEER methods (Youd, et al.) using the computer program LiquefyPro, developed by CivilTech Software. Liquefaction and seismic settlement analysis were also conducted based on the soil properties from building pad boring B-7 (drilled to 15 feet BSG) and using the soil properties from building pad boring B-6 from depths of 15 to 50 feet BSG. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 1.055g was determined for the site using the Ground Motion Parameter Calculator provided by SEOAC and OSHPD (http://seismicmaps.org). A Maximum Considered Earthquake magnitude of 6.51 was applied in the analysis based on deaggregation analysis (United States Geological Survey deaggregation website, Dynamic Conterminous U.S. 2014, Update). A groundwater depth of 127 feet was used for the analysis based on historical groundwater data from a nearby well.

Due to the depth of groundwater being greater than 50 feet, the potential for liquefaction to occur is low. 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. The seismic settlements were estimated to be about 1½ inches total and ¾ inches differential in 40 feet.

In the event the estimated seismic settlement is considered an impact to the structure design, a supplemental investigation using Cone Penetration Tests (CPTs) may be desirable to allow for a more detailed analysis of seismic settlement. CPTs provide nearly continuous soil penetration resistance information as compared with the typical sampling interval (about 5 feet) for soil borings. Therefore, the CPTs provide more data for use in refining the estimated seismic settlement.

In the event the differential seismic settlement is considered excessive for design of a conventional spread foundation system, alternative methods of site preparation or foundation design could be considered for the project. Based on our experience with other projects considering the size of the building, structural mitigation such as an interconnected gradebeam foundation system would be anticipated to be the most cost effective approach, if special mitigation is required. This report assumes the foundations can be designed for support of shallow spread foundations. In the event conventional shallow spread foundations cannot be designed to tolerate the anticipated seismic settlements, Moore Twining should be contacted to provide alternative recommendations.

6.6 Asphalt Concrete (AC) Pavements: Recommendations for asphalt concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphalt concrete (AC) pavements. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highway Design Manual. The analysis was based on traffic index values ranging from 5.0 to 10.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

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

Based on the results of the testing included in this report (R-values of 72 and 74), the procedures of the Caltrans Highway Design Manual, an R-value of 50 was used to provide the pavement section thickness recommendations.

6.7 Portland Cement Concrete (PCC) Pavements: Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the 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 loading which is equivalent to traffic indexes ranging from 6 to 10. The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements." Considering a minimum 4-inch layer of aggregate base material (minimum R-value of 78), a modulus of subgrade reaction, K-value, of 230 psi/in at the top of the aggregate base was used for pavement design.

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

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the severity ratings listed in 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 "moderately corrosive" corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the "moderately corrosive" corrosion potential. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

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

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

Table No. 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 soils encountered generally consisted of loose to very dense silty sands with varying amounts of gravel extending from the ground surface to a depth of 50 feet BSG. Some near surface or deeper layers of loose to dense poorly graded sand with silt and well graded sand with silt (both with varying amounts of gravel) were also encountered. It should be noted that the upper 2½ to 3½ feet of soils encountered in borings B-6 and B-7 in the Circle K building pad area were loose. Borings B-6 and B-7 were drilled in the northern portion of the building footprint, north of the fault trench and in soils that were undisturbed from the fault trench activities.

7.3 Silty sand fill soils with cloth debris were encountered at the site in boring B-1 where the underground fuel storage tanks are planned as described in Section 6.1 of this report. The fill soils are shown on the boring log for boring B-1 as extending to a depth of about 8½ feet BSG. It is our understanding that excavations for the proposed underground fuel storage tanks typically extend to a depth of about 20 feet BSG. Thus, it is anticipated that the fill soils will be removed as part of the excavation for the proposed underground fuel storage tanks. Any cloth or other debris encountered in the fill soils during the excavation for the underground fuel storage tanks should be removed from the site and not mixed with any soils to be used as engineered fill for the site. During the excavation for the underground fuel storage tanks, if it appears that the fill soils with cloth debris or any other debris extend beyond the limits of the proposed underground fuel storage tanks, then additional removals will be required to remove the fill soils in other areas of planned improvements that may be sensitive to settlement.

7.4 The near surface granular soils encountered are non-plastic and have a very low expansion potential, low compressibility characteristics and negligible collapse potential, high shear strength characteristics and excellent support characteristics for pavements when compacted as engineered fill.

7.5 Although the on-site soils have a very low expansion potential, this report recommends that interior and exterior concrete slabs-on-grade and Portland cement concrete pavements be underlain by 4 inches of aggregate base for constructability purposes.

7.6 Groundwater was not encountered in the borings to the maximum depth explored, about 50 feet BSG. Based on our review of groundwater data from wells on the Department of Water Resources website, a well located about ½ mile west of the Circle K site indicates groundwater has ranged from about 127 feet in 2012 to about 324 feet in 2008 for dates measured between the years 1980 and 2012.

7.7 Due to the depth of groundwater being greater than 50 feet, the potential for liquefaction to occur is low. 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. The seismic settlements were estimated to be about 1½ inches total and ¾ inches differential in 40 feet.

In the event the estimated seismic settlement is considered an impact to the structure design, a supplemental investigation using Cone Penetration Tests (CPTs) may be desirable to allow for a more detailed analysis of seismic settlement. CPTs provide nearly continuous soil penetration resistance information as compared with the typical sampling interval (about 5 feet) for soil borings. Therefore, the CPTs provide more data for use in refining the estimated seismic settlement.

In the event the differential seismic settlement is considered excessive for design of a conventional spread foundation system, alternative methods of site preparation or foundation design could be considered for the project. Based on our experience with other projects considering the size of the building, structural mitigation such as an interconnected gradebeam foundation system would be anticipated to be the most cost effective approach, if special mitigation is required. This report assumes the foundations can be designed for support of shallow spread foundations. In the event conventional shallow spread foundations cannot be designed to tolerate the anticipated seismic settlements, Moore Twining should be contacted to provide alternative recommendations.

7.8 The near surface soils tested exhibited low compressibility characteristic but loose silty sands were encountered in building pad borings B-6 and B-7 as extending to depths of about 2½ to 3½ feet BSG in the northern portion of the building pad. The fault trench was excavated in the central and southern portions of the building pad. In order to provide a uniform blanket of engineered fill across the building pad and limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet, this report recommends that the entire building pad be over-excavated to at least 4 feet below preconstruction site grades and to the depth required to provide at least 2 feet of engineered fill below bottom of footings, whichever is greater.

7.9 The result at percolation test B-2/P-1 at 10 feet BSG indicated an unfactored infiltration rate of 9.1 inches per hour. The result at percolation test P-2 at 40 feet BSG indicated an unfactored infiltration rate of 11.2 inches per hour. The result at percolation test B-5/P-3 at 10 feet BSG indicated an unfactored infiltration rate of 5.8 inches per hour. The results indicate that storm water infiltration systems appear feasible for this site. This report recommends that the lower unfactored infiltration rate of 5.8 inches per hour be considered for use in design for infiltration systems 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. 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...."

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

7.11 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).

7.12 The project site is located in an Alquist-Priolo Earthquake Fault Zone for the informally named "Avalon Avenue Splay" fault trace within the Eureka Peak Fault Zone. The Fault Activity Map of California, prepared by the California Geological Survey, shows a short segment of the Eureka Peak Fault Zone as being about 200 feet southwest of the Circle K site. The eastern edge of the Earthquake Fault Zone for the "Avalon Avenue Splay" fault trace of the Eureka Peak Fault Zone is roughly parallel with the western edge of the proposed Circle K store. Thus, a fault investigation was conducted that is briefly discussed in Section 4.1.1 of this report, and the fault trench investigation report is included in Appendix E of this report. The fault trench was excavated throughout the central and southern portions of the building pad and extended 50 feet beyond each end of the long length of the proposed Circle K building footprint (which was roughly perpendicular to the mapped orientation of the "Avalon Avenue Splay" fault trace for the Eureka Peak Fault Zone. The fault trench revealed no features indicative of active surface faulting within the exploratory fault trenches excavated. Accordingly, the potential for ground rupture associated with any active faulting within the proposed building pad area and extending 50 feet beyond the long length of the building pad area is considered low. When shadowing the fault trench in a southeastward direction toward the proposed fuel canopy, the fault trench also extended at least 20 feet beyond the foundations at the outer ends of the proposed fuel canopy which was deemed to be acceptable by the third party reviewer hired by the City of Yucca Valley when discussing the fault trench findings with Moore Twining at the site on August 1, 2024. Thus, the potential for ground rupture associated with any active faulting within the proposed fuel canopy area and extending at least about 20 feet beyond the foundations at the outer ends of the proposed fuel canopy area is also considered low.

8.0 RECOMMENDATIONS

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

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

8.1 General

8.1.1 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 It is anticipated that the Circle K structure may have maximum column loads of about 40 kips, maximum perimeter wall loads of 3.5 kips per linear foot and a floor load of 175 pounds per square foot. 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 including this report are sufficient for the Contractor to provide an accurate bid. If the data are not sufficient to properly bid any aspect of the project, the Contractor shall notify Circle K Stores, Inc. in writing what aspect of the project documents including the geotechnical engineering investigation report is not sufficient to bid the project. The Contractor shall provide this notification in writing to Circle K Stores, Inc. prior to submitting their bid.

8.1.5 The soils at the site typically have very low moisture contents and will require significant moisture conditioning and mixing of soils to ensure a uniform moisture content prior to placement and compaction of soils as engineered fill.

8.1.6 The site has several Yucca Trees. It is anticipated that the Yucca Trees may need to be moved out of the project area and replanted so that they are saved. Circle K should consult with the City of Yucca Valley and a "desert native plant specialist" regarding the requirements for relocation or restoration of native California desert vegetation.

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 five 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 on paved surfaces at a minimum of 2 percent.

8.2.6 The result at percolation test B-2/P-1 at 10 feet BSG indicated an unfactored infiltration rate of 9.1 inches per hour. The result at percolation test P-2 at 40 feet BSG indicated an unfactored infiltration rate of 11.2 inches per hour. The result at percolation test B-5/P-3 at 10 feet BSG indicated an unfactored infiltration rate of 5.8 inches per hour. The results indicate that storm water infiltration systems appear feasible for this site. This report recommends that the lower unfactored infiltration rate of 5.8 inches per hour be considered for use in design for infiltration systems 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. 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...."

In the event subsurface storm water collection systems such as bioswales or similar designs that allow concentrated runoff to wet the soils are planned, the proposed locations and details of these features should be provided to Moore Twining for review and comment. If these types of features are required, sufficient setbacks to existing improvements should be maintained such as a minimum 20 foot setback to planned foundations, unless the systems are fully lined with a waterproof liner. In addition, specific measures such as deepened curbs, perimeter cutoffs, liners, etc. should be incorporated in the designs to reduce the potential for excessive movement of other adjacent improvements due to moisture, concentrated wetting, and freewater migration from storm water infiltration systems.

8.3 Site Preparation

8.3.1 Stripping should be conducted in all areas of existing improvements to remove surface vegetation and root systems. The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. The site has several Yucca Trees. It is anticipated that the Yucca Trees may need to be moved out of the project area and replanted so that they are saved. Circle K should consult with the City of Yucca Valley and a "desert native plant specialist" regarding the requirements for relocation or restoration of native California desert vegetation.

8.3.2 As part of the site preparation, existing surface and subsurface improvements in the Circle K parcel such as power poles, utility boxes and underground utility lines (electric line and water line discussed in section 6.1 of this report), etc. should be completely removed (or relocated) and not abandoned in-place. A careful search should be conducted by the contractor for potential buried improvements, underground utilities, and buried obstructions. All existing subsurface improvements and associated backfill soils should be excavated to at least 12 inches below the improvements to be removed, to the depth required to remove all disturbed soils, and all associated backfill, whichever requires the deeper excavation. Upon approval of the excavation by Moore Twining, the base of the excavation should be scarified to a depth of 8 inches, moisture conditioned and compacted as engineered fill prior to backfilling the excavations as engineered fill. Localized areas such as where power poles are removed may be backfilled with controlled low strength material with a minimum of 2 sacks of cement per cubic yard.

8.3.3 Silty sand fill soils with cloth debris were encountered at the site in boring B-1 where the underground fuel storage tanks are planned as described in Section 6.1 of this report. The fill soils are shown on the boring log for boring B-1 as extending to a depth of about 8½ feet BSG. It is our understanding that excavations for the proposed underground fuel storage tanks typically extend to a depth of about 20 feet BSG. Thus, it is anticipated that the fill soils will be removed as part of the excavation for the proposed underground fuel storage tanks. Any cloth or other debris encountered in the fill soils during the excavation for the underground fuel storage tanks should be removed from the site and not mixed with any soils to be used as engineered fill for the site. During the excavation for the underground fuel storage tanks, if it appears that the fill soils with cloth debris or any other debris extend horizontally beyond the limits of the proposed underground fuel storage tanks, then additional removals will be required to remove the fill soils in other areas of planned improvements that may be sensitive to settlement.

8.3.4 After stripping and removal of organics, relocation of any native California desert vegetation, in order to provide a uniform blanket of engineered fill across the building pad and limit the total and differential static settlement of foundations to 1 inch total and ½ inch differential in 40 feet, this report recommends that the entire building pad be over-excavated to at least 4 feet below preconstruction site grades, to the depth required to provide at least 2 feet of engineered fill below bottom of footings, and to at least 12 inches

below any subsurface improvements to be removed (if any), whichever is greater. The horizontal limits of over-excavation should include the footprint of the buildings, all foundations, all concrete walkways adjacent to the building, and a minimum of 5 feet beyond these features, whichever is greater. Upon review of the Contractor's survey data (documenting 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. Due to the anticipated low moisture content of the undisturbed native soil conditions, the contractor should anticipate significant moisture conditioning and mixing of soils will be needed to achieve the moisture content recommendations of this report (optimum to three (3) percent above optimum moisture content).

8.3.5 The horizontal limit of over-excavation for the building pads should be depicted on the project plans.

8.3.6 It is recommended that extra care be taken by the Contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for 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, areas to receive miscellaneous lightly loaded foundations with a line load of less than 1 kip per foot, such as site walls, screen walls and retaining walls, should be over-excavated to the bottom of foundations, to at least 2 feet below preconstruction site grades, to the depth required to remove all undocumented fill soils (if any), and to at least 12 inches below subsurface improvements to be removed (if any), whichever is greater. The over-excavation for retaining walls/screen walls should extend to at least 3 feet beyond the edges of the foundations. The bottom of the over-excavation should be scarified to a depth of at least 8 inches, moisture conditioned and compacted as engineered fill.

8.3.8 Following stripping, 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 at least18 inches below preconstruction site grade, to the depth required to remove all undocumented fill (if any), and to at least 12 inches below subsurface improvements to be removed (if any), whichever is greater. The limits of excavation for pavement areas and exterior slabs should extend at least 3 feet beyond the edge of these improvements. Upon approval of the bottom of the over-excavation by Moore Twining, the bottom of the excavation should be scarified to a minimum depth of 8 inches, the scarified soils should be moisture conditioned and compacted as engineered fill. 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.

It should be noted that up to about 8½ feet of undocumented silty sand fill with cloth debris was encountered in the area of boring B-1 where the underground fuel storage tanks are planned. It is possible that the excavation needed for the proposed underground fuel storage tanks will completely remove the undocumented silty sand fill soils with cloth debris. However, the extent of the undocumented silty sand fill soils with cloth debris is unknown. Thus, the extent of this undocumented fill will need to be determined during the excavation for underground fuel storage tanks so as to ensure the undocumented fill is also removed in other areas of proposed improvements that would be sensitive to settlement.

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

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

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

8.3.12 Final grading shall produce a building pad which is smooth, planar, and resistant to rutting. The finished pad subgrade (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 with trace gravel and occasional poorly graded sands with silt with trace gravel. The on-site soils with an expansion index of less than 20 will be suitable for use as engineered fill below the recommended aggregate base, provided they are free of debris, organics (less than 3 percent by weight and no roots larger than ¼ inch in diameter), irreducible material greater than 3 inches, and the moisture content of the soil meets the recommendations of this report at the time of placement. This report recommends that interior and exterior slabs-on-grade and all Portland cement concrete be underlain by at least 4 inches of Class 2 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 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; however, as indicated in this report, the on-site soils area anticipated to have low moisture contents and require significant moisture conditioning and mixing to ensure a uniform moisture content prior to placement of soils

for compaction as engineered fill. Accordingly, special provisions to moisten the soils should be anticipated for the project. The contractor should account for these conditions in their bid.

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

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	20 - 50
Expansion Index (ASTM D4829)	Less than 20
Organics	Less than 3 percent by weight
R-Value	Minimum 50*
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 5,000 ohm-cm

* for pavement areas only

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

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

8.4.5 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at a frequency listed in Table No. 4.

Table No. 4

Area	Minimum Test Frequency
Building Pad Preparation Limits	1 test per 5,000 square feet per compacted lift, but not less than two tests per building pad per lift
Pavement Subgrade and Mass Grading Outside Building Pad Preparation Limits	1 test per 5,000 square feet per compacted lift
Walkways Outside Building Pad Preparation Limits	1 test per 50 lineal feet
Utility Lines	1 test per 100 feet per lift

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

8.4.7 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 exterior slabs-on-grade and pavement construction outside the building pad 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 Conventional Shallow Spread Foundations for Buildings

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 should be anticipated for design: 1) a total static settlement of 1 inch; 2) a differential static settlement and swell of ½-inch in 40 feet, 3) a total seismic settlement of 1½ inches, and 4) a differential seismic settlement of ¾ inch in 30 feet. In the event conventional shallow spread foundations cannot be designed to tolerate the anticipated seismic settlements, Moore Twining should be contacted to provide alternative recommendations.

8.5.2 Foundations supported on subgrade soils prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.5.3 Perimeter foundations should have a minimum depth of 12 inches below the bottom of the slab, as well as a minimum depth of 12 inches below the exterior finished grades, whichever requires the deeper footing depth. Interior footings for the Circle K store 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 34.133844 degrees and a longitude of -116.387756 degrees and a Site Class D. The data provided in Table No. 5 are based upon the procedures of ASCE 7-16 and were not determined based upon a ground motion hazard analysis. The structural engineer should review the values in Table No. 5 and determine whether a ground motion hazard analysis is required for the project considering the seismic design category, structural details, and requirements of ASCE 7-16 (Section 11.4.8 and other applicable sections). If required, Moore Twining should be notified and requested to conduct the additional analysis, develop updated seismic factors for the project, and update the following values.

Table No. 5

Seismic Factor	ASCE 7-16 Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	1.055g
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration (PGA)	0.959g
Spectral Response At Short Period (0.2 Second), S_s	2.29
Spectral Response At 1-Second Period, S_1	0.817
Site Coefficient (based on Spectral Response At Short Period), F_a	1.0
Site Coefficient (based on spectral response at 1-second period) F_v	See Note
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	2.29
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	See Note
Five percent damped design spectral response accelerations for short period, S_{DS}	1.527
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 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 The moisture conditions of the subgrade soils for the building pad and foundation excavations should be maintained in accordance with the recommendations for engineered fill until placement of concrete for foundations or until aggregate base is placed for the building pad areas.

8.5.8 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 on 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 with a minimum width of 15 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.

8.5.9 Sight lighting and pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot may be used to resist axial loads. Lateral load resistance may be estimated using the 2022 CBC non-constrained procedure. The allowable passive resistance of the native soils may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot to a maximum of 3,500 pounds per square foot. The passive pressure may be applied over twice the pier diameter. The upper 12 inches of subgrade soils in landscaped areas should be neglected in determining the total passive resistance.

8.5.10 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.

8.5.11 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 12 inches of subgrade in landscaped areas should be neglected in determining the total passive resistance.

8.6 Fuel Canopy Foundations Supported on Cast-in-Drilled-Hole (CIDH) Pile Foundations

The following recommendations were prepared for drilled pier type foundations for support of the fuel canopy foundations. Due to the shallow groundwater conditions, if spread footings are desired for use instead of drilled pier foundations, the spread foundations may be designed and the subgrade soils prepared in accordance with the recommendations in this report for the proposed buildings.

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

8.6.2 Skin friction in the upper portion of the piles, to a depth of 12 inches should be neglected for design. The allowable vertical downward load capacity of the CIDH pile foundations below a depth of 12 inches below site grade may be designed based on an allowable skin friction value of 200 pounds per square foot based on a minimum embedment depth of 11.5 feet BSG. The above stated values assume that the cast-in-drilled-hole foundations are placed into the existing undisturbed native soils. These values may be increased one-third (⅓) for short duration loading.

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

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

8.6.5 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on a total static settlement of 1 inch and a differential static settlement of ½ inch between foundations.

8.6.6 Passive resistance in the upper portion of the piles, to a depth of 1 foot should be neglected for design. The allowable passive resistance of the soils below a depth of 1 foot below site grade may be assumed to be equal to the pressure developed by a fluid with a density of 350 pounds per cubic foot to a maximum of 3,500 pounds per square foot. These values may be increased by one-third (1/3) for short duration wind or seismic loads. The passive pressure for drilled pile foundations spaced at three (3) pile diameters may be applied over a width equal to 2 pile diameters.

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

8.7 Cast-In-Drilled-Hole Pile Construction

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

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

8.7.3 The casing should be slowly removed from the shaft excavation during placement of concrete to ensure the casing is not raised above the level of the concrete during shaft construction, to prevent sidewall soils from sloughing into the shaft excavation. As an alternative, it may be possible to utilize a drilling slurry for temporary support of the foundation excavations if unstable sidewalls occur. The Contractor will be required to provide temporary excavation support of the drilled pile excavations as necessary to construct the foundations.

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

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

8.7.6 The concrete should be placed from the bottom of the excavation by extending the tremie pipe or pump pipe to the bottom of the excavation and maintaining the outlet of the pipe below the wet concrete to prevent entrapment of freewater or slurry in the concrete. The concrete should be placed in a continuous manner to provide a seamless deep foundation element.

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

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

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

8.8 Interior Slabs-on-Grade

8.8.1 Interior slabs-on-grade and all concrete walkways attached to the building should be constructed 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.

8.8.2 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, concrete mixers, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

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

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

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

8.8.5 The 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 and sealed per manufacturer's recommendations.

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

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

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

8.8.9 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.

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

8.9 Exterior Slabs-On-Grade

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

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

8.9.2 Exterior concrete slabs-on-grade (such as sidewalks, etc.) located directly adjacent to the building (i.e., slabs within the building pad preparation limits) and exterior slabs-on-grade located outside the building pad preparation limits (not directly attached to the building) should be supported on a minimum of 4 inches of aggregate base over engineered fill prepared as recommended in the Site Preparation section of this report.

8.9.3 The moisture content of the native silty sand subgrade soils should be verified to be at least optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.9.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges or liners which extend to the bottom of the aggregate base. This should reduce the potential for infiltration of water (and associated expansive soil movement) into the aggregate base below exterior slabs.

8.9.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. The on-site silty sand soils should be maintained to at least optimum moisture content. 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.

8.10 Asphalt Concrete (AC) Pavements

8.10.1 The subgrade soils for asphalt 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.

8.10.2 The following pavement sections are based on an R-value of 50, traffic index values ranging from 5.0 to 10.0, and a minimum of 3 inches of asphalt concrete and a minimum of 4 inches of aggregate base. 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	4.0	12
5.5	3.0	4.0	12
6.0	3.5	4.0	12
6.5	3.5	4.5	12
7.0	4.0	4.5	12
7.5	4.5	7.5	12
8.0	4.5	6.0	12
8.5	5.0	6.0	12
9.0	5.5	6.5	12
9.5	6.0	6.5	12
10.0	6.0	8.0	12

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

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

8.10.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions. The Contractor should notify our firm in writing if the areas to receive pavements receive imported fill so that our firm can evaluate this condition.

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

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

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

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

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

8.10.10 The asphalt concrete should comply with the requirements for a Type A asphalt concrete 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.

8.11 Portland Cement Concrete (PCC) Pavements

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

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

8.11.2 The following preliminary Portland cement concrete pavement sections have been prepared for average daily truck traffic ranging from about 2 to 145 trucks per day which corresponds to Traffic Indices ranging from 6 to 10. The design pavement sections should be selected by the civil engineer based on the anticipated traffic loading. If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) a design life of 20 years, 3) load transfer by aggregate interlock or dowels, 4) concrete shoulder, 5) a load safety factor of 1.1, 6) truck loading consisting of 1 single axle load of 18 kips, and 5) a maximum single axle weight of 12,000 pounds, and two tandem axle loads of 32,000 pounds.

Table No. 7
Portland Cement Concrete Pavements

Traffic Index	ADTT	PCC Thickness (inches)	Aggregate Base (inches)	Compacted Subgrade (inches)
6.0	2.0	5.5	4	12
7.0	7.3	6.0	4	12
8.0	22.2	6.0	4	12
9.0	59.8	6.5	4	12
10.0	144.8	6.5	4	12

ADTT - Average Daily Truck Traffic
PCC - Portland Cement Concrete (minimum Modulus of Rupture=500 psi)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)
Note - Aggregate base may be substituted for imported, non-expansive fill if desired

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

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

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

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

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

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

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

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

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

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

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

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

8.12 Underground Fuel Storage Tanks

Silty sand fill soils with cloth debris were encountered at the site in boring B-1 where the underground fuel storage tanks are planned as described in Section 6.1 of this report. The fill soils are shown on the boring log for boring B-1 as extending to a depth of about 8½ feet BSG. It is our understanding that excavations for the proposed underground fuel storage tanks typically extend to a depth of about 20 feet BSG. Thus, it is anticipated that the fill soils will be removed as part of the excavation for the proposed underground fuel storage tanks. Any cloth or other debris encountered in the fill soils during the excavation for the underground fuel storage tanks should be removed from the site and not mixed with any soils to be used as engineered fill for the site. During the excavation for the underground fuel storage tanks, if it appears that the fill soils with cloth debris or any other debris extend horizontally beyond the limits of the proposed underground fuel storage tanks, then additional removals will be required to remove the fill soils in other areas of planned improvements that may be sensitive to settlement.

8.12.1 It is our understanding underground fuel storage tanks extending to a depth of about 20 feet BSG are planned as part of fuel delivery. The excavation for the tanks may be in close proximity to the foundations for the fuel canopies and other improvements. The contractor needs to consider these conditions in planning the excavation and the footings for the canopies. Since the underground fuel storage tanks are to be backfilled with pea gravel that is encapsulated with a filter fabric, the contractor will need to shore, or selectively backfill the excavation such that adjacent improvements are protected and a minimum 8-foot horizontal separation is provided between the foundations for the fuel canopies and the pea gravel backfill for the underground fuel storage tank. Shoring of excavations should also be employed based on the recommendations for temporary excavations included in Section 8.13 of this report.

8.12.2 The excavation limits, type of backfill, and the compaction requirements for the storage tanks should be specified by the applicable design professional and should be in compliance with the manufacturer's requirements and the requirements of the governing agency, whichever is more stringent.

8.12.3 If open graded gravel or rock material such as pea gravel or crushed rock is required by a regulatory agency or designer for use as backfill, the material should be placed in thin lifts and compacted using a vibratory compactor or other appropriate methods to a non-yielding condition as determined by Moore Twining. The backfill should be conducted in accordance with the tank manufacturer's requirements, and this report (if applicable per the manufacturer), whichever is most stringent. Each lift must be approved by Moore Twining prior to placing the next lift. All open graded materials should be fully encased in a geotextile filter fabric to reduce the potential for fines to migrate into the rock section. It should be noted that the use of open graded rock as backfill increases the potential for settlement, migration of water, etc.

8.12.4 Open graded gravel (such as pea gravel) should be placed in thin (less than 8 inch) lifts and densified per section 8.4.6 of this report if vibratory compaction of the pea gravel is allowed by the manufacturer.

8.13 Slopes, Shoring and Temporary Excavations

8.13.1 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.

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

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

8.13.4 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 engineered fill or a controlled density fill material to backfill the voids created by removal of the shoring. All voids resulting from removal of shoring shall be backfilled and all soils disturbed from the shoring removal shall be over-excavated and compacted as engineered fill.

8.13.5 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.

8.14 Utility Trenches

8.14.1 Excavation of utility trenches in areas where select fill materials such as aggregate base has been placed will require selective excavation to enable the select fill materials to be used as trench backfill. Where select materials are mixed with native on-site soils, the excavation spoils will not be suitable for use as backfill within the upper portion of the trenches where select fill is specified below concrete slabs on grade. The trench backfill should be conducted in a manner to re-establish the specified select fill materials.

8.14.2 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable, the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 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.

8.14.3 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The Contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The Contractor is responsible for notifying the Owner and Moore Twining if the requirements of the agency and this report conflict, the most stringent applies. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent, assuming a hydraulic gradient exists (gravel, rock, crushed gravel, etc. cannot be used as backfill on the project). The width of the trench should provide a minimum clearance of 8 inches between the sidewalls of the pipe and the trench, or as necessary to provide a trench width that is 12 inches greater than 1.25 times the outside diameter of the pipe, whichever is greater. As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 92 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported, non-expansive materials moisture conditioned and compacted per Sections 8.4.4 of this report. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

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

percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321 listed in table below (minimum manufacturer's requirements), or as necessary to provide sufficient space to achieve the required compaction, whichever is greater. As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to at least 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

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

8.14.6 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be placed in 8 inch lifts, moisture conditioned and compacted per the recommendations of Sections 8.4.4 of this report. 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.

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

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

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

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

8.14.11 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.

8.15 Corrosion Protection

8.15.1 Based on the corrosion severity ratings listed in the Table No. 2 of section 6.8 of this report, the analytical results of sample analyses indicate the samples tested had a resistivity value of 9,200 ohms-centimeter, with corresponding pH value of 7.8. Based on the resistivity value, buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "moderately 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, were 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.

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

8.15.3 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 provide design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

9.0 DESIGN CONSULTATION

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

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

9.3 If Moore Twining is not retained for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

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

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

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

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

10.5 If Moore Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance satisfactory to the owner 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 Circle K. 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
Certified Engineering Geologist
Geotechnical Engineering Division

Harry D. Moore, RGE
President

REFERENCES

California Division of Mines and Geology, 1993a, State of California Special Studies Zones, Yucca Valley North Quadrangle. Scale 1:24,000

Dibblee, T.W., 1967, Geologic Map of the Joshua Tree Quadrangle, San Bernardino and Riverside Counties, California: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-516, Scale 1:62,500.

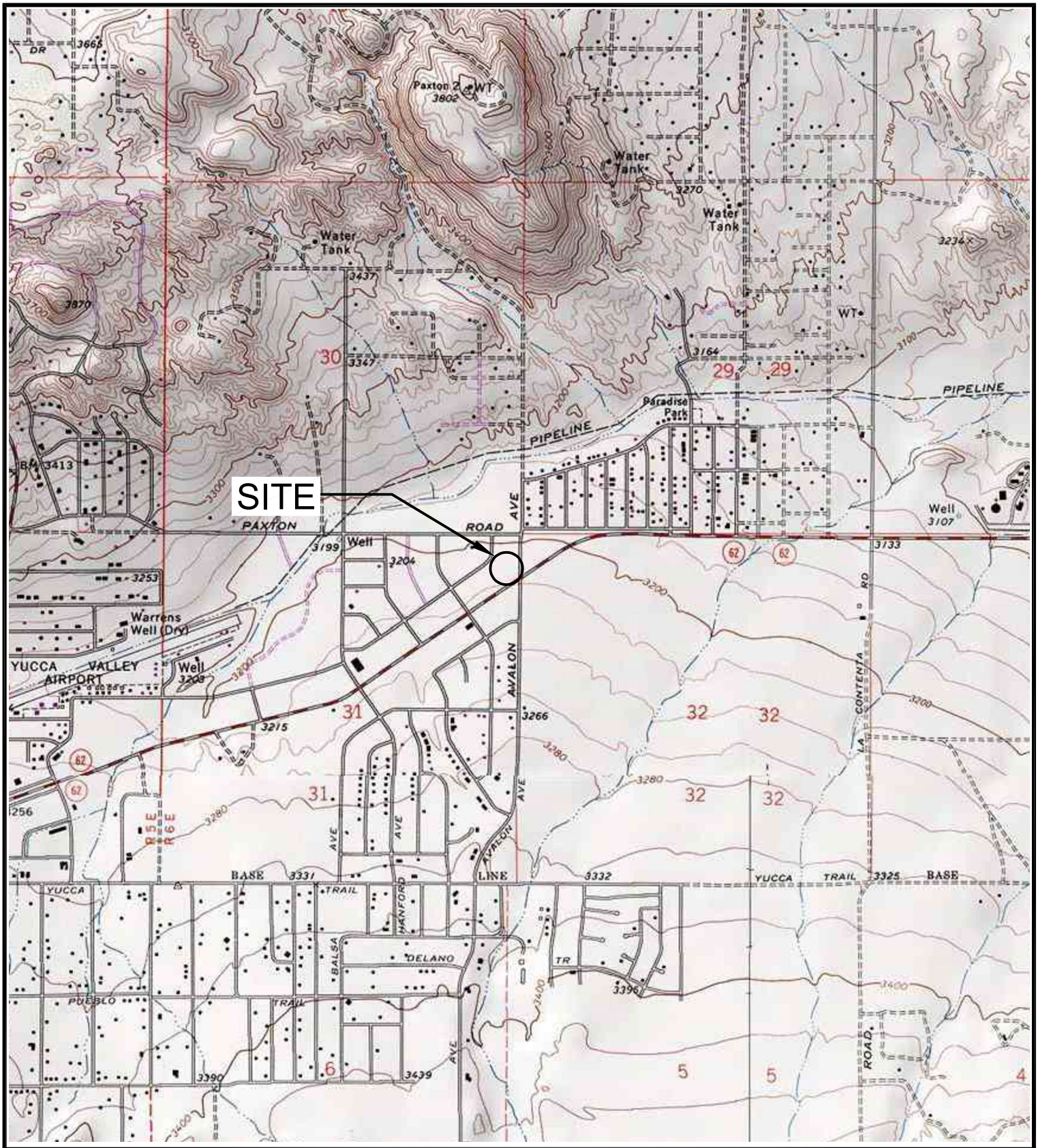
Sladden Engineering, "Fault Investigation, Proposed Retail Center, NWC of Avalon Avenue and Palisade Drive, APN 601-201-41, Yucca Valley, California," dated January 20, 2017

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map

Drawing No. 2 - Test Boring and Percolation Test Location Map for Proposed Circle K Store



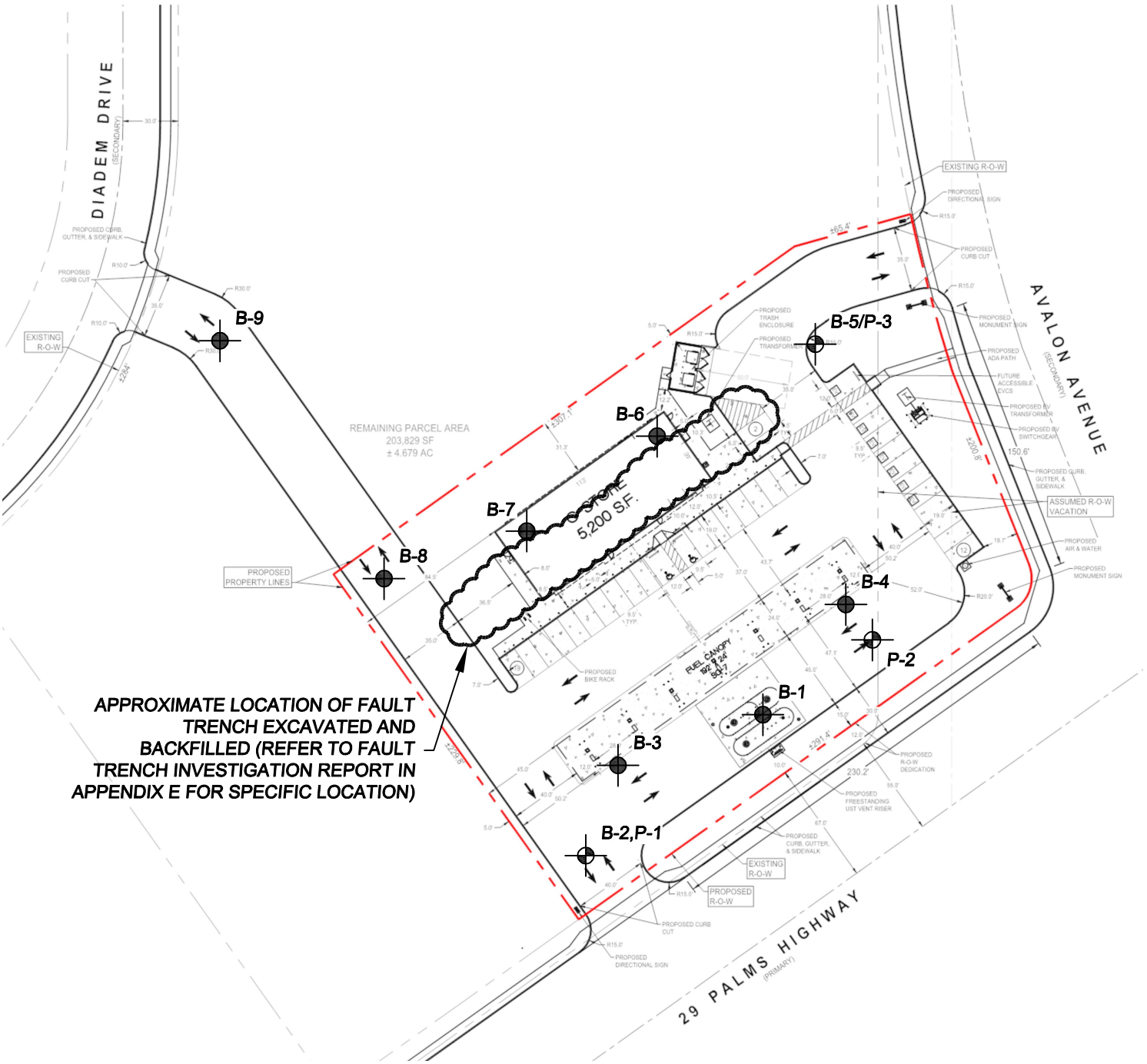
SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
YUCCA VALLEY NORTH, CALIFORNIA QUADRANGLE 1994

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP
PROPOSED CIRCLE K STORE
NORTHWEST CORNER OF TWENTYNINE PALMS HIGHWAY
AND AVALON AVENUE
YUCCA VALLEY, CALIFORNIA

FILE NO.: 28851-02-01	DATE: 08/27/2024
DRAWN BY: RM	APPROVED BY:
PROJECT NO. G28851.02	DRAWING NO. 1

MOORE TWINING ASSOCIATES, INC.



APPROXIMATE TEST BORING LOCATION

APPROXIMATE PERCOLATION TEST LOCATION

N

0 60

APPROXIMATE SCALE IN FEET

TEST BORING LOCATION MAP PROPOSED CIRCLE K STORE NORTHWEST CORNER OF TWENTYNINE PALMS HIGHWAY AND AVALON AVENUE YUCCA VALLEY, CALIFORNIA	FILE NO. 28851-02-02	DATE DRAWN: 08/23/2024
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. G28851.02	DRAWING NO. 2

MOORE TWINING ASSOCIATES, INC.

APPENDIX B

LOGS OF BORINGS

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

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

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

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

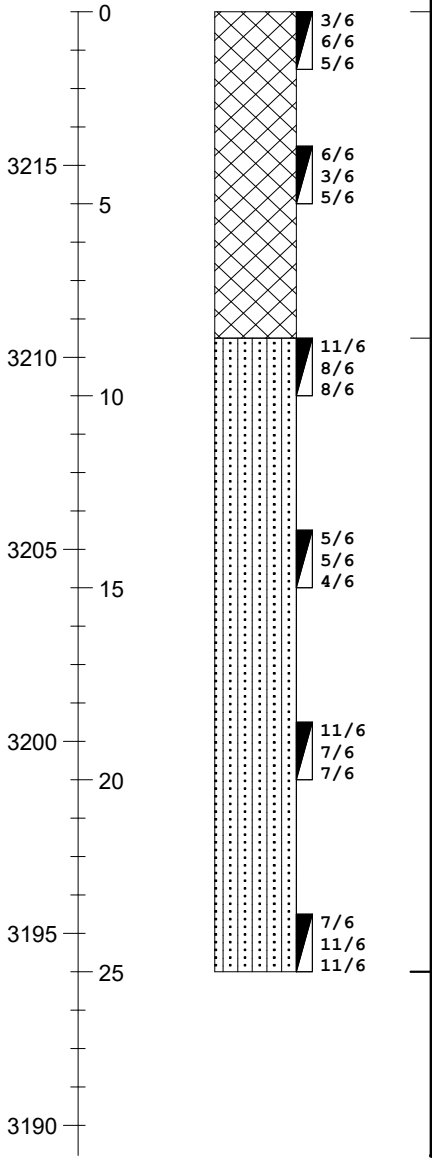
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 16, 2024

Elevation: 3,219 feet

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 %
		FILL	FILL - SILTY SAND; medium dense, damp, fine to coarse grained, light brown, with a little fine gravel, with cloth fabric		11	
			Loose, moist, trace fine gravel, with cloth fabric		8	
		SM	NATIVE - SILTY SAND; medium dense, damp, fine to coarse grained, light brown, with a little fine to coarse gravel		16	1.0
			Loose, moist, fine grained, brown, increase in fines content, trace fine gravel	From 13.5-15': Gravel = 0.7% Sand = 82.3% -200 = 17.0%	9	
			Medium dense, fine to medium grained, slight increase in fine gravel content		14	1.6
			Damp, fine to coarse grained, increase in fine gravel content, light brown		22	
			Bottom of Boring B-1 at 25 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2/P-1

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

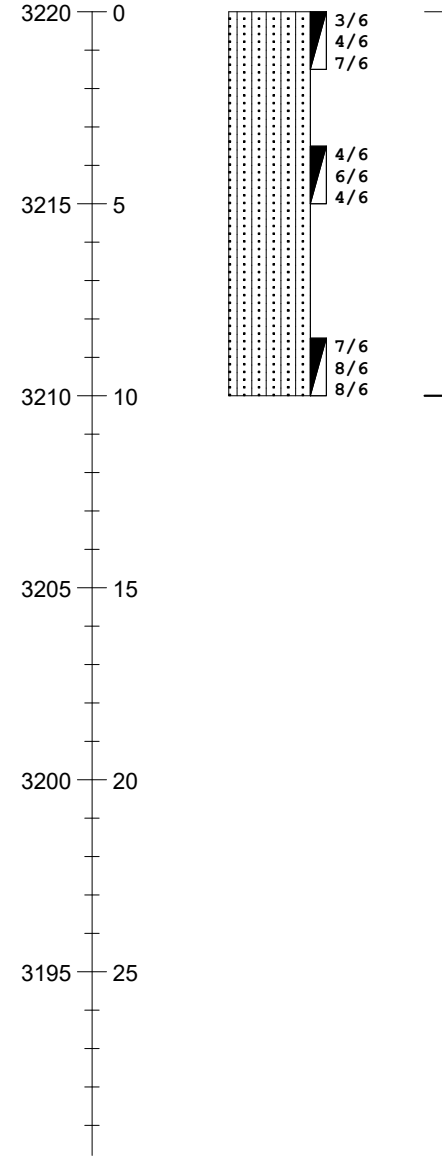
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,220 feet

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 %
		SM	SILTY SAND; medium dense, dry, fine to coarse grained, light brown, with some fine gravel	From 0-3.5': R-value = 74	11	0.2
			Loose, moist, brown, increase in fine grained sand content, decrease in gravel content		10	
			Medium dense, trace fine gravel	From 8.5-10': Gravel = 1.8% Sand = 83.2% -200 = 15.0%	16	
			Bottom of Percolation Test Boring B-2/P-1 at 10 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

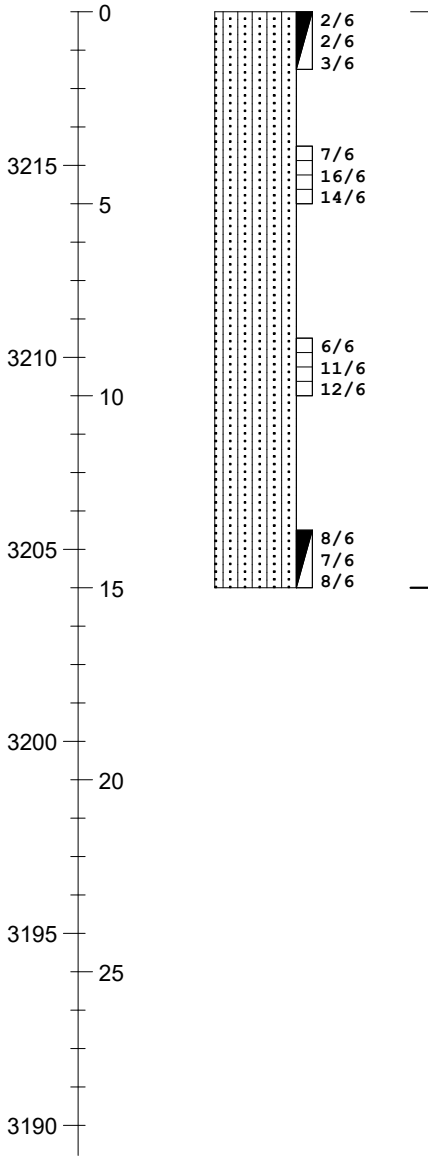
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 16, 2024

Elevation: 3,219 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		SM	SILTY SAND; loose, dry, predominantly fine to medium grained, with some coarse sand, light brown, trace fine gravel		5	0.4
3215 / 5			Medium dense, increase in medium to coarse sand content and fine gravel content	DD = 120.6 pcf	30	0.7
3210 / 10			Damp, increase in find sand content, trace fine gravel, with a rootlet	DD = 115.3 pcf	23	1.1
3205 / 15			No rootlets		15	
			Bottom of Boring B-3 at 15 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

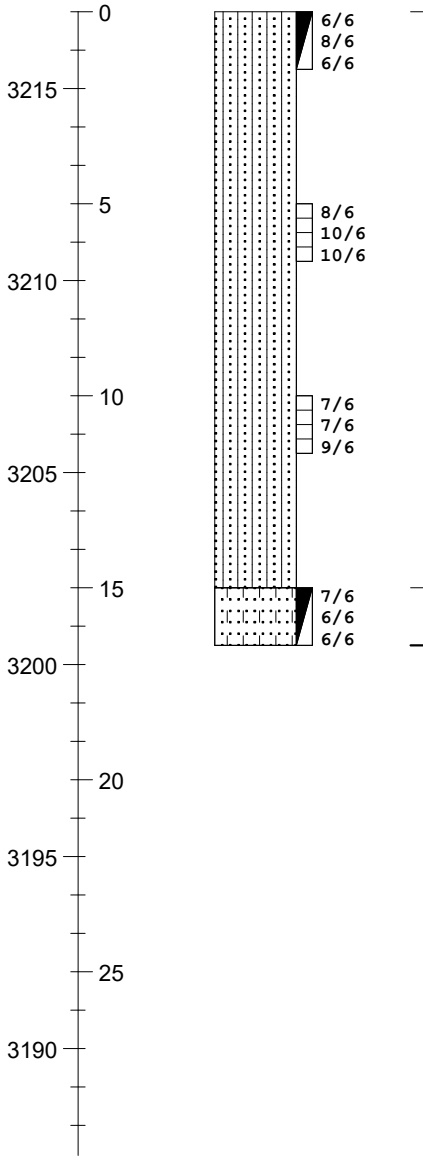
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,217 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		SM	SILTY SAND; medium dense, dry, predominantly fine to medium grained, some coarse sand, brown, with a little fine gravel		14	0.9
5			Damp	From 5-6.5': DD = 116.0 pcf ø = 38° c = 370 psf	20	1.1
10			Loose, increase in fine sand content, with a 2-inch by 3.5-inch rock	DD = 109.8 pcf	16	2.1
15		SW-SM	WELL GRADED WITH SILT; medium dense, moist, fine to coarse grained, brown, with some fine gravel	From 15-16.5': Gravel = 11.6% Sand = 79.6% -200 = 8.8%	12	
			Bottom of Boring B-4 at 16.5 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5/P-3

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

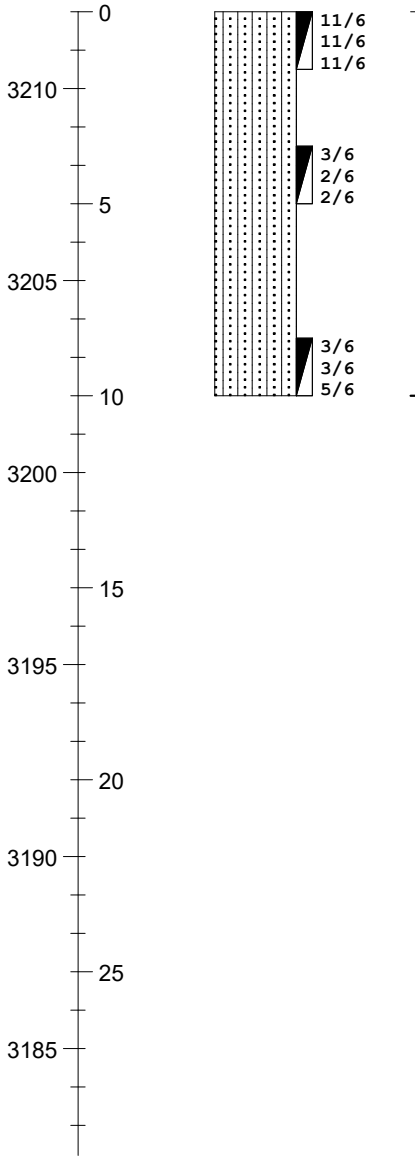
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,212-1/2 feet

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	11/6, 11/6, 11/6	SM	SILTY SAND; medium dense, damp, predominantly fine to medium grained, some coarse sand, light brown, with some fine gravel, trace coarse gravel	From 0-3.5': R-value = 72	22	1.2
5	3/6, 2/6, 2/6		Loose, moist, increase in fine grained sand content, brown, decrease in fine gravel content, no coarse gravel		4	
10	3/6, 3/6, 5/6		Predominantly fine to medium grained, trace coarse sand and fine gravel	From 8.5-10': Gravel = 1.5% Sand = 85.7% -200 = 12.8%	8	
			Bottom of Percolation Test Boring B-5/P-3 at 10 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

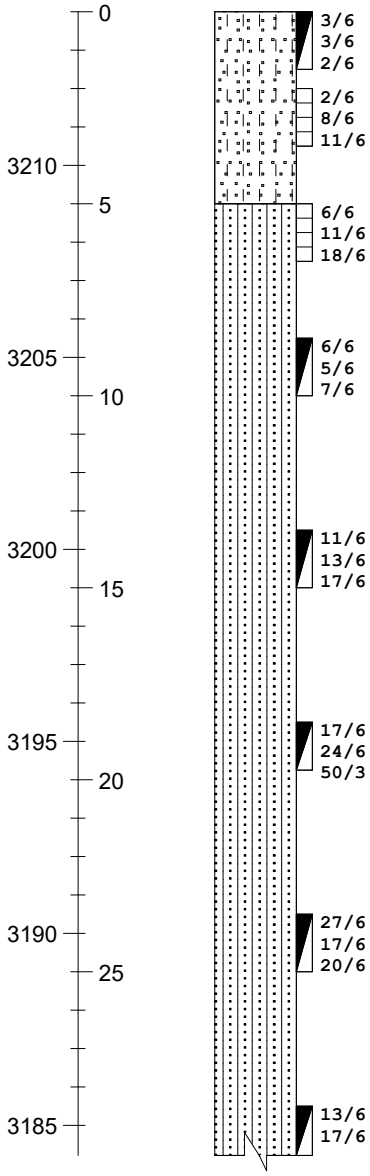
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,214-1/2 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
	SP-SM	POORLY GRADED SAND WITH SILT; loose, dry, predominantly fine to medium grained, some coarse sand, light brown, trace fine gravel At 2 feet - Medium dense, with some fine to coarse gravel	From 0-3.5': Gravel = 3.8% Sand = 84.3% -200 = 11.9% LL = Non-viscous PI = Non-plastic EI = 0 pH = 8.1 SR = 9,200 ohm-cm Cl = 0.0018% SS = 0.0033%	5 19	0.5 0.9
	SM	SILTY SAND; medium dense, damp, fine to medium grained, brown, with trace fine gravel		29	1.4
			From 2-3.5': DD = 113.0 pcf Coarse gravel in shoe of sampler	12	
			From 5-6.5': DD = 116.2 pcf		
		Decrease in moisture content, sharp increase in fine gravel content and medium to coarse sand content	From 8.5-10': Gravel = 0.9% Sand = 83.2% -200 = 15.9%	30	0.9
		SILTY SAND WITH GRAVEL; very dense, moist, fine to coarse grained, brown, gravel is fine to coarse gravel	Coarse gravel in shoe of sampler	>74	5.4
		Dense	No recovery	37	
		SILTY SAND; dense, fine to coarse		32	4.3

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

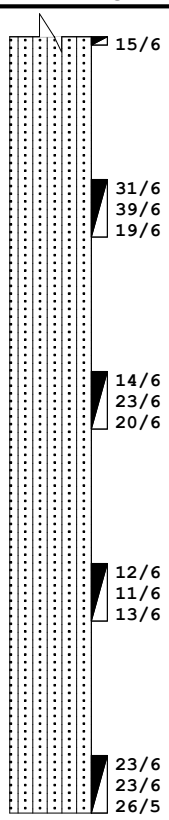
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,214-1/2 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30		grained, light brown, with some fine gravel, trace coarse gravel			
3180 35		Very dense, dry, increase in fine sand content, with some fine to coarse gravel		58	0.8
3175 40		Dense, increase in fines content		43	0.7
3170 45		Medium dense, damp, predominantly fine to medium grained, brown, sharp decrease in coarse sand and fine gravel content, trace fine gravel, no coarse gravel	From 43.5-45': Gravel = 1.0% Sand = 85.3% -200 = 13.7%	24	1.1
3165 50		Dense, increase in fine gravel content		48	1.0
		Bottom of Boring B-6 at 50 feet			
3160 55					
3155					

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-7

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

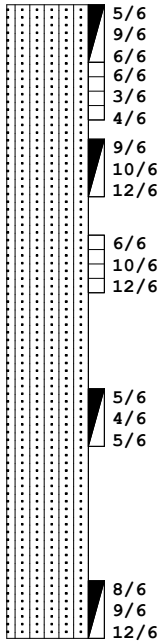
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,215 feet

Depth to Groundwater First Encountered During Drilling: N/E



ELEVATION/ DEPTH (feet)	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
3215 — 0	SM	SILTY SAND; medium dense, damp, fine to coarse grained, brown, with some fine to coarse gravel	Low recovery, gravel in shoe of sampler	15	2.6
		At 1.5 feet - Loose, dry, decrease in gravel content and coarse sand content, light brown, trace fine gravel		7	0.9
		At 3.5 feet - Medium dense, damp, increase in medium to coarse sand content and fine gravel content	From 1.5-3': DD = 115.7 pcf Gravel = 3.1% Sand = 84.1% -200 = 12.8% ø = 42° c = 240 psf	22	1.1
3210 — 5		At 6 feet - Damp		22	1.3
			From 6-7.6': DD = 115.1 pcf		
3205 — 10		Loose, brown, increase in fines content and fine sand content, decrease in medium sand content and fine gravel content, trace fine gravel	From 10-11.5': Gravel = 0.9% Sand = 82.5% -200 = 16.6%	9	1.6
3200 — 15		Medium dense, damp, light brown, increase in medium to coarse sand content		21	
		Bottom of Boring B-7 at 16.5 feet			
3195 — 20					
3190 — 25					

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-8

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

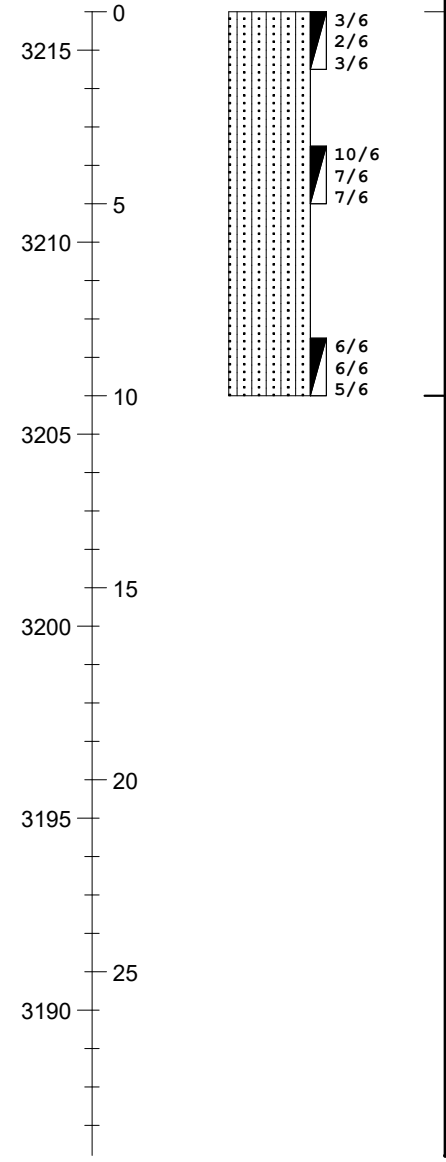
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 16, 2024

Elevation: 3,216-1/2 feet

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 %
		SM	SILTY SAND; loose, dry, predominantly fine to medium grained, some coarse sand, light brown, trace fine gravel	From 0-3.5': Gravel = 1.3% Sand = 83.0 -200 = 15.7%	5	0.6
			Medium dense, damp, increase in fines content, with a little fine gravel, trace coarse gravel		14	
			Brown, trace fine gravel with no coarse gravel		11	
			Bottom of Boring B-8 at 10 feet			

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: B-9

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,213-1/2 feet

Depth to Groundwater First Encountered During Drilling: N/E

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	4/6 4/6 4/6	SM	SILTY SAND; loose, dry, predominantly fine to medium grained, some coarse sand, light brown, with a little fine to coarse gravel	Very low recovery	8	0.4
3210	9/6 11/6 9/6		Medium dense, with some coarse gravel	Very low recovery, gravel in shoe of sampler	20	
5						
3205	9/6 6/6 5/6		Decrease in medium to coarse sand content, trace fine gravel		11	
10			Bottom of Boring B-9 at 10 feet			
3200						
15						
3195						
20						
3190						
25						
3185						

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-2

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

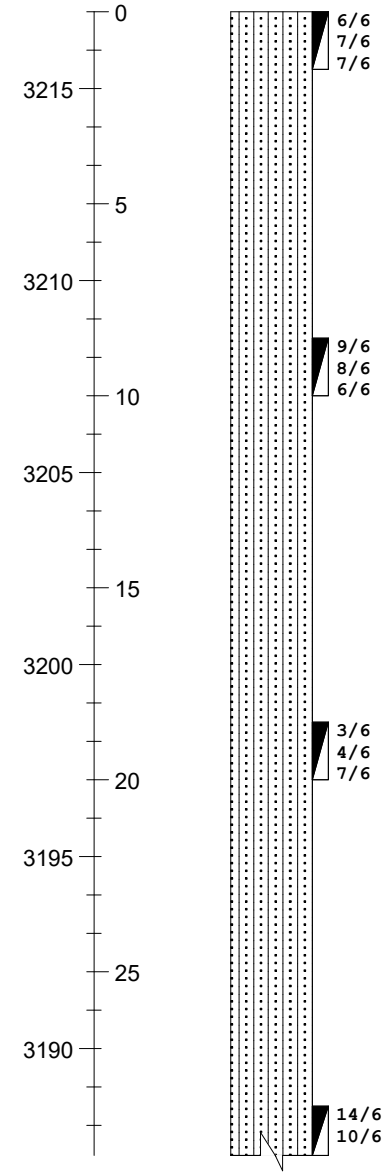
Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,217-1/2 feet

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 %
		SM	SILTY SAND; medium dense, damp, predominantly fine to medium grained, some coarse sand, light brown, with a little fine gravel, trace coarse gravel		14	1.1
			Brown, with a coarse gravel, trace fine gravel	Very low recovery, gravel in shoe of sampler	14	
			Moist, increase in fine sand content and fines content		11	
			Damp, increase in medium to coarse		20	

Notes:

Figure Number

MOORE TWINING ASSOCIATES, INC.

Test Boring: P-2

Project: Circle K Yucca Valley

Project Number: G28851.02

Drilled By: J.S.

Drill Type: CME 75

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

Hammer Type: 140 Pound Auto Trip

Logged By: G.M.

Date: August 17, 2024

Elevation: 3,217-1/2 feet

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		sand content, with a little fine gravel			
3185						
35						
3180						
	13/6 13/6 22/6	SP-SM	POORLY GRADED SAND WITH SILT; dense, damp, predominantly fine to medium grained, some coarse sand, brown, with some fine gravel	From 38.5-40': Gravel = 11.9% Sand = 78.0% -200 = 10.1%	35	
40			Bottom of Percolation Test Boring P-2 at 40 feet			
3175						
45						
3170						
50						
3165						
55						
3160						

Notes:

Figure Number

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Fill
	SM: Silty sand
	SW-SM: Well graded sand with silt
	SP-SM: Poorly 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 8/16/24 and 8/17/24 using a CME 75 drill rig equpped with 6-5/8" and 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. Elevations of the borings were interpolated to the nearest 1/2 foot based on our review of the topographic elevation contours shown on the ALTA/NSPS Land Title and Topographic Survey, dated April 9, 2024, prepared by Duryea & Associates Survey and Mapping.

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
Expansion Index (ASTM D4829)	Swell potential of soil with increases in moisture content.
Consolidation (ASTM 2435)	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
Moisture-Density Relationship (D1557)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.
R-Value (ASTM 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	0.7	3.9	33.0	45.4	17.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.3		
#8	96.7		
#16	88.1		
#30	71.7		
#50	52.5		
#100	32.3		
#200	17.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.02 D_{60}= 0.390 D_{50}= 0.276
D_{30}= 0.137 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-1 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 13.5-15'

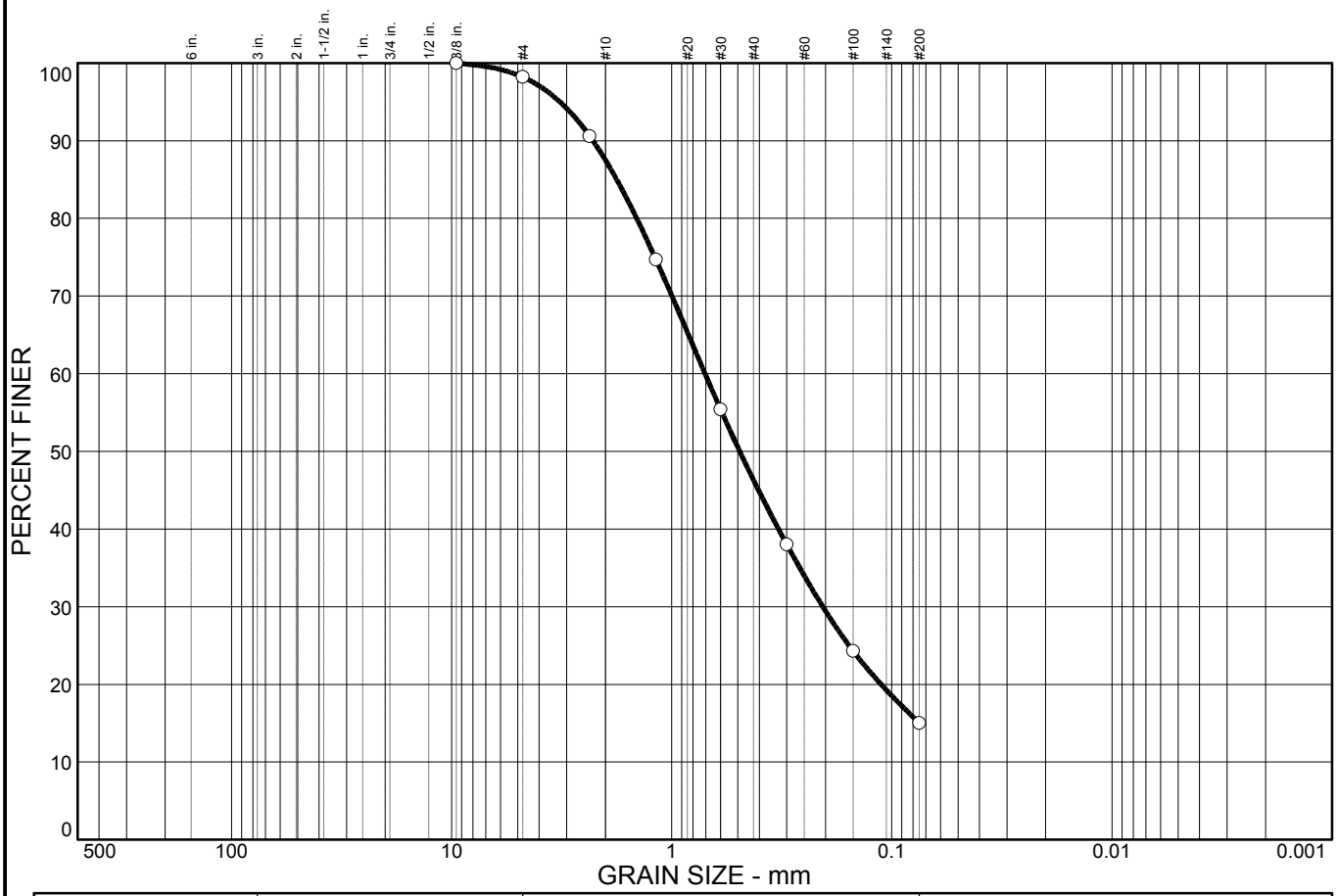
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.8	10.7	41.2	31.3	15.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.2		
#8	90.6		
#16	74.7		
#30	55.4		
#50	38.0		
#100	24.3		
#200	15.0		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.78 D_{60}= 0.706 D_{50}= 0.491
D_{30}= 0.206 D_{15}= 0.0750 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-2/P-1 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 8.5-10'

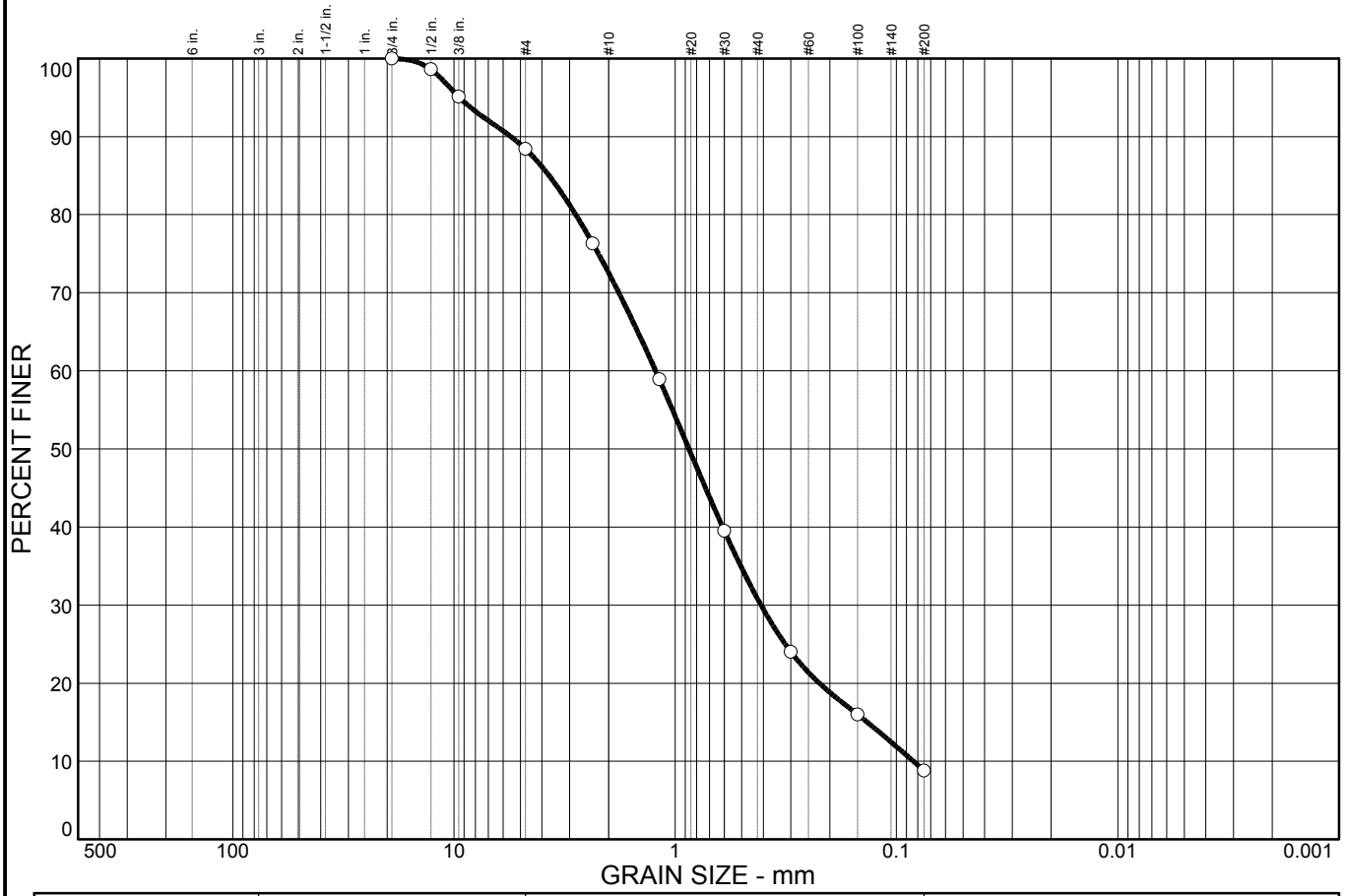
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 2

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	11.6	15.9	41.7	22.0	8.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	98.6		
3/8 in.	95.1		
#4	88.4		
#8	76.3		
#16	58.9		
#30	39.5		
#50	24.0		
#100	16.0		
#200	8.8		

* (no specification provided)

Material Description

Well-graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.72 D_{60}= 1.23 D_{50}= 0.867
D_{30}= 0.410 D_{15}= 0.136 D_{10}= 0.0839
C_u= 14.63 C_c= 1.63

Classification

USCS= SW-SM AASHTO=

Remarks

Sample No.: B-4 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 15-16.5'

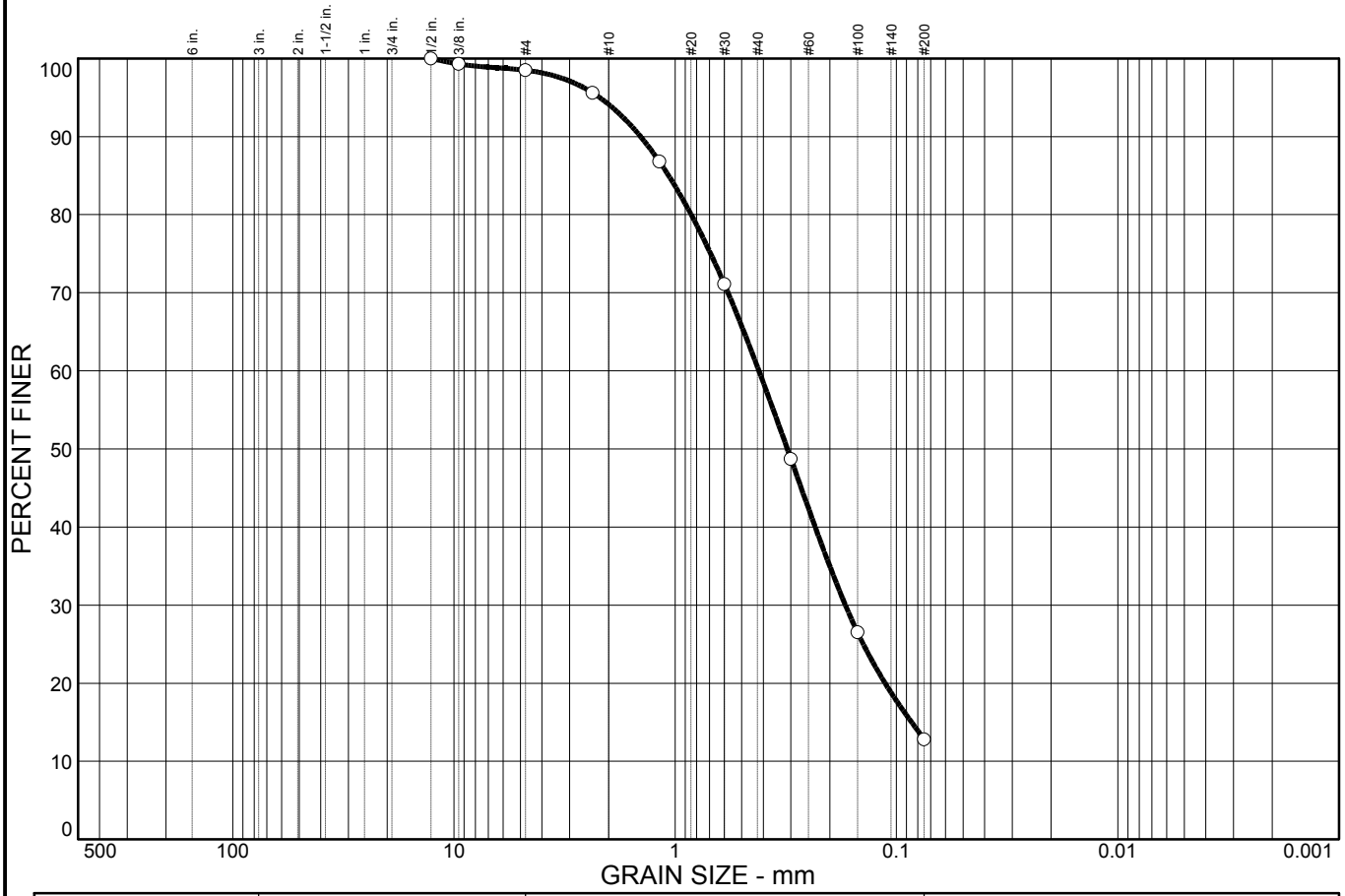
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 3

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.5	4.4	33.6	47.7	12.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.3		
#4	98.5		
#8	95.6		
#16	86.8		
#30	71.1		
#50	48.7		
#100	26.5		
#200	12.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.07 D_{60}= 0.419 D_{50}= 0.311
D_{30}= 0.170 D_{15}= 0.0855 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-5/P-3 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 8.5-10'

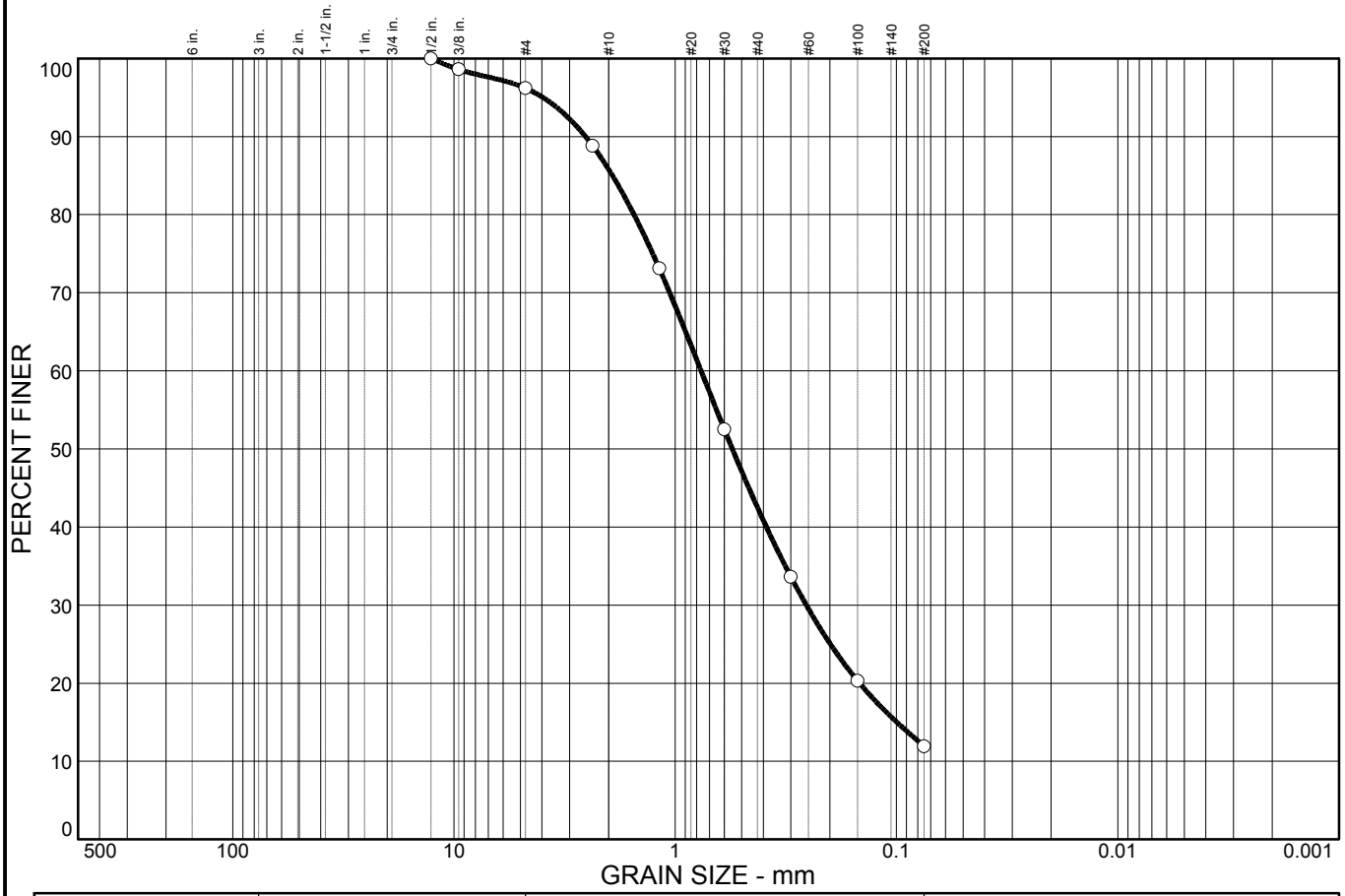
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 4

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	3.8	10.4	43.3	30.6	11.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	98.6		
#4	96.2		
#8	88.8		
#16	73.1		
#30	52.5		
#50	33.6		
#100	20.3		
#200	11.9		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 1.92 D_{60}= 0.764 D_{50}= 0.552
D_{30}= 0.256 D_{15}= 0.0995 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 0-3.5'

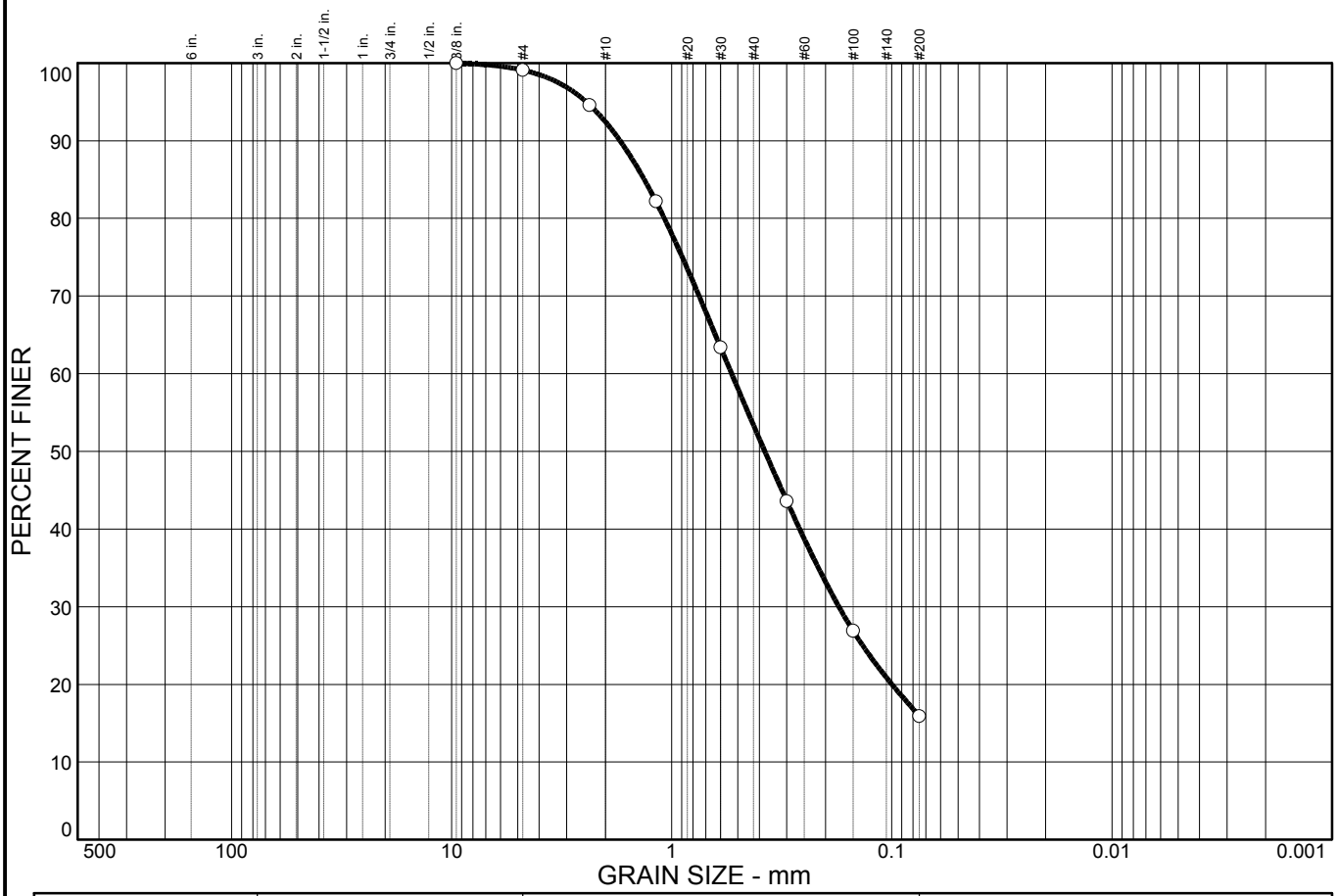
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 5

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.9	6.7	39.1	37.4	15.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.1		
#8	94.6		
#16	82.2		
#30	63.4		
#50	43.6		
#100	26.9		
#200	15.9		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.33 D_{60}= 0.534 D_{50}= 0.378
D_{30}= 0.174 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 8.5-10'

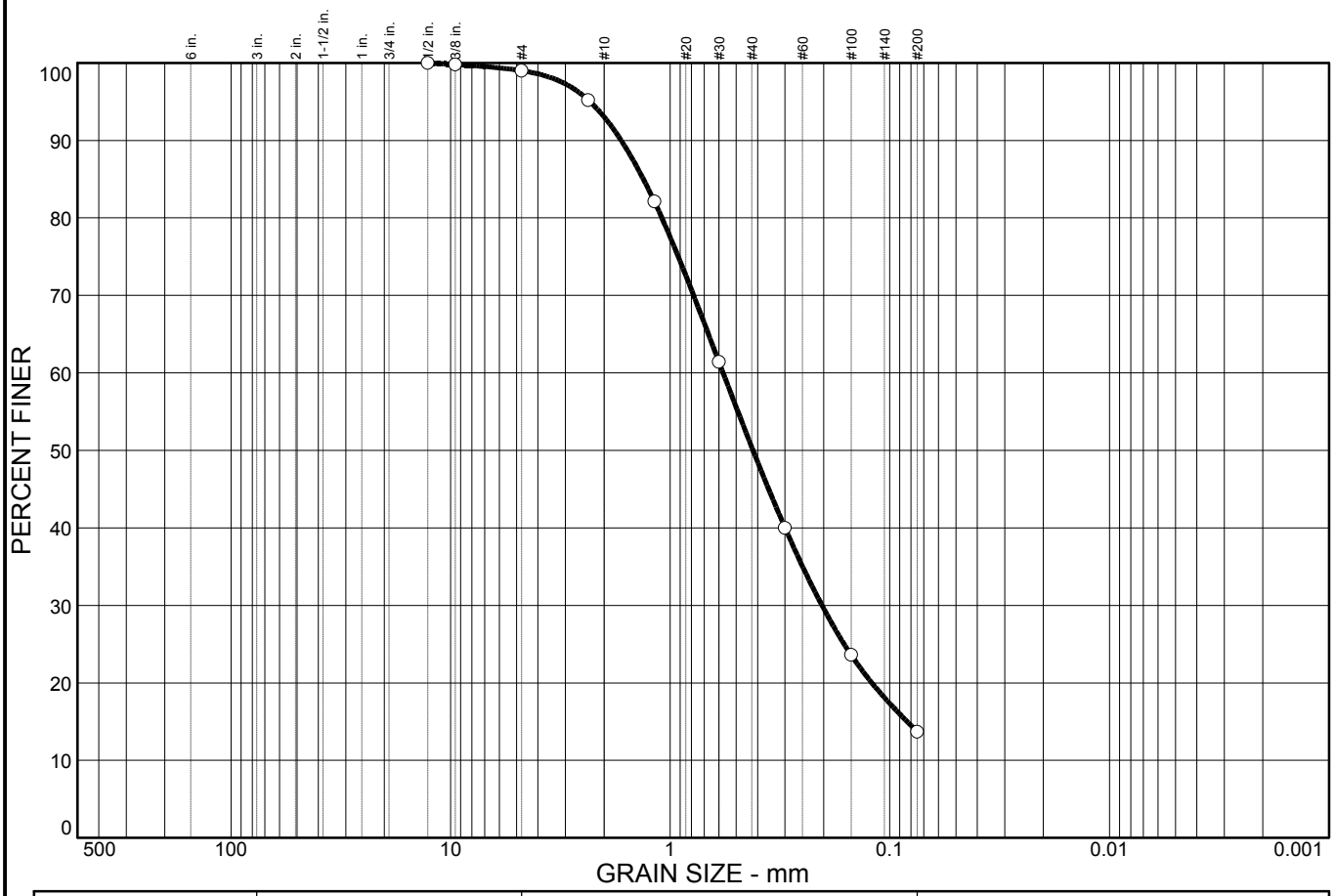
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 6

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.0	6.0	42.6	36.7	13.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.8		
#4	99.0		
#8	95.2		
#16	82.1		
#30	61.4		
#50	40.0		
#100	23.6		
#200	13.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.32 D_{60}= 0.575 D_{50}= 0.420
D_{30}= 0.204 D_{15}= 0.0834 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-6 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 43.5-45'

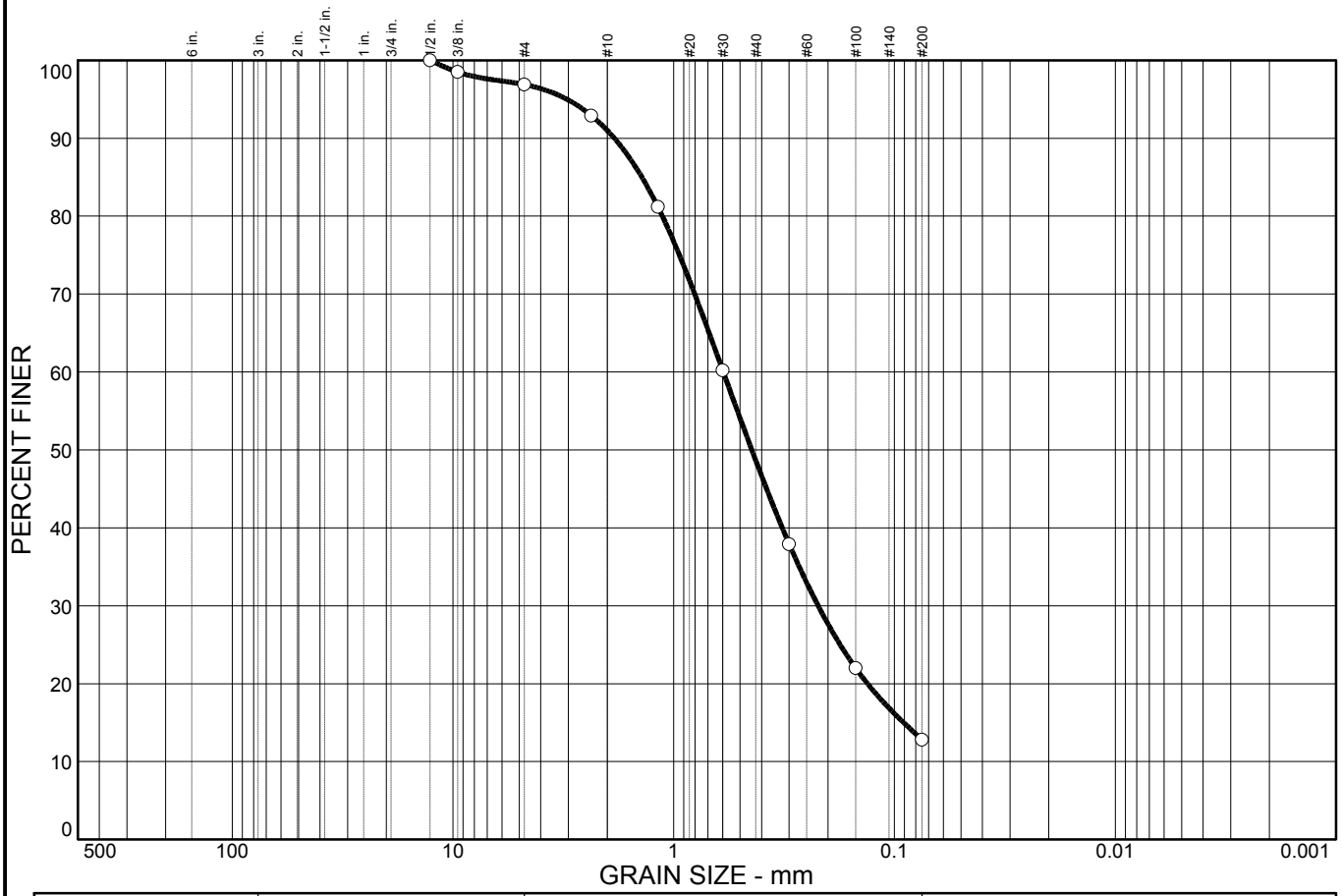
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 7

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	3.1	5.9	42.4	35.8	12.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	98.5		
#4	96.9		
#8	92.9		
#16	81.2		
#30	60.2		
#50	37.9		
#100	22.0		
#200	12.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.39 D_{60}= 0.596 D_{50}= 0.443
D_{30}= 0.222 D_{15}= 0.0909 D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-7 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 1.5-3'

Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 8

Particle Size Distribution Report

% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.9	5.8	34.6	42.1	16.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	99.1		
#8	95.2		
#16	84.4		
#30	67.9		
#50	49.0		
#100	29.8		
#200	16.6		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.21 D_{60}= 0.446 D_{50}= 0.311
D_{30}= 0.151 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-7 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 10-11.5'

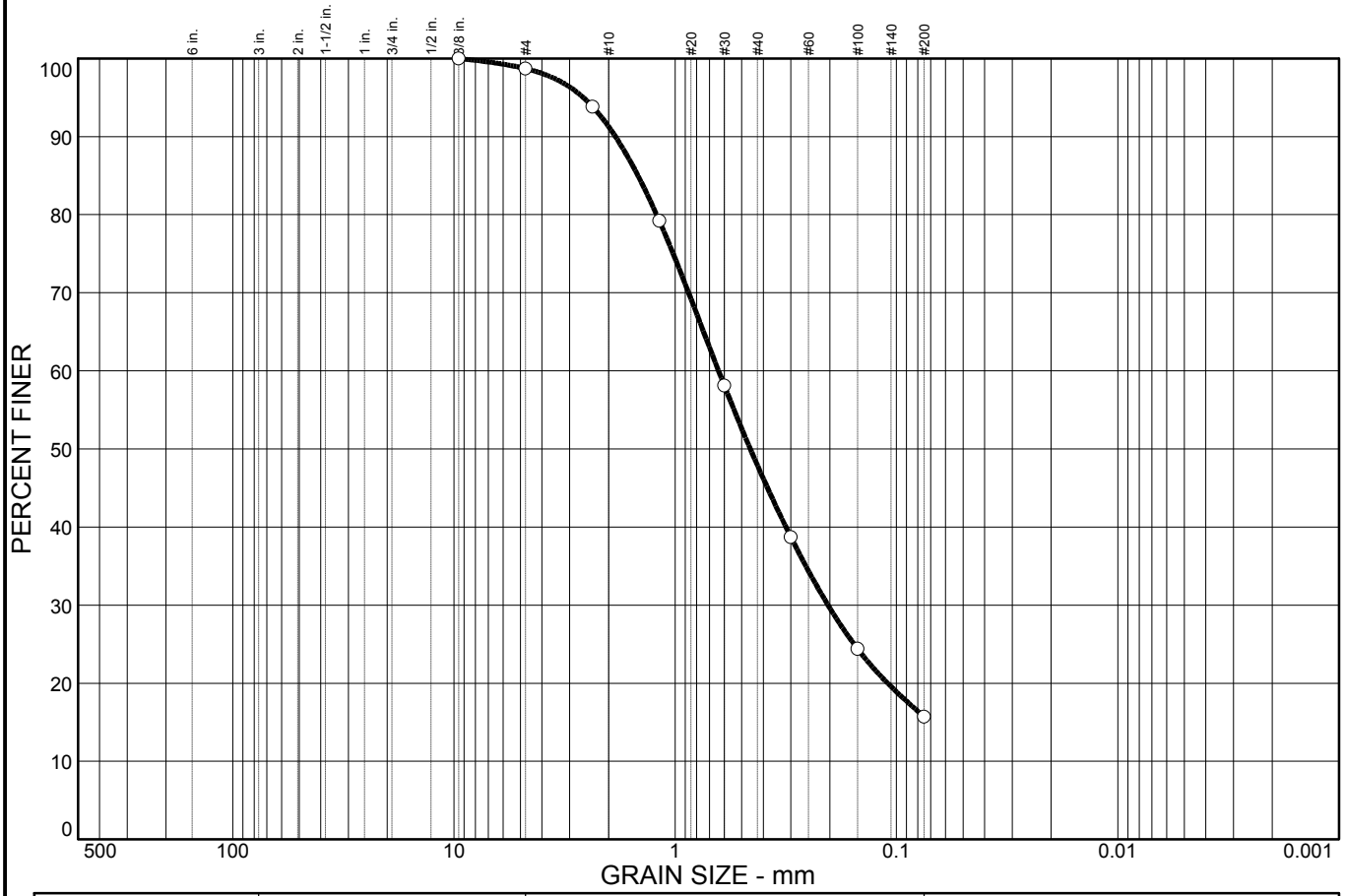
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 9

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.3	7.4	43.4	32.2	15.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100.0		
#4	98.7		
#8	93.8		
#16	79.2		
#30	58.1		
#50	38.7		
#100	24.4		
#200	15.7		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.47 D_{60}= 0.637 D_{50}= 0.458
D_{30}= 0.204 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample No.: B-8 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 0-3.5'

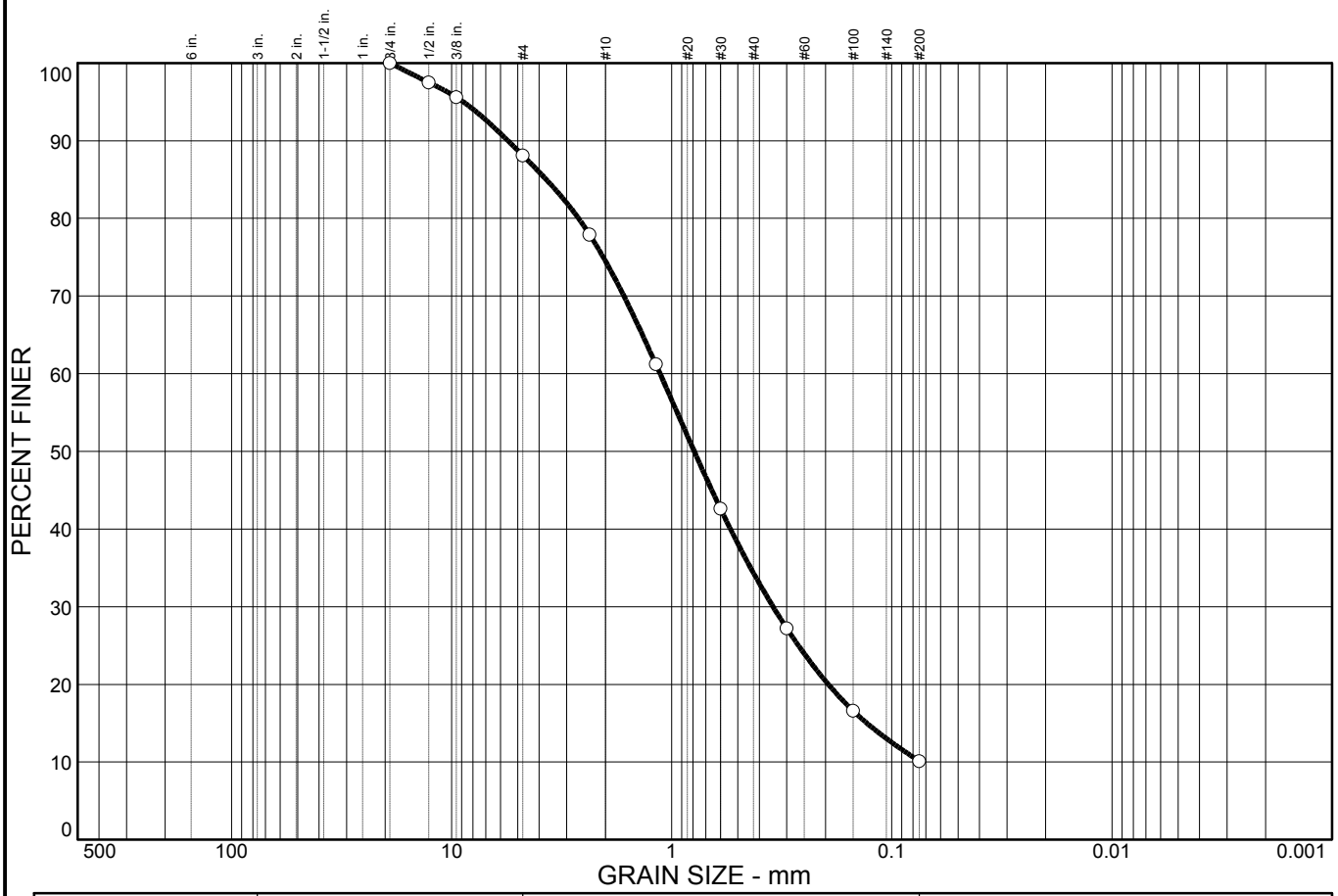
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 10

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	11.9	13.6	40.2	24.2	10.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	97.5		
3/8 in.	95.6		
#4	88.1		
#8	77.9		
#16	61.2		
#30	42.6		
#50	27.2		
#100	16.6		
#200	10.1		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 3.71 D_{60}= 1.13 D_{50}= 0.790
D_{30}= 0.346 D_{15}= 0.130 D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample No.: P-2 **Source of Sample:** **Date:** 8/16/24
Location: **Elev./Depth:** 38.5-40'

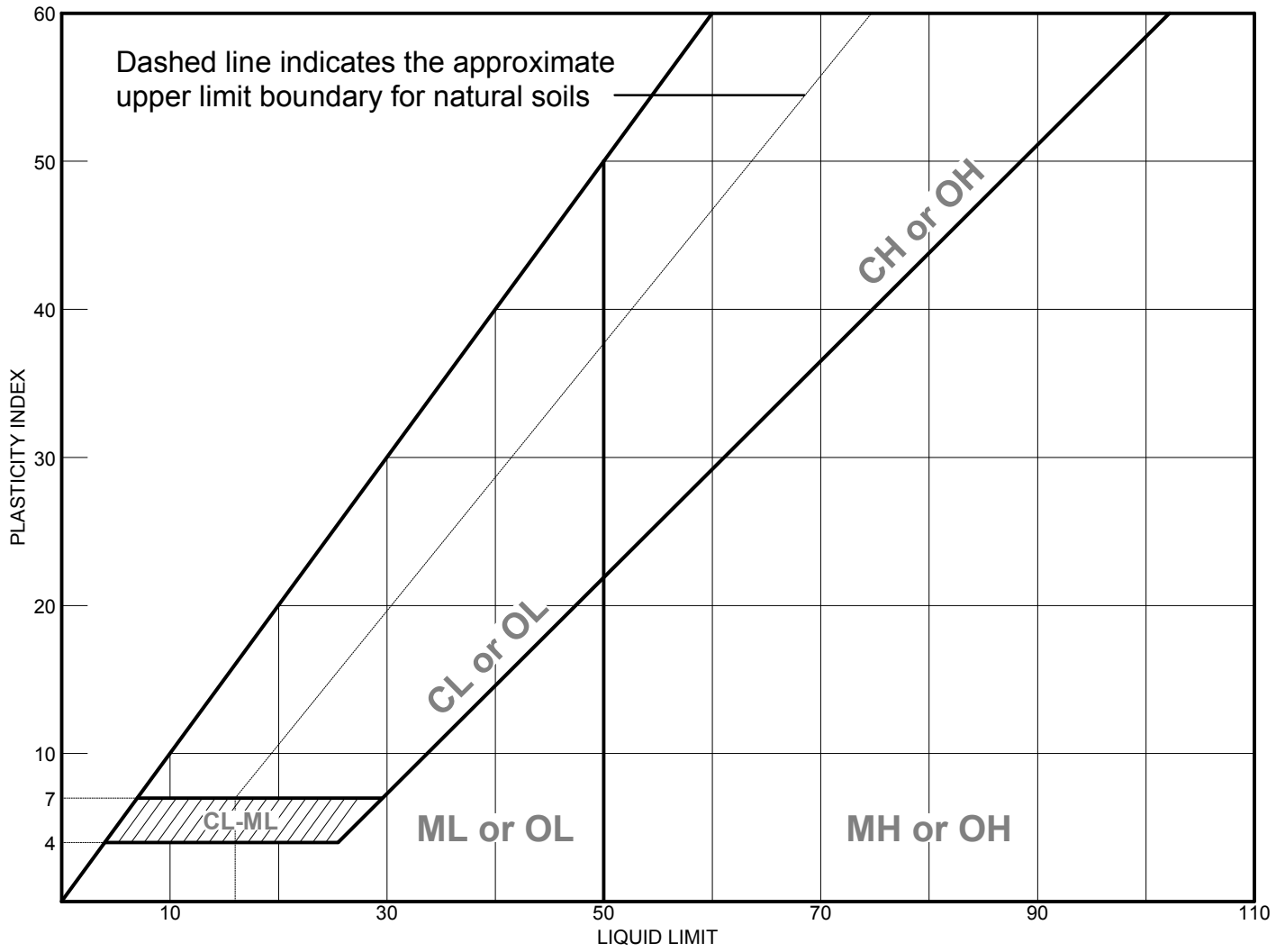
Moore Twining Associates, Inc.

Fresno, CA

Client: Circle K Stores, Inc.
Project: Circle K Yucca Valley

Project No: G28851.02 **Figure** 11

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly graded sand with silt	NV	NP	NP	42.5	11.9	SP-SM

Project No. G28851.02 **Client:** Circle K Stores, Inc.

Project: Circle K Yucca Valley

● **Source:** **Sample No.:** B-6 **Elev./Depth:** 0-3.5'

Remarks:

●

Moore Twining Associates, Inc.

Fresno, CA

Figure 12

MOORE TWINING ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME:	Circle K Yucca Valley	REPORT DATE:	8/26/2024
		TEST DATE:	8/21/2024
MTA PROJECT NO.:	G28851.02		
SAMPLE I.D.:	B-6 @ 0-3'		
SAMPLED BY:	GM		
SAMPLE DATE:	8/16/2024	TESTED BY:	CJ
MATERIALS DESCRIPTION:	Poorly graded sand with silt		
% PASSING # 4 SIEVE	100		

Initial Moisture Determination:		Final Moisture Determination:	
Pan + Wet Soil Wt., gm	250.0	Wet Soil Wt., lbs	1.0214
Pan + Dry Soil Wt., gm	234.4	Dry Soil Wt., lbs	0.9081
Pan Wt., gm	0.0		
Initial % Moisture Content	6.7	Final % Moisture Content	12.5

Initial Expansion Data:		**Final Expansion Data:**	
Ring + Sample Wt., lbs	0.9685	Ring + Sample Wt., lbs	1.0214
Ring Wt., lbs	0.0000	Ring Wt., lbs	0.0000
Remolded Wt., lbs	0.9685	Remolded Wt., lbs	1.0214
Remolded Wet Density, pcf	133.2	Remolded Wet Density, pcf	140.6
Remolded Dry Density, pcf	124.9	Remolded Dry Density, pcf	125.0

Expansion Data:		Initial Volume	Final Volume
		0.00727222	0.007266
Initial Gage Reading, in:	0.0477		
Final Gage Reading, in:	0.0469		
Expansion, in:	-0.0008		
Expansion Index	0	**Comments:**	**Very Low Expansion Potential**

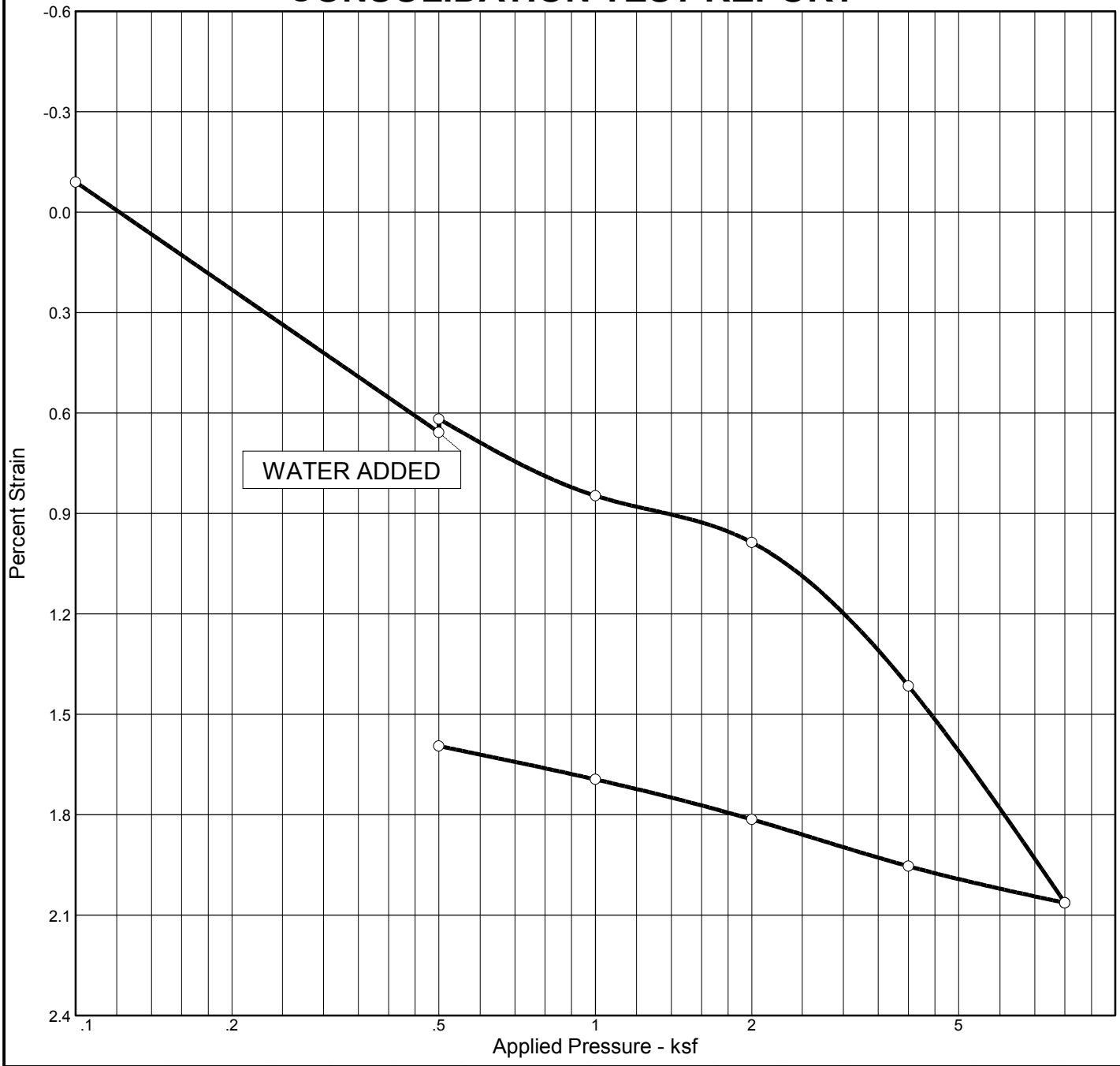
Classification of Expansive Soils. (Table No.1 From ASTM D4829)

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

Figure 13

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

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
30.8 %	4.6 %	118.4			2.65		2.75	0.03	0.01	0.55	0.1	0.397

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G28851.02 **Client:** Circle K Stores, Inc.

Project: Circle K Yucca Valley

Source: **Sample No.:** B-7 **Elev./Depth:** 1.5-3'

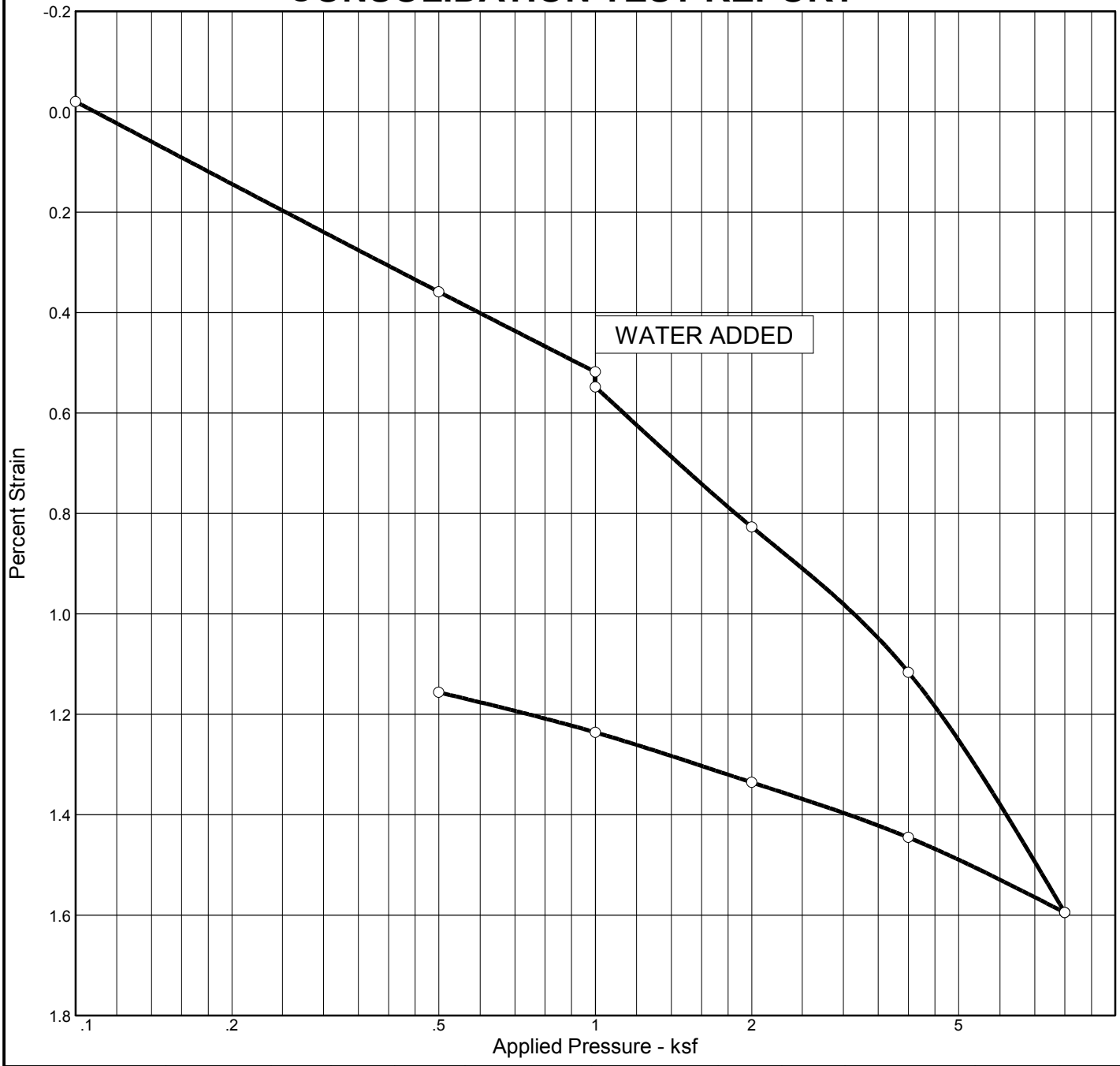
Moore Twining Associates, Inc.

Fresno, CA

Remarks:

Figure 14

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.											
21.0 %	3.8 %	112.0			2.65		4.24	0.02	0.01			0.477

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G28851.02 **Client:** Circle K Stores, Inc.

Project: Circle K Yucca Valley

Source: **Sample No.:** B-6 **Elev./Depth:** 5-6.5'

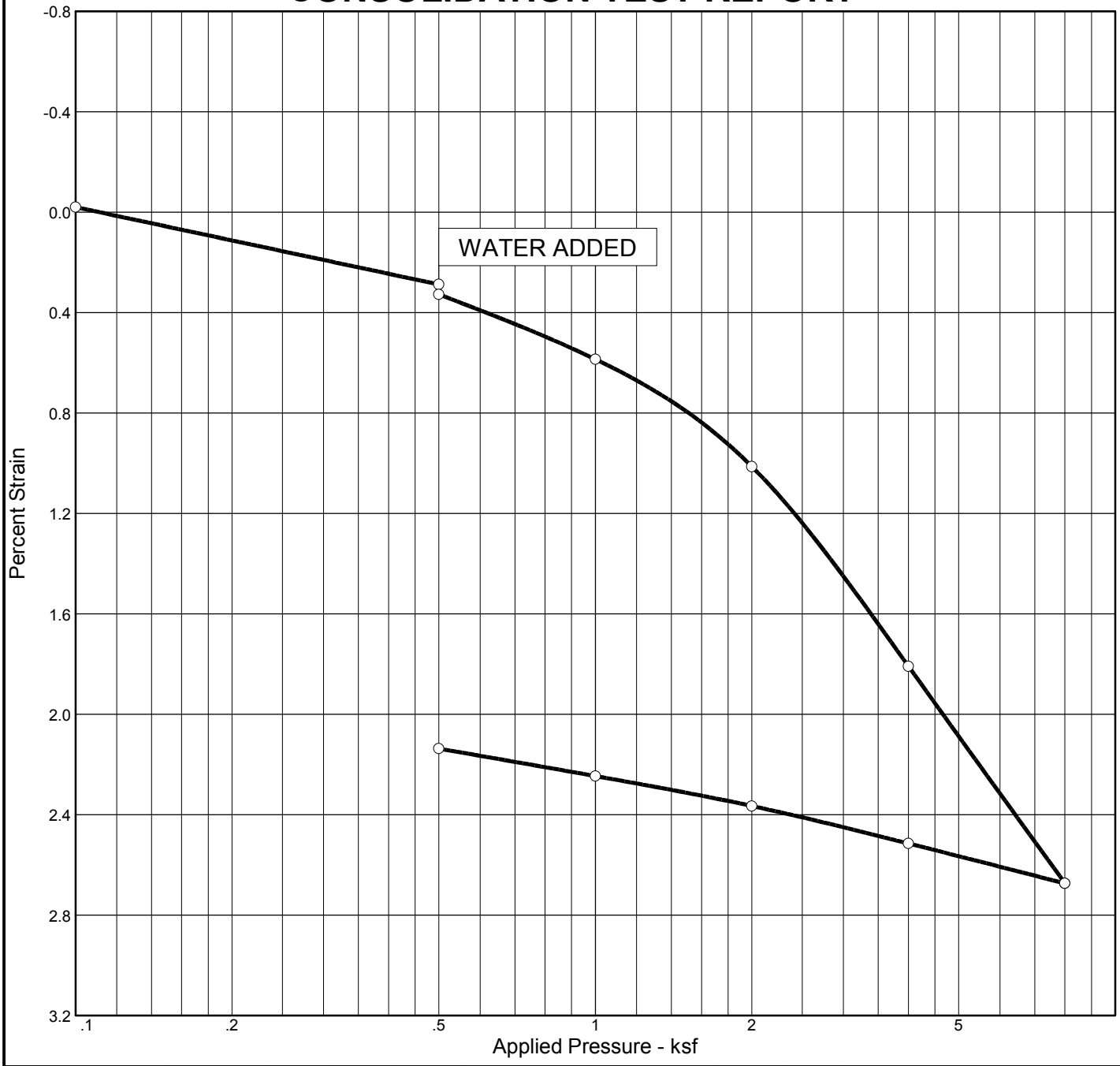
Remarks:

Moore Twining Associates, Inc.

Fresno, CA

Figure 15

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.											
28.6 %	5.3 %	111.0			2.65		2.07	0.04	0.01			0.490

MATERIAL DESCRIPTION	USCS	AASHTO
Silty sand	SM	

Project No. G28851.02 **Client:** Circle K Stores, Inc.

Project: Circle K Yucca Valley

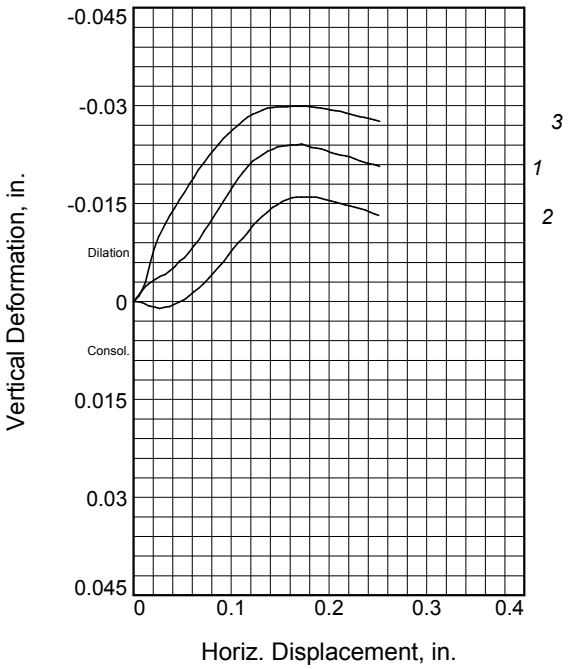
Source: **Sample No.:** B-6 **Elev./Depth:** 2-3.5'

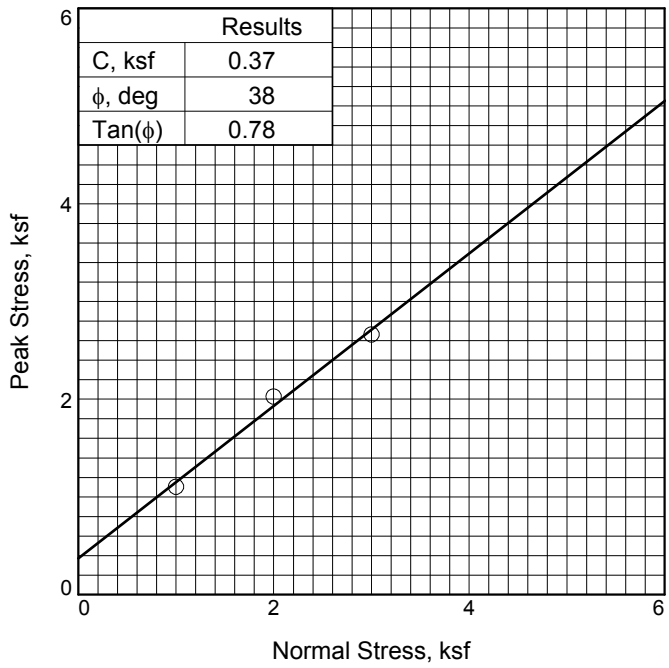
Moore Twining Associates, Inc.

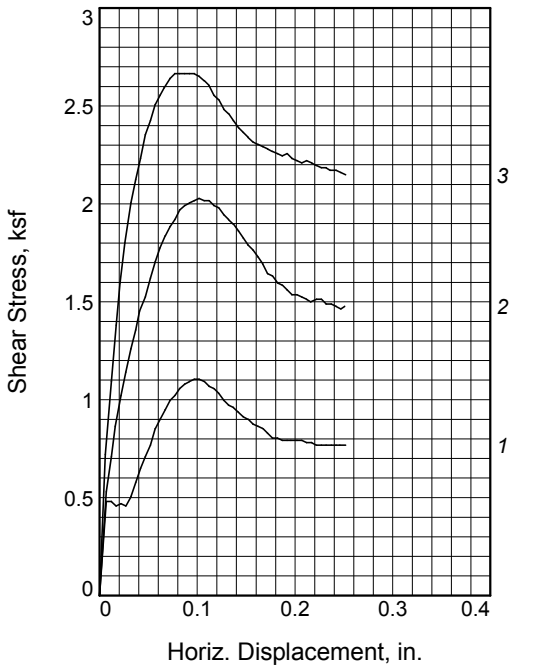
Fresno, CA

Remarks:

Figure 16







Sample No.		1	2	3
Initial	Water Content, %	4.6	5.3	5.6
	Dry Density, pcf	118.0	114.9	117.8
	Saturation, %	30.6	31.8	36.5
	Void Ratio	0.4020	0.4395	0.4050
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	14.8	16.0	14.7
	Dry Density, pcf	118.0	115.0	118.0
	Saturation, %	97.5	96.7	96.7
	Void Ratio	0.4018	0.4381	0.4021
	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		1.10	2.03	2.66
Displacement, in.		0.10	0.10	0.08
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		4.0	4.2	3.2

Sample Type:
Description: Silty sand

Specific Gravity= 2.65
Remarks:

Figure 17

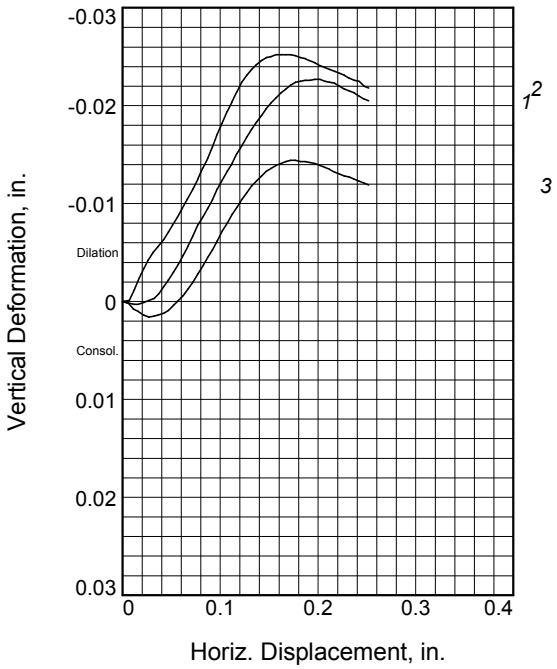
Client: Circle K Stores, Inc.

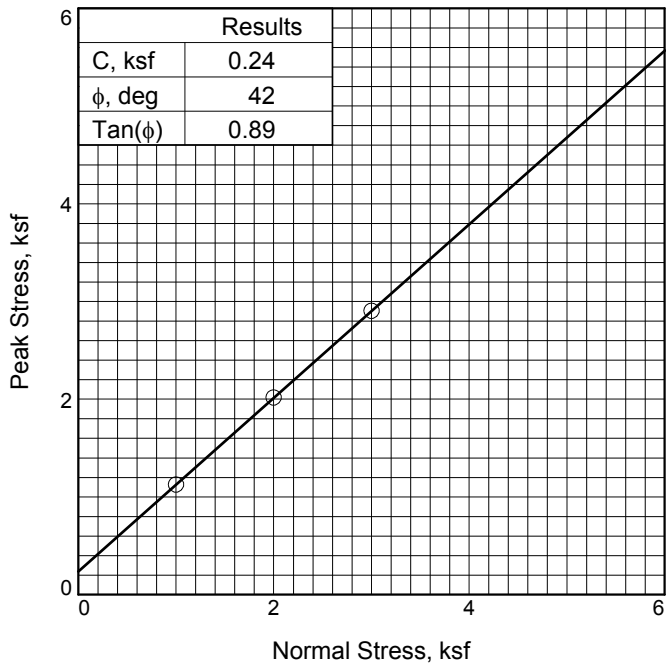
Project: Circle K Yucca Valley

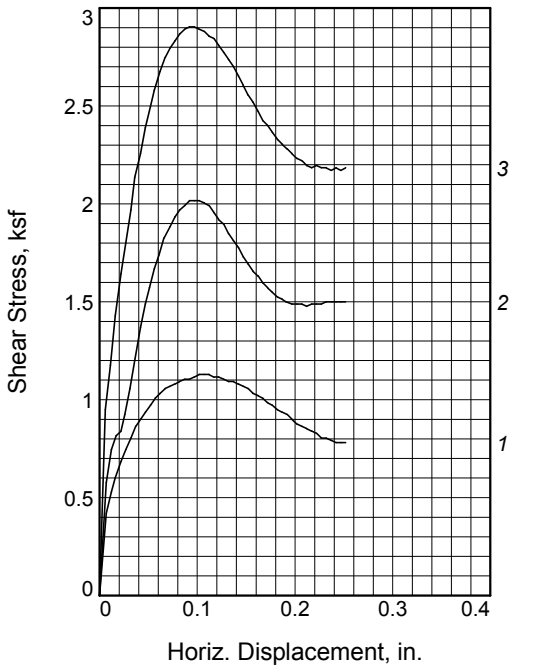
Sample Number: B-4 **Depth:** 5-6.5'

Proj. No.: G28851.02 **Date Sampled:** 8/16/24

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







Sample No.		1	2	3
Initial	Water Content, %	5.8	5.0	7.3
	Dry Density, pcf	113.2	112.1	109.2
	Saturation, %	33.3	27.7	37.6
	Void Ratio	0.4617	0.4758	0.5151
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.1	17.5	18.9
	Dry Density, pcf	113.4	112.3	109.4
	Saturation, %	98.7	98.0	97.7
	Void Ratio	0.4595	0.4731	0.5119
	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		1.13	2.02	2.90
Displacement, in.		0.10	0.09	0.09
Ultimate Stress, ksf				
Displacement, in.				
Strain at peak, %		4.2	3.8	3.8

Sample Type:
Description: Silty sand

Specific Gravity= 2.65
Remarks:

Figure 18

Client: Circle K Stores, Inc.

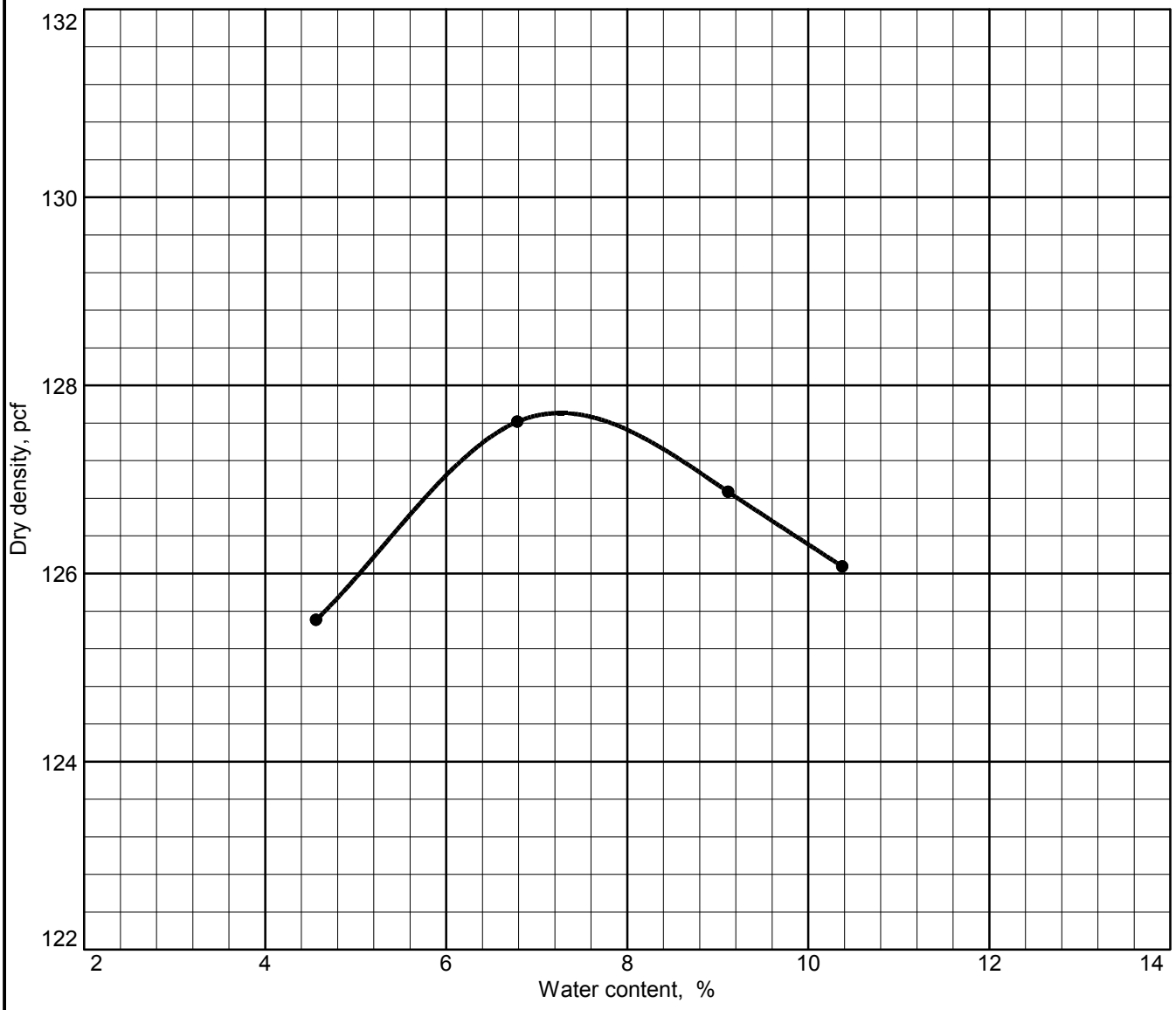
Project: Circle K Yucca Valley

Sample Number: B-7 **Depth:** 1.5-3'

Proj. No.: G28851.02 **Date Sampled:** 8/16/24

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

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0-3.5'	SM						1.3	15.7

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 127.7 pcf Optimum moisture = 7.3 %	Silty sand

Project No. G28851.02 **Client:** Circle K Stores, Inc.
Project: Circle K Yucca Valley

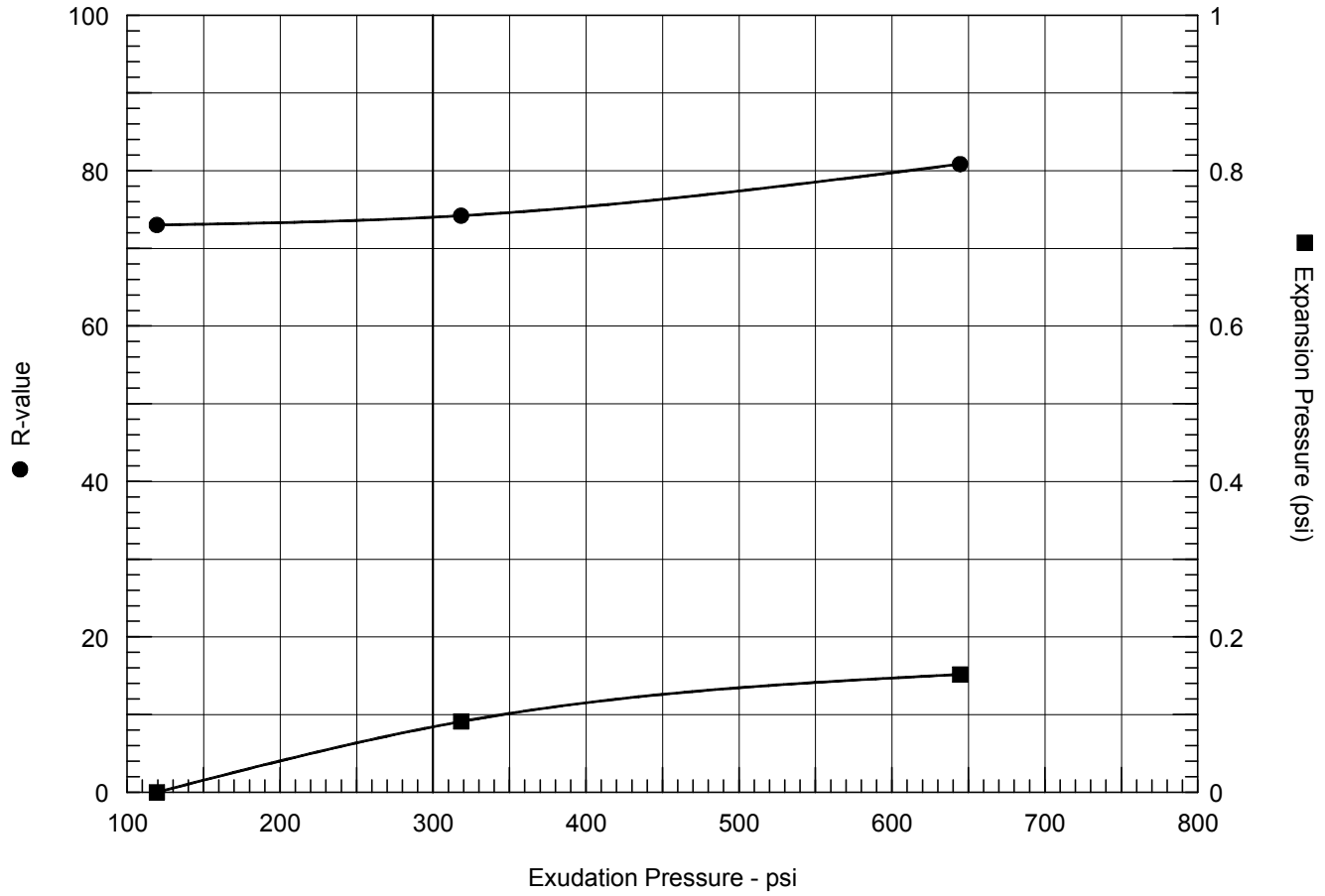
● **Source:** **Sample No.:** B-8 **Elev./Depth:** 0-3.5'

Remarks:

Moore Twining Associates, Inc.
Fresno, CA

Figure 19

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	125.1	8.8	0.15	18	2.62	645	79	81
2	350	124.0	9.7	0.09	24	2.64	318	71	74
3	350	122.6	10.6	0.00	29	2.66	119	70	73

Test Results	Material Description
R-value at 300 psi exudation pressure = 74 **Exp. pressure at 300 psi exudation pressure =** 0.08 psi	Silty sand

Project No.: G28851.02
Project: Circle K Yucca Valley
Sample Number: B-2/P-1 **Depth:** 0-3.5'
Date: 8/26/2024

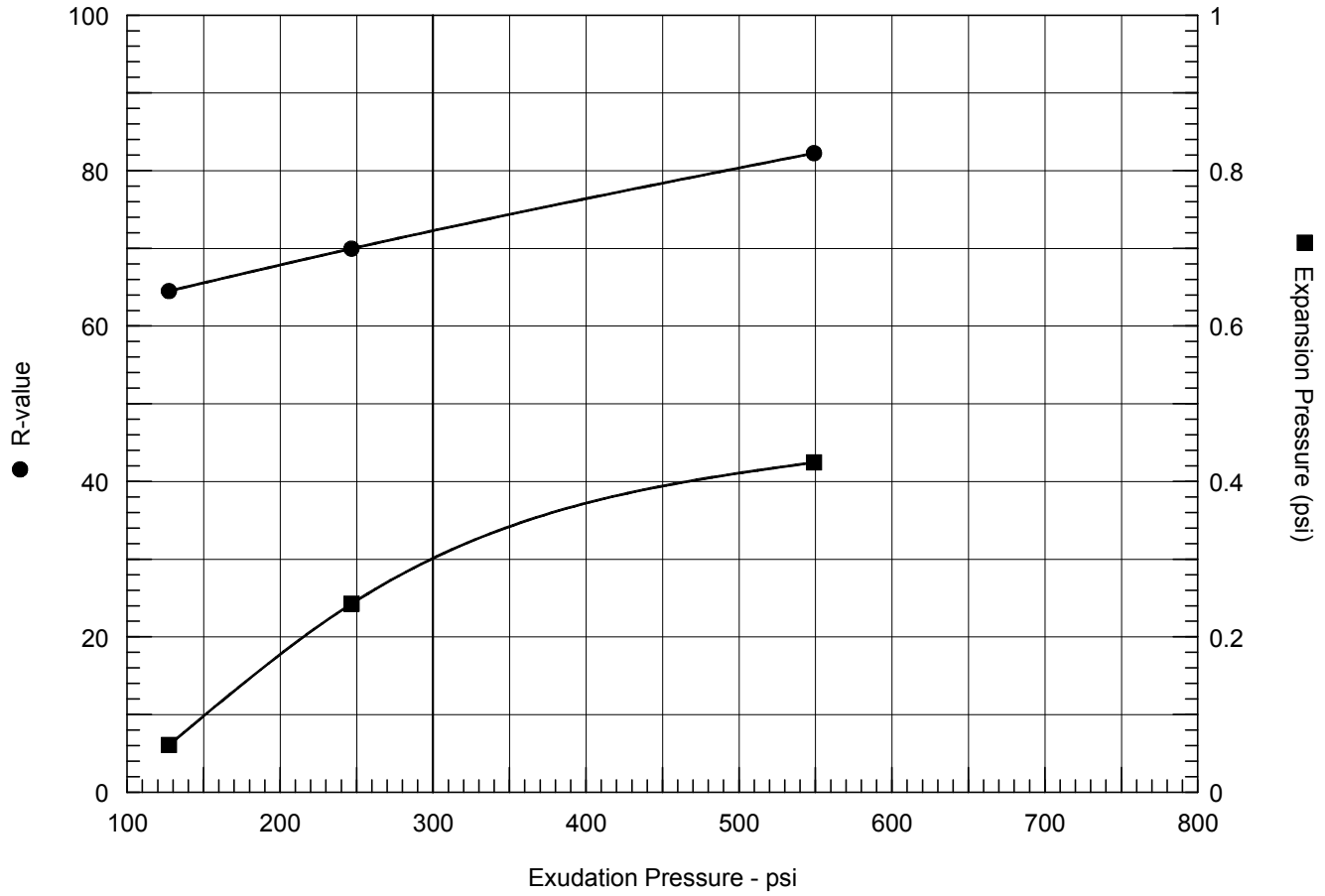
Tested by: CG
Checked by: MS
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure 20

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	127.6	9.4	0.42	17	2.54	549	82	82
2	350	128.2	10.3	0.06	34	2.53	127	64	64
3	350	128.5	9.8	0.24	29	2.52	247	70	70

Test Results	Material Description
R-value at 300 psi exudation pressure = 72 **Exp. pressure at 300 psi exudation pressure =** 0.30 psi	Silty sand

Project No.: G28851.02
Project: Circle K Yucca Valley
Sample Number: B-5/P-3 **Depth:** 0-3.5'
Date: 8/26/2024

Tested by: CG
Checked by: MS
Remarks:

R-VALUE TEST REPORT

Moore Twining Associates, Inc.

Figure 21

MOORE TWINING ASSOCIATES, INC.
Since 1998
California ELAP Certificate #1371

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

August 23, 2024

Work Order #: **KH20001**

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

RE: Circle K- Yucca Valley

Enclosed are the analytical results for samples received by our laboratory on **08/20/24** . For your reference, these analyses have been assigned laboratory work order number **KH20001**.

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 22

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division	**Project:** Circle K- Yucca Valley	**Reported:**
2527 Fresno Street	**Project Number:** G28851.02	08/23/2024
Fresno CA, 93721	**Project Manager:** Allen Harker	

Analytical Report for the Following Samples

Sample ID	Notes	Laboratory ID	Matrix	Date Sampled	Date Received
B-6 @ 0-3		KH20001-01	Soil	08/17/24 00:00	08/20/24 08:29

MOORE TWINING ASSOCIATES, INC.
California ELAP Certificate #1371

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

MTA Geotechnical Division	**Project:** Circle K- Yucca Valley	**Reported:**
2527 Fresno Street	**Project Number:** G28851.02	08/23/2024
Fresno CA, 93721	**Project Manager:** Allen Harker	

B-6 @ 0-3

KH20001-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Flag
Inorganics								
Chloride	**0.0018**	0.00060	% by Weight	[CALC]	08/22/24	08/22/24	[CALC]	
Chloride	**18**	6.0	mg/kg	B4H2120	08/21/24	08/22/24	Cal Test 422	
pH	**8.1**	0.10	pH Units	B4H2120	08/21/24	08/22/24	Cal Test 643 M	
Sulfate as SO4	**0.0033**	0.00060	% by Weight	[CALC]	08/22/24	08/22/24	[CALC]	
Sulfate as SO4	**33**	6.0	mg/kg	B4H2120	08/21/24	08/22/24	Cal Test 417	

Notes and Definitions

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.

Project Name:	Circle K Yucca Valley	Report Date:	8/26/2024
		Sample Date:	8/16/2024
Project Number:	G28851.02		
		Sampled By:	GM
Subject:	Minimum Resistivity, ASTM G187	Tested By:	RS
Material Description:	Poorly graded sand with silt	Test Date:	8/26/2024
Location:	B-6 @ 0-3.5'		

Laboratory Test Results, Minimum Resistivity - ASTM G187

Total Water Added, mls	Resistivity, Ohm-cm
25 mls	32,000
50 mls	29,000
75 mls	25,000
100 mls	23,000
125 mls	21,000
150 mls	9,600
175 mls	9,200
200 mls	9,500

Remarks: Min. Resistivity is 9,200 Ohm-cm

Figure 23

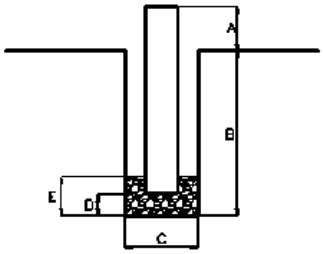
PH: 559.268.7021
FX: 559.268.7126
2527 Fresno Street
Fresno, CA 93721

APPENDIX D

RESULTS OF PERCOLATION TESTS

PERCOLATION TEST
No. B-2/P-1

Project: Proposed Circle K Store
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA
Coordinates:
Project No. G28851.02
Test Date: 8/18/2024



A. Top of Pipe Above Ground	0 Inches
B. Depth of Hole	122 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	24 Inches
F. Pipe Length	120 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: to 13 inches from bottom on 8/17/24 at 14:55
Checked 0 inches from bottom on the morning of 8/18/24

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)
Refill 1	8/18/2024	10:16:00	8				
	8/18/2024	10:16:23	8.5	0.38	6	0.2	28.9
Refill 2	8/18/2024	10:17:00	8				
	8/18/2024	10:17:31	8.5	0.52	6	0.2	21.5
Refill 3	8/18/2024	10:19:00	8				
	8/18/2024	10:21:00	9.01	2.00	12.12	0.4	12.8
Refill 4	8/18/2024	10:22:00	8				
	8/18/2024	10:24:00	8.96	2.00	11.52	0.4	12.0
Refill 5	8/18/2024	10:26:00	8				
	8/18/2024	10:28:00	8.89	2.00	10.68	0.5	10.9
Refill 6	8/18/2024	10:29:00	8				
	8/18/2024	10:31:00	8.86	2.00	10.32	0.5	10.4
Refill 7	8/18/2024	10:32:00	8				
	8/18/2024	10:34:00	8.85	2.00	10.2	0.5	10.3
Refill 8	8/18/2024	10:35:00	8				
	8/18/2024	10:37:00	8.85	2.00	10.2	0.5	10.4
Refill 9	8/18/2024	10:38:00	8				
	8/18/2024	10:40:00	8.81	2.00	9.72	0.5	9.8
Refill 10	8/18/2024	10:41:00	8				
	8/18/2024	10:43:00	8.8	2.00	9.6	0.5	9.7
Refill 11	8/18/2024	10:44:00	8				
	8/18/2024	10:46:00	8.8	2.00	9.6	0.5	9.7
Refill 12	8/18/2024	10:49:00	8				

* Depth to water measured from top of pipe

PERCOLATION TEST
No. B-2/P-1

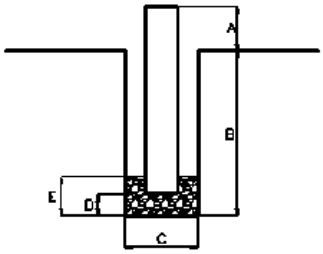
Project: Proposed Circle K Store **Project No.** G28851.02
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA **Test Date:** 8/18/2024
Coordinates:

	8/18/2024	10:51:00	8.8	2.00	9.6	0.5	9.7
Refill 13	8/18/2024	10:52:00	8				
	8/18/2024	10:54:00	8.78	2.00	9.36	0.5	9.4
Refill 14	8/18/2024	10:55:00	8				
	8/18/2024	10:57:00	8.77	2.00	9.24	0.6	9.3
Refill 15	8/18/2024	10:58:00	8				
	8/18/2024	11:00:00	8.77	2.00	9.24	0.6	9.3
Refill 16	8/18/2024	11:01:00	8				
	8/18/2024	11:03:00	8.77	2.00	9.24	0.6	9.3
Refill 17	8/18/2024	11:05:00	8				
	8/18/2024	11:07:00	8.76	2.00	9.12	0.6	9.1
Refill 18	8/18/2024	11:08:00	8				
	8/18/2024	11:10:00	8.76	2.00	9.12	0.6	9.1
Refill 19	8/18/2024	11:12:00	8				
	8/18/2024	11:14:00	8.77	2.00	9.24	0.6	9.3
Refill 20	8/18/2024	11:16:00	8				
	8/18/2024	11:18:00	8.76	2.00	9.12	0.6	9.1
Refill 21	8/18/2024	11:19:00	8				
	8/18/2024	11:21:00	8.76	2.00	9.12	0.6	9.1

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

Project: Proposed Circle K Store
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA
Coordinates:
Project No. G28851.02
Test Date: 8/18/2024



A. Top of Pipe Above Ground	4 Inches
B. Depth of Hole	478 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	24 Inches
F. Pipe Length	480 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: to 12 inches from bottom on 8/17/24 at 14:45
Checked 0 inches from bottom on morning of 8/18/24

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)
Refill 1	8/18/2024	6:51:30	38				
	8/18/2024	6:51:55	38.5	0.42	6	0.2	26.6
Refill 2	8/18/2024	6:51:55	38.5				
	8/18/2024	6:52:52	39	0.95	6	0.4	15.4
Refill 3	8/18/2024	7:00:00	38.01				
	8/18/2024	7:02:00	39.07	2.00	12.72	0.4	13.7
Refill 4	8/18/2024	7:04:00	38				
	8/18/2024	7:06:00	39.01	2.00	12.12	0.4	12.8
Refill 5	8/18/2024	7:08:00	38				
	8/18/2024	7:10:00	39	2.00	12	0.4	12.6
Refill 6	8/18/2024	7:11:00	38				
	8/18/2024	7:13:00	38.99	2.00	11.88	0.4	12.4
Refill 7	8/18/2024	7:15:00	38				
	8/18/2024	7:17:00	38.98	2.00	11.76	0.4	12.3
Refill 8	8/18/2024	7:18:00	38				
	8/18/2024	7:20:00	38.98	2.00	11.76	0.4	12.5
Refill 9	8/18/2024	7:22:00	38				
	8/18/2024	7:24:00	38.95	2.00	11.4	0.4	12.0
Refill 10	8/18/2024	7:25:00	38				
	8/18/2024	7:27:00	38.94	2.00	11.28	0.5	11.8
Refill 11	8/18/2024	7:29:00	38				
	8/18/2024	7:31:00	38.9	2.00	10.8	0.5	11.2
Refill 12	8/18/2024	7:32:00	38				

* Depth to water measured from top of pipe

PERCOLATION TEST
No. P-2

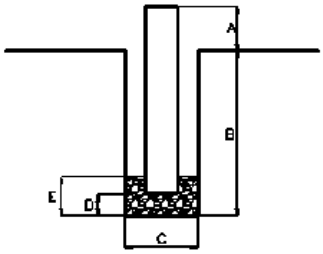
Project: Proposed Circle K Store **Project No.** G28851.02
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA **Test Date:** 8/18/2024
Coordinates:

	8/18/2024	7:34:00	38.9	2.00	10.8	0.5	11.2
Refill 13	8/18/2024	7:35:00	38				
	8/18/2024	7:37:00	38.9	2.00	10.8	0.5	11.2
Refill 14	8/18/2024	7:39:00	38				
	8/18/2024	7:41:00	38.9	2.00	10.8	0.5	11.2
Refill 15	8/18/2024	7:42:00	38				
	8/18/2024	7:44:00	38.9	2.00	10.8	0.5	11.2
Refill 16	8/18/2024	7:46:00	38				
	8/18/2024	7:48:00	38.9	2.00	10.8	0.5	11.2
Refill 17	8/18/2024	7:49:00	38				
	8/18/2024	7:51:00	38.9	2.00	10.8	0.5	11.2
Refill 18	8/18/2024	7:53:00	38				
	8/18/2024	7:55:00	38.9	2.00	10.8	0.5	11.2
Refill 19	8/18/2024	7:56:00	38				
	8/18/2024	7:58:00	38.9	2.00	10.8	0.5	11.2
Refill 20	8/18/2024	7:59:00	38				
	8/18/2024	8:01:00	38.9	2.00	10.8	0.5	11.2

* Depth to water measured from top of pipe

PERCOLATION TEST
No. B-5/P-3

Project: Proposed Circle K Store
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA
Coordinates:
Project No. G28851.02
Test Date: 8/18/2024



A. Top of Pipe Above Ground	0 Inches
B. Depth of Hole	122 Inches
C. Diameter of Hole	8 Inches
D. Depth of Gravel Below Pipe	2 Inches
E. Total Gravel Layer Depth	24 Inches
F. Pipe Length	120 Inches
G. Pipe Diameter	2 Inches

Pre-saturated: to 13 inches from bottom on 8/17/24 at 14:25
Checked 0 inches from bottom on morning of 8/18/24

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)
Refill 1	8/18/2024	8:43:00	7.5				
	8/18/2024	8:44:00	8	1.00	6	0.4	8.9
Refill 2	8/18/2024	8:44:00	8				
	8/18/2024	8:45:00	8.5	1.00	6	0.4	11.1
Refill 3	8/18/2024	8:50:00	8				
	8/18/2024	8:52:00	8.6	2.00	7.2	0.7	6.8
Refill 4	8/18/2024	8:53:00	8				
	8/18/2024	8:55:00	8.58	2.00	6.96	0.7	6.6
Refill 5	8/18/2024	8:56:00	8				
	8/18/2024	8:58:00	8.55	2.00	6.6	0.8	6.2
Refill 6	8/18/2024	5:59:00	8				
	8/18/2024	9:01:00	8.55	181.96	6.6	70.6	0.1
Refill 7	8/18/2024	9:02:00	8				
	8/18/2024	9:04:00	8.54	2.00	6.48	0.8	6.0
Refill 8	8/18/2024	9:05:00	8				
	8/18/2024	9:07:00	8.54	2.00	6.48	0.8	6.1
Refill 9	8/18/2024	9:08:00	8				
	8/18/2024	9:10:00	8.51	2.00	6.12	0.8	5.8
Refill 10	8/18/2024	9:11:00	8				
	8/18/2024	9:13:00	8.52	2.00	6.24	0.8	5.9
Refill 11	8/18/2024	9:15:00	8				
	8/18/2024	9:17:00	8.53	2.00	6.36	0.8	6.0
Refill 12	8/18/2024	9:18:00	8				

* Depth to water measured from top of pipe

PERCOLATION TEST
No. B-5/P-3

Project: Proposed Circle K Store **Project No.** G28851.02
Location: NWC of 29 Palms Hwy and Avalon Ave, Yucca Valley, CA **Test Date:** 8/18/2024
Coordinates:

	8/18/2024	9:20:00	8.52	2.00	6.24	0.8	5.9
Refill 13	8/18/2024	9:21:00	8				
	8/18/2024	9:23:00	8.52	2.00	6.24	0.8	5.9
Refill 14	8/18/2024	9:25:00	8				
	8/18/2024	9:27:00	8.53	2.00	6.36	0.8	6.0
Refill 15	8/18/2024	9:28:00	8				
	8/18/2024	9:30:00	8.51	2.00	6.12	0.8	5.8
Refill 16	8/18/2024	9:31:00	8				
	8/18/2024	9:33:00	8.51	2.00	6.12	0.8	5.8
Refill 17	8/18/2024	9:35:00	8				
	8/18/2024	9:37:00	8.52	2.00	6.24	0.8	5.9
Refill 18	8/18/2024	9:38:00	8				
	8/18/2024	9:40:00	8.51	2.00	6.12	0.8	5.8
Refill 19	8/18/2024	9:41:00	8				
	8/18/2024	9:43:00	8.51	2.00	6.12	0.8	5.8
Refill 20	8/18/2024	9:45:00	8				
	8/18/2024	9:47:00	8.51	2.00	6.12	0.8	5.8
Refill 21	8/18/2024	9:49:00	8				
	8/18/2024	9:51:00	8.51	2.00	6.12	0.8	5.8

* Depth to water measured from top of pipe

APPENDIX E

FAULT TRENCH INVESTIGATION REPORT

REPORT OF LIMITED FAULT HAZARD EVALUATION
PROPOSED CONVENIENCE STORE AND FUEL CANOPIES
APN 0601-543-01
AVALON AVENUE
YUCCA VALLEY
SAN BERNARDINO COUNTY
CALIFORNIA

PROJECT NO.: 1664-01
REPORT NO.: 1

AUGUST 22, 2024

SUBMITTED TO:

MOORE TWINING ASSOCIATES, INC.
2527 FRESNO STREET
FRESNO, CA 93721

PREPARED BY:

HILLTOP GEOTECHNICAL, INC.
786 SOUTH GIFFORD AVENUE
SAN BERNARDINO, CA 92408

HILLTOP GEOTECHNICAL
INCORPORATED

786 S. GIFFORD AVENUE • SAN BERNARDINO • CA 92408
Phone **909-890-9079** • FAX 909-890-9055
hilltopg@hgeotech.com

August 22, 2024

Moore Twining Associates, Inc.
2527 Fresno Street
Fresno, CA 93721

Project No.: 1664-01
Report No.: 1

Attention: Mr. Read Andersen

Subject: **Report of a Limited Fault Hazard Evaluation, Proposed Convenience Store and Fuel Canopies, APN 0601-543-01, Avalon Avenue, Yucca Valley, California.**

References:
1. **Greenberg Farrow,** October 16, 2023, *Site Plan, NWC 29 Palms Highway & Avalon Avenue, Yucca Valley, Sheet Number CSP 6.0W, Project Number 20221223.0.*
2. Technical References – See Appendix 'B.'

Gentleman:

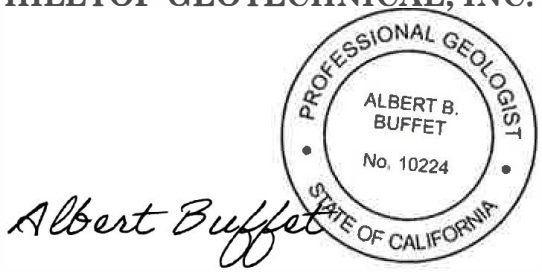
According to your request, we have completed a limited fault hazard evaluation for design and construction of the proposed convenience store and fuel canopies at the subject site. The purpose of this investigation was to evaluate the potential hazard for fault related ground rupture within the project site located within the A-P Earthquake Fault Zone. We are presenting, herein, our findings and recommendations.

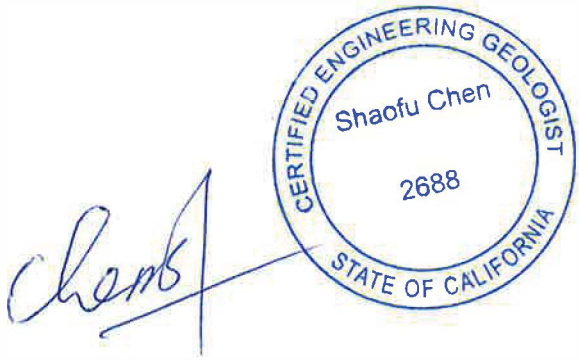
Per our review of applicable geologic reports and faulting literature, analysis of aerial photographs, geologic reconnaissance, excavation of two exploratory fault trenches to depths of up to approximately 12 feet, and evaluation and geologic

logging of all exploratory fault trenches, no fault trace evidence was found at area of the proposed convenience store including 50-foot setbacks from both ends of the proposed convenience store, the proposed convenience store at the project area is considered feasible.

If you have any questions after reviewing the findings and recommendations contained in the attached report, please do not hesitate to contact this office. This opportunity to be of professional service is sincerely appreciated.

Respectfully Submitted,
HILLTOP GEOTECHNICAL, INC.





Albert Buffet, P.G. 10224
Professional Geologist

S. Mack Chen, C.E.G. 2688
Principal Geologist

Distribution: (1) Addressee
Via Email
Mr. Read Andersen ReadA@mooretwining.com

REPORT OF LIMITED FAULT HAZARD EVALUATION
PROPOSED CONVENIENCE STORE AND FUEL CANOPIES
APN 0601-543-01, AVALON AVENUE
YUCCA VALLEY
SAN BERNARDINO COUNTY
CALIFORNIA

AUGUST 22, 2024

INTRODUCTION

This report presents results of a limited fault hazard evaluation conducted on the subject site for the proposed convenience store and fuel canopies to be located at APN 0601-543-01, northwest corner of 29 Palms Highway & Avalon Avenue in the Yucca Valley area of San Bernardino County, CA 92284. The general location of the subject site is indicated on the 'Site Location Map' Figure No. 1.

Authorization to perform this study was in the form of a signed agreement and proposal from **Hilltop Geotechnical, Inc. (HGI)** (Geotechnical / Geologic Consultant) to **Moore Twining Associates, Inc.** (Client), dated June 28, 2024 and January 15, 2024, Proposal No.: P23001R.

PURPOSE AND SCOPE OF STUDY

The purpose of this investigation was to evaluate the potential hazard for fault related ground rupture within the project site located within the A-P Earthquake Fault Zone. The scope of work performed for this study was to evaluate fault hazard in the vicinity of the proposed convenience store on the subject site with respect to potential fault hazard that may affect the proposed development of the site. The scope of work included the following:

a) Meet or discuss with representatives of pertinent personnel and determine the trench locations prior to trench excavation.

b) Research available pertinent geologic literature, geologic reports, and aerial photographs.

c) Excavate up to approximately 213 feet of exploratory trench with a backhoe or excavator.

d) Log all subsurface excavations.

e) Walk through with the City's consulting geologist in the field to confirm fault hazard investigation trenches.

f) Backfill the trenches and perform observation and compaction test. Backfilled soil will be compacted to a minimum of 90 percent of the maximum dry density. A compaction report for backfilling the fault trench will be prepared.

g) Establish reservoir setback zones, if needed.

h) Present our professional opinions in a report that will include, in addition to our conclusions and recommendations, a plot plan and trench logs.

The scope of work performed for this report did not include any testing of earth materials or groundwater for environmental purposes, an environmental assessment of the property, or opinions relating to the possibility of surface or subsurface contamination by hazardous or toxic substances.

This study was prepared for the exclusive use of Moore Twining Associates, Inc. and its clients for specific application to the development of the proposed convenience store and canopies in accordance with generally accepted standards of the geotechnical and geologic professions and generally accepted geologic principles and practices at the time this report was prepared. Other warranties, implied or expressed, are not made. Although reasonable effort has been made to obtain information regarding geologic conditions of the site, limitations exist with respect to knowledge of unknown regional or localized off-site conditions which may have an impact at the site. The conclusions presented in this report are valid as of the date of this report. However, changes in conditions of a property can

occur with passage of time, whether they are due to natural processes or to works of man on this and/or adjacent properties.

If conditions are observed or information becomes available during the design and construction process which are not reflected in this report, **HGI**, as Geological Engineering Consultant of record for the project, should be notified so that supplemental evaluations can be performed and conclusions and recommendations presented in this report can be verified or modified in writing, as necessary. Changes in applicable or appropriate standards of care in the geological professions occur, whether they result from legislation or the broadening of knowledge and experience. Accordingly, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes outside the influence of the project Geologic Engineering Consultant which will occur in the future.

PREVIOUS SITE STUDIES

No previous geotechnical and/or geological studies for the subject site are known to have been performed or were made available for review at the time of this study, if any had been performed. However, A fault investigation was conducted by Sladden Engineering (Cohrt et al, 2017) for a proposed retail center at APN: 0601-201-41, NWC Avalon Avenue and Palisade Drive, Yucca Valley, which is located immediately south of 29 Palms Highway right opposite to the subject site over 29 Palms Highway. This report was referenced to characterize soil stratigraphy in our fault study trenches.

SITE DESCRIPTION

The subject site comprises approximately 6.32 acres and was irregular in shape, which is bounded by Twentynine Palms Highway to the southeast, by Avalon Avenue to the east, by Paxton Road to the north, by Diadem Drive to the west, and by a vacant lot to the southwest. At the time of our field exploration, the

subject site was a vacant lot with scattered Joshua or Yucca trees. The site was relatively flat. In general, the existing ground surface is regionally sloping towards the north.

Utilities consisting of electric, telephone, gas, sewer, water, as well as other unknown underground lines, were observed to be present along Twentynine Palms Highway.

REGIONAL GEOLOGIC SETTING

The site is located at the eastern Transverse Ranges Physiographic Province of California and within the central portion of Yucca Valley. Yucca Valley is located in the Mojave Desert region of the California high desert. The Pinto Mountain fault, which is located to the north and west of the site, forms the boundary between the eastern Transverse Ranges and the Mojave Desert Physiographic provinces. The eastern Transverse Ranges Physiographic province consists of elevated masses of mainly Mesozoic plutonic rocks, Paleozoic metasedimentary rocks and Precambrian gneiss. Such mountains are transected by roughly east-west trending, convergent (north-south compressional) deformational structures. The convergent deformational features of the eastern Transverse Ranges are a result of north-south crustal shorting due to plate tectonic movement that locally folds and uplifts the Little San Bernardino Mountains and lowers the intervening valleys, along with propagation of thrust faults (including blind thrusts) and infilling of valley basins with sediments (Cohrt, et al, 2017).

LOCAL GEOLOGIC SETTING

The project site is located to the east of the San Bernardino Mountains. Per the Dibblee geologic map (1967), the geologic units at the project site consist of recent Quaternary alluvial deposits (Qa) and older Quaternary alluvial deposits (Qoa). The geologic units observed within the San Bernardino Mountains, to the northwest and north of the site, consist of intrusive Miocene quartz monzonite

(qm) and older granitic rocks leucocratic quartz monzonite (lqm).

REGIONAL FAULTING

Significant active faults that cross the eastern Transverse Ranges and the Mojave Desert Physiographic provinces and have been defined by California Geologic Survey (CGS) as Earthquake Fault Zones in the vicinity of the project site are the Eureka Peak fault, the Johnson Valley fault, and the Pinto Mountain fault. The east-west trending Pinto Mountain fault is located between Eureka Peak and the Johnson Valley faults. The north to northwest trending Eureka Peak and Johnson Valley faults have generated damaging earthquakes, including the June 28, 1992 Moment Magnitude (Mw) 7.5 Landers earthquake. During the Landers earthquake event, earthquake activity extended across the Holocene-active Pinto Mountain fault (Cohrt et al, 2017).

The Eureka Peak fault is located approximately 0.6 miles southwest of the site per Earthquake Zone of Required Investigation (**Figure 3**). The Eureka Peak fault is included in an Alquist-Priolo Earthquake Fault Zone, designated by CGS, as a result of the Landers Earthquake. Investigations by Rasmussen & Associates documented up to 8 inches of right-lateral, strike-slip offset along the Eureka Peak fault associated with the Landers earthquake (Rasmussen, 1992).

The Johnson Valley fault is located about 2 miles north of the site. Ground surface rupture for the M_w7.5 Landers earthquake initiated along the Johnson Valley fault and propagated to the north-northwest along the northwest trending Homestead Vally, Emerson and Camp Rock faults, as well as numerous, previously unidentified, north to northeast trending faults (Rasmussen, 1992).

The Pinto Mountain fault is a major east-west trending fault, located approximately 0.3 miles north-northwest of the site per Dibblee Geologic Map of Joshua Tree Quadrangle (1967). The Pinto Mountain fault extends from at least

the Mission Creek fault (north branch of the San Andreas fault) west of Morongo Valley to the Turtle Mountains, 30 miles east of Twentynine Palms (Bader and Moyle, 1960; Dibblee, 1967, 1970; Bortugno and Spittler, 1986). The Pinto Mountain fault is a left-lateral, strike-slip fault generally considered to represent the southern structural boundary of the Mojave Block (Dibblee, 1968).

A well-defined zone of cracks, located near Avalon Avenue and 29 Palms Highway, was originally mapped by Rasmussen & Associates (1992) subsequent to the June 28, 1992 Landers Earthquake and later zoned by CGS (Treiman, 1993). Treiman (1993) indicates that although this short northwest trending fracture set is isolated and is not clearly related to any of the better defined 1992 fault rupture, the distinct left-stepping pattern is strongly suggestive of right-lateral shear (Cohrt et al, 2017). Per California Geological Survey Fault Activity Map of California https://maps.conservation.ca.gov/cgs/fam/, the well-defined zone of cracks is an unnamed fault within Eureka Peak fault zone, which is considered as a splay of Eureka Peak fault. This unnamed fault is also designated as a Special Studies Zone (**Figures 3 & 4**). Since this unnamed fault is close to Avalon Avenue, here we tentatively name this fault as Avalon Avenue Splay fault.

REGIONAL SEISMICITY

An area of triggered slip was observed during the 1992 Landers earthquake on the Pinto Mountain fault approximately 0.3 miles north and northwest of the site. Minor left-lateral cracks were observed on the ground surface (Bryant, 1992). Some other minor cracks were observed at the ground surface after the 1992 Landers earthquake (Bryant, 1992). The Mw 7.5 Landers earthquake occurred on June 28, 1992, and involved ground surface along the Johson valley, Kickapoo, Homestead Valley, Emerson, Camp Rock, Eureka Peak and Burnt Mountain faults. The epicenter of Landers earthquake is located approximately 5 miles north of the site. The Mw 6.2 Joshua Tree earthquake occurred on April 22, 1992,

and is considered to be a foreshock of the Landers earthquake (Hauksson, et al., 1992).

Several earthquakes with Richter magnitudes between 4.0 and 5.0 have occurred along the Pinto Mountain fault between Yucca Valley and Twentynine Palms during historic time. This fault has been documented as disrupting Holocene material in the Joshua Tree and Twentynine Palms (Rasmussen, 1977, 1990) and impedes the flow of ground water in alluvial materials in Twentynine Palms. It also forms prominent scarps in older alluvium. The largest historic earthquake that may have been associated with the Pinto Mountain fault was M5.9 and was located east of Twentynine Palms.

Significant earthquakes affecting the site may occur on the Eureka Peak, Pinto Mountain, Johnson Valley faults during the lifetime of the proposed development. Due to the proximity of the site to the Eureka Peak, Pinto Mountain, Johnson Valley faults, near-field effects from strong ground motion associated with a large earthquake may occur at the site.

GROUNDWATER CONDITIONS

Per the DWR (2004), the subject site is located within the Colorado River Hydrologic Region and within the Warren Valley Groundwater Basin. Groundwater bearing formations in the basin have been reported to consist of continental deposits of Miocene to Quaternary age (DWR, 2004). Water-bearing geologic units are reported to occur in unconsolidated alluvium deposits that consist of gravels, conglomerates, and silts. The alluvial deposits are interpreted regionally to exceed a thickness of 2,000 feet. Groundwater flow near the site vicinity has a northward direction from the Little San Bernardino Mountains towards the Pinto Mountain fault (Cohrt et al, 2017).

Groundwater was not encountered to 12 feet bgs during our field trenching at the subject site. Per California Department of Water Resources Website: https://wdl.water.ca.gov/waterdatalibrary/Map.aspx, data from an observation well (State Well No. 01N06E29N001S) is located approximately 1,000 feet to the northeast of the subject site and is approximately 3192.7 feet above Mean Sea Level (MSL), which is about 20 feet lower than the subject site. The depth to groundwater in this well was 294 feet bgs measured on November 22, 1967. The groundwater at the subject site is not considered an important factor for the proposed development.

AERIAL PHOTOGRAPHIC REVIEW

HGI conducted a lineament analysis for the site by reviewing aerial photographs in Google Earth database. The following aerial photographs, which show the subject site and its vicinity, were reviewed: aerial photos date July 1989, October 1995, September 2003, June 2009, and June 2021.

Tonal, geomorphic, and vegetative lineaments were evaluated. No lineaments to be considered relevant to fault traces were found in the immediate vicinity of the subject site.

EXPLORATORY FAULT TRENCH INVESTIGATION

HGI's fault trench investigation of the site was conducted during July 2024. Two northeast trending exploratory trenches totaling approximately 218 feet in length were excavated across the building pad of the proposed convenience store to perpendicularly intercept the designated northwest trending Avalon Avenue Splay fault trace. The excavation contractor Norfork, Inc. was retained to conduct trench excavation.

Prior to commencing the trench excavation, four corners of the proposed building were surveyed and staked by a surveyor, which was hired by the client. DigAlert was also notified prior to the trench excavation. A John Deere 85G equipped with

a 4-foot-wide bucket was utilized by Norfork Inc to excavate exploratory trenches. The contractor performed the trench excavation under the direction and monitoring of HGI's field geologist. Following Cal OSHA criteria, the exploratory fault trenches were excavated utilizing symmetrical 3-foot-high step benches. The exploratory trenches were excavated to approximately 30 feet in width. Since a Joshua or Yucca Tree was located at the northeast end of the major trench T-1, a second minor trench T-2 paralleling to the major trench and continuing the major trench was excavated to meet a minimum of 50 feet of setback from the proposed building. The excavated trenches were re-surveyed to confirm the exploratory trenches meet the Code requirement after the completion of the trench excavation.

The exploratory trenches were excavated to expose the near surface soil to dismantle any fault traces or offset of the soil profile indicative of surface rupture. After two exploratory trenches were excavated, the trench side walls were brushed and scraped. The existing soil smears, debris and slough were removed. The trench walls were placed with field stationing marks every 20 feet on the horizontal. A thorough initial inspection of each trench was conducted and followed with detailed measurements and recording of observations. The profile of the logged sidewall, total length, and total depth were graphically drawn on a trench log. For elevation readings, a peep sight was used along with a tape measure and fiberglass level. The geologic logging was conducted by HGI's professional geologist. The trench wall soil profiles were logged at a scale of 1 inch equals 5 feet, which are shown in Plate Nos 2a through 2c. Exploratory trenching and geologic logging of the exploratory trenches were directed by HGI's field geologist under supervision of a California Certified Engineering Geologist.

Age of Subsurface Materials

Soil age can be determined using absolute and relative age dating methods. Absolute age dating can be conducted using radiometric age dating method on

carbon bearing samples such as charcoal, shell, bone fragments or wood that is encountered in certain soil stratum. Relative ages can be conducted by soil stratigraphy studies including dry color, moist color, moist content, texture, structure, consistency, cementation, calcium carbonate stage level, etc. No datable materials were encountered in sufficient quantities to allow a direct determination of the age of the sediments. However, a qualitative estimate of the age of the sediments can be determined by referencing to the fault investigation report for adjacent site (Cohrt et al, 2017). The subsurface materials encountered in the exploratory trenches can be roughly classified as follows.

Quaternary Younger Alluvium Materials (Qal and Qal2) range from 2,000 to 8,000 years old in Holocene age:

Qal-Silty Sand with some gravel; yellowish brown to olive brown, dry, silty fine to coarse grained sand, some angular gravels, abundant rootlets.

Qal2-Silty Sand; yellowish brown, dry, silty, mostly fine-medium grained sand, scatter angular gravels and some coarse-grained sand existing in some sections.

Quaternary Older Alluvium (Qoa, Qoa3, and Qoa7) range from 13,200 to 90,000 years old in Pleistocene age:

Qoa-Gravelly Sand; light yellowish brown to yellowish brown, dry, fine-to coarse-grained sand, angular to subangular granitic gravel.

Qoa3-Silty Sand; yellow to yellowish brown; dry, thinly inter-bedded/laminated, some scoured gravel.

Qoa7-Silty Sand; yellowish brown, silty fine-grained sand.

On August 1, 2024, Yucca Valley City consulting geologist Mr. Mathew Cohrt arrived the site to walk the trench along with HGI geologists and Allen Harker with Moore Twining Associates, Inc. for additional input and observations. We discussed our findings and further analyzed trench observations, as necessary.

After the exploratory trenches were walked through, the trenches were backfilled with soil cuttings. The backfilled soil cuttings were compacted to a minimum of

90 percent of the maximum dry density as determined by ASTM D1557 and tested by HGI technician. The test results are included in our compaction report in a separate report.

CONCLUSIONS AND RECOMMENDATIONS

Per our aerial photo review, no lineaments which can be interpreted as potential fault ruptures were observed in the immediate vicinity of the subject site. Two exploratory trenches were excavated to a depth of 12 feet and 218 feet in length, which meet the San Bernardino County fault study criteria. The relative soil ages indicate that Holocene and Pleistocene age earth materials were exposed within the exploratory fault trenches. Fault trenching results indicate that there is no fault trace encountered within the investigated area.

Based on our document research, field reconnaissance, review of aerial photographs, and lineament analysis, the potential for surface fault rupture on the investigated area is considered low. But any potential fault trace beyond the fault study area cannot be excluded. Therefore, the proposed building should be limited to the investigated area, which is shown on Plate No. 1.

LIMITATIONS

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our understanding of the project requirements based on an evaluation of subsurface earth material conditions encountered at the subsurface exploration locations and the assumption that earth material conditions do not deviate appreciably from those encountered. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of **HGI** so that we may make modifications, if necessary.

CHANGE IN SCOPE

HGI should be advised of any changes in the project scope of proposed site grading so that it may be determined if recommendations contained herein are valid. This should be verified in writing or modified by a written addendum.

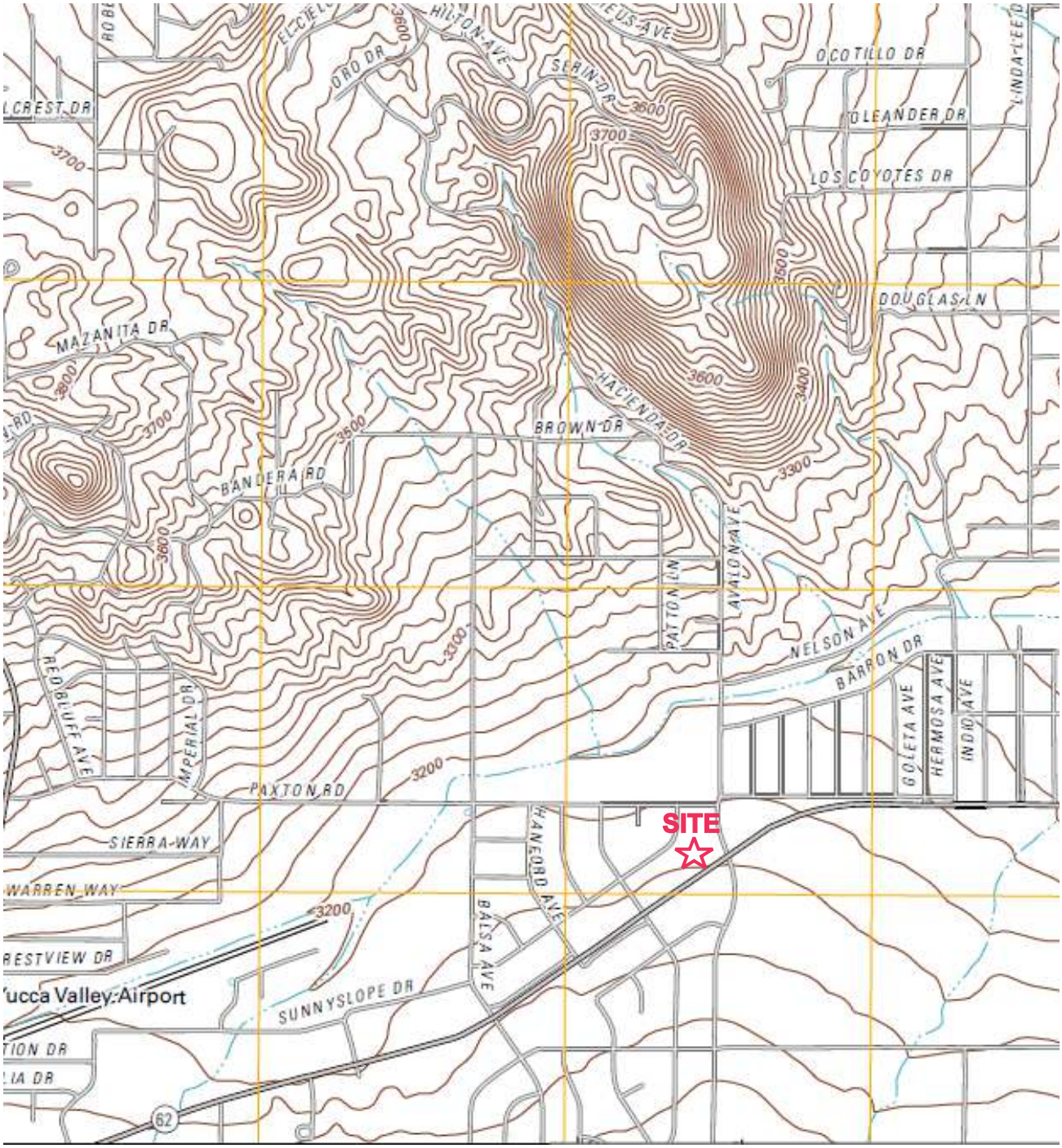
TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the State-of-the-Art and/or government codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two (2) years without a review by **HGI** verifying the validity of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with the standard of care and skill ordinarily exercised under similar circumstances by members of the geologic professions currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our surveys and exploratory excavations were made, and that our data, interpretations, and recommendations are based solely on information obtained by us. We will be responsible for those data, interpretations, and recommendations, but should not be responsible for interpretations by others of the information presented and/or developed. Our services consist of professional consultation and observation only, and other warranties, expressed or implied, are not made or intended in connection with work performed by **HGI** or by the proposal for consulting or other services or by the furnishing of oral or written reports or findings.

FIGURES & PLATES



SCALE 1:24 000

1 0.5 0 KILOMETERS 1 2

1000 500 0 METERS 1000 2000

N

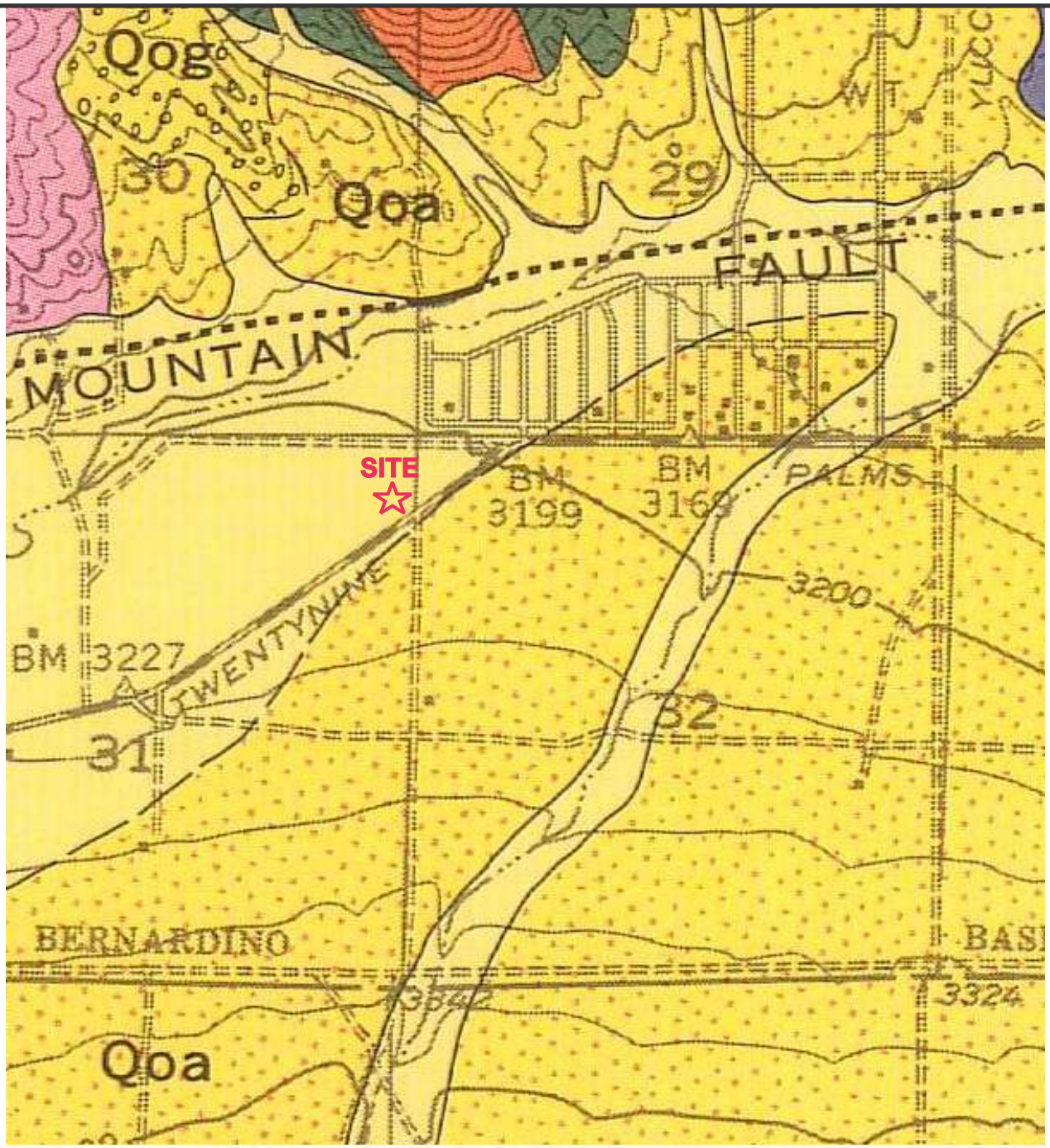
Source: Copied from USGS Topo Map-
Yucca Valley North Quadrangle 2012

HILLTOP GEOTECHNICAL
INCORPORATED

SITE LOCATION MAP

APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Figure 1**



1 ½ 0 1 MILE

Legend

Qa — Qa, *alluvium*

Qoa — Qoa, *older alluvium*

N

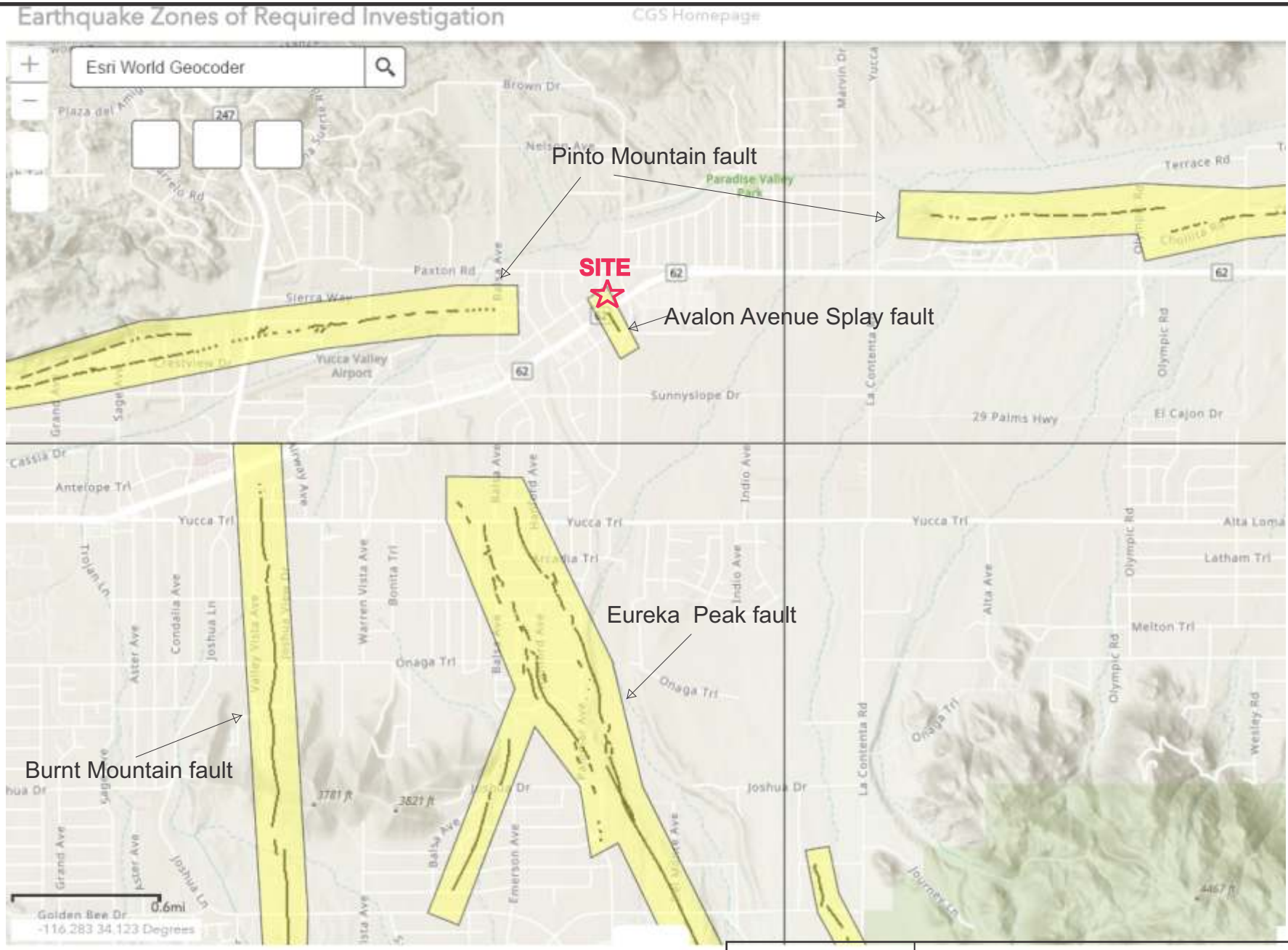
Source: excerpt from Geologic Map of Joshua Tree Quadrangle, San Bernadino and Riverside Counties, California by T. W. Dibblee, Jr. 1967

HILLTOP GEOTECHNICAL INCORPORATED

REGIONAL GEOLOGIC MAP

APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Figure 2**



Legend

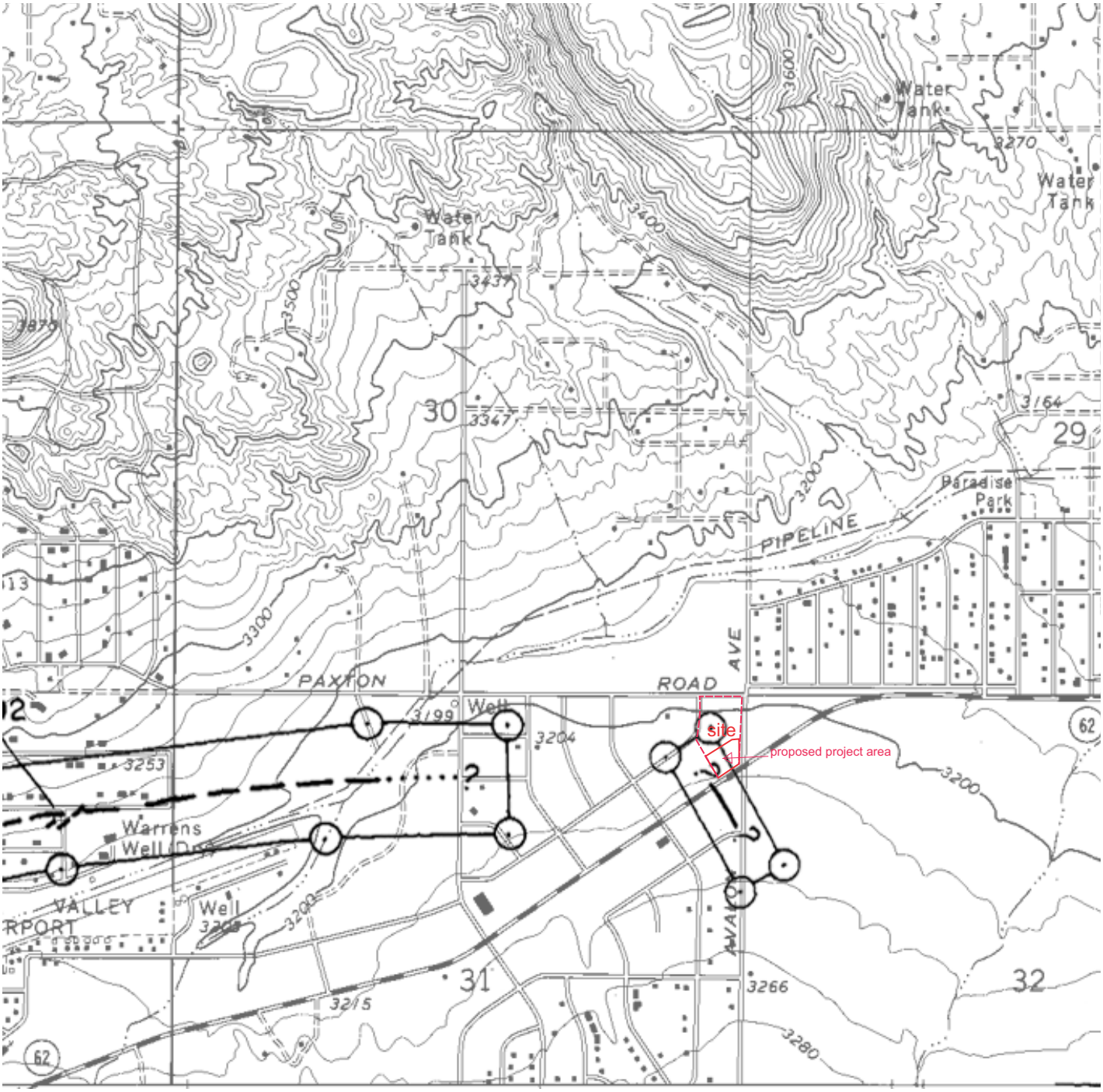
required fault study zone

HILLTOP GEOTECHNICAL INCORPORATED

Earthquake Zone of Required Investigation

APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Figure 3**



Source: excerpt for Special Studies Zones-Yucca Valley North Quadrangle-Revised Official Map Effective: July 1, 1993.

HILLTOP GEOTECHNICAL INCORPORATED

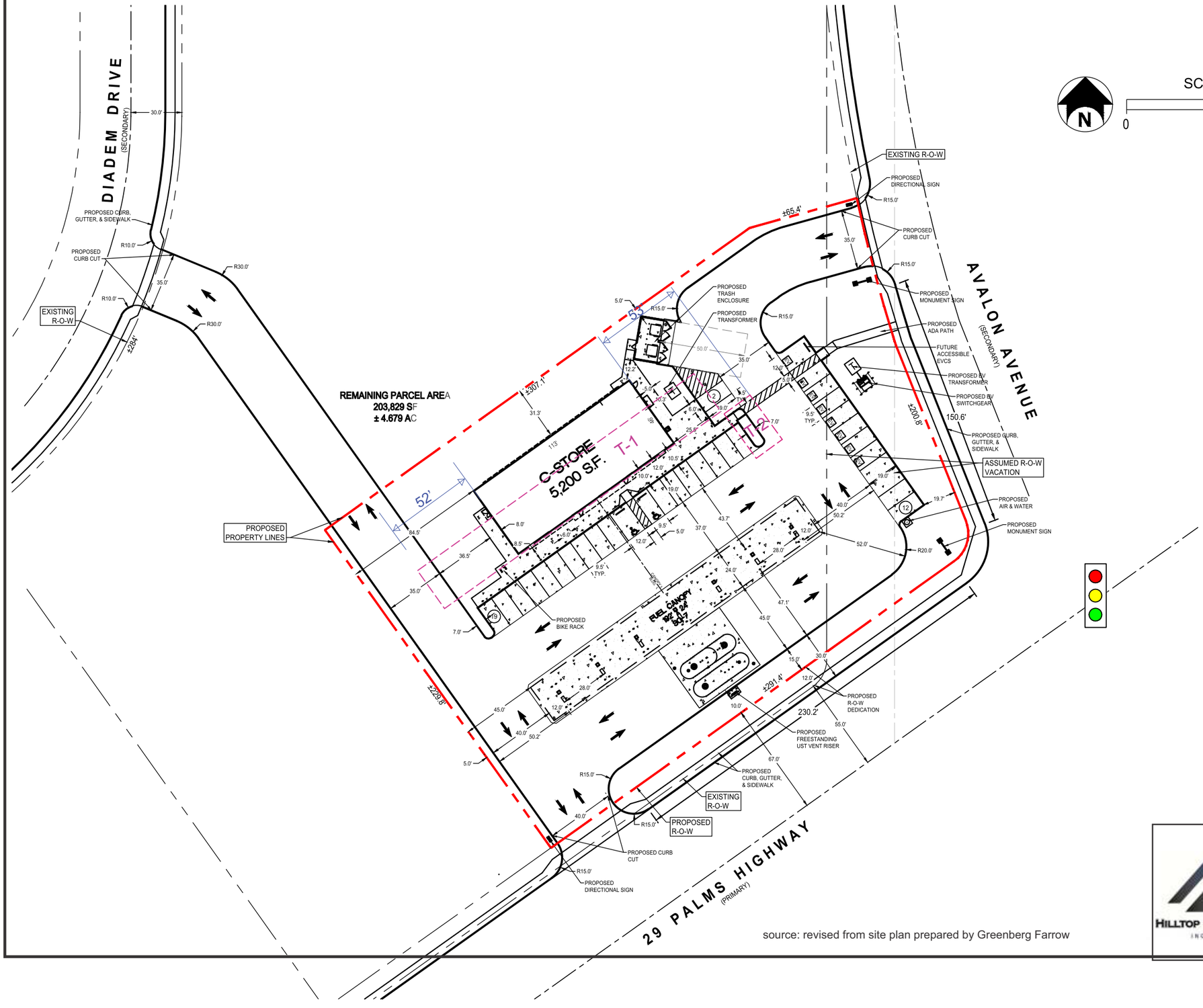
AP Fault Special Studies Zones

APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Figure 4**

CIRCLE K STORE
SITE PLAN

NWC 29 PALMS & AVALON AVENUE, YUCCA VALLEY, CA



source: revised from site plan prepared by Greenberg Farrow

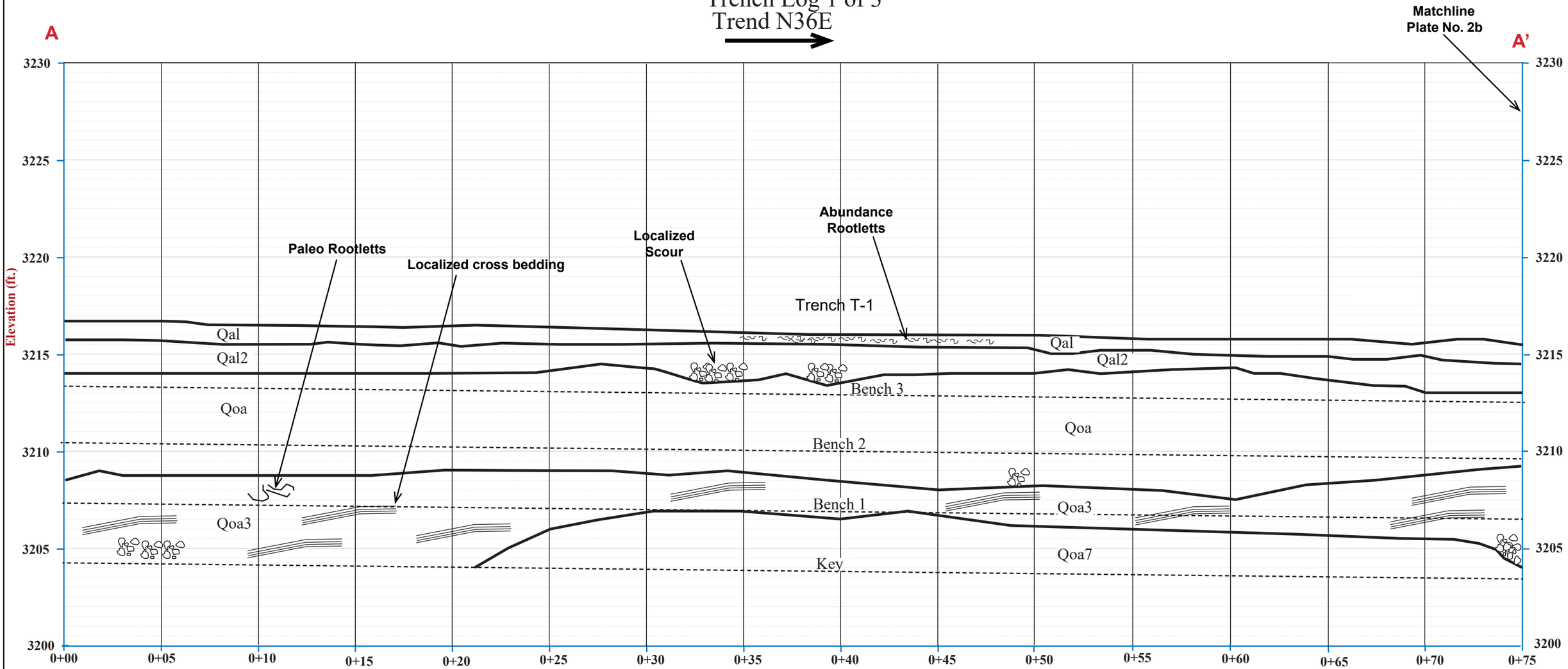
HILLTOP GEOTECHNICAL INCORPORATED

Site Plan and Exploratory Trench Locations

APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Plate No. 1**

Trench Log 1 of 3
Trend N36E



Legend

- Approximate Soil Stratigraphy Contacts
- Trench Benching

Qal	Young Alluvium	Qoa3	Old Alluvium Unit 3
Qal2	Young Alluvium Unit 2	Qoa7	Old Alluvium Unit 7
Qoa	Old Alluvium		

0 5 10’

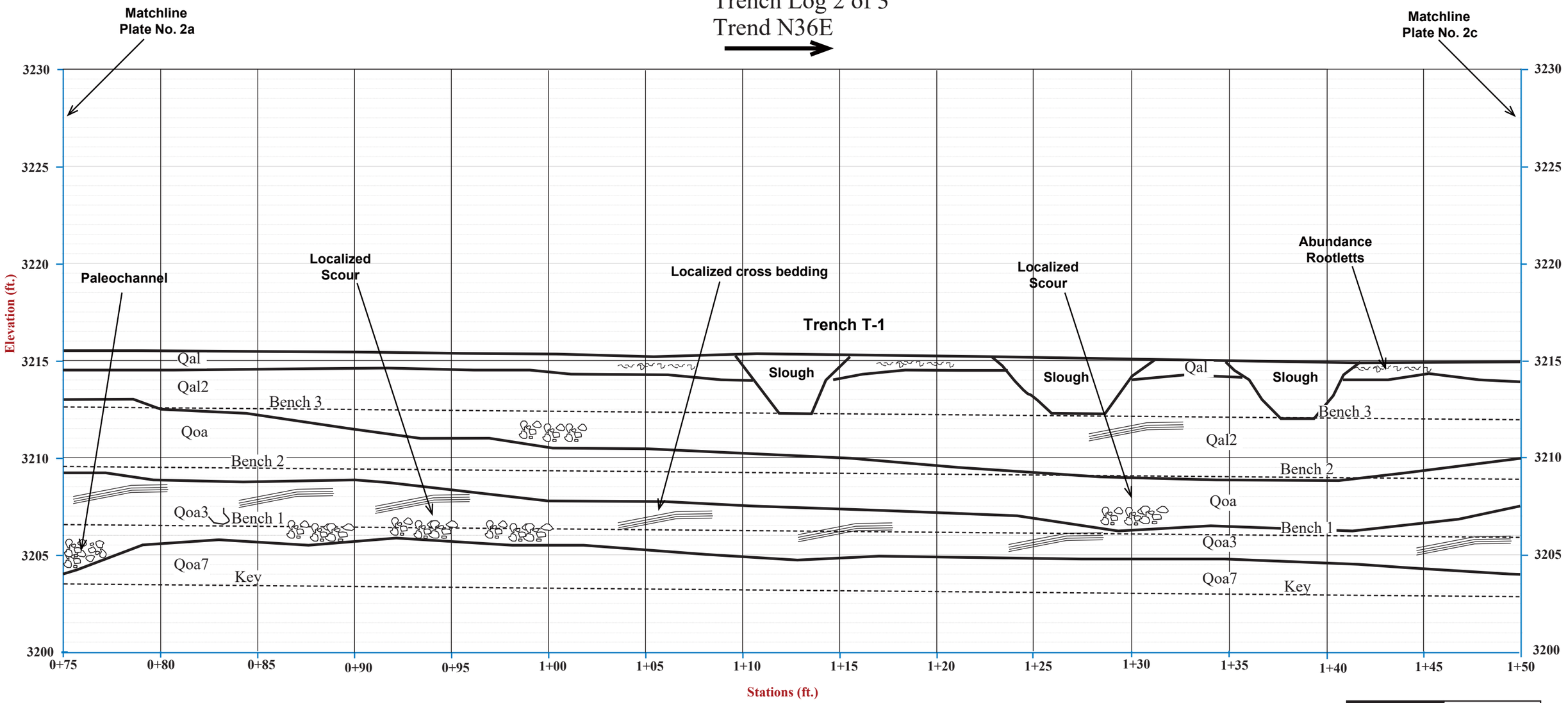
SCALE: 1”=5’
HORIZONTAL = VERTICAL

Note: Elevations are approximate and based on data obtained by plan sheets provided by client.
(for illustration purpose only).

HILLTOP GEOTECHNICAL INCORPORATED

Project Name: Avalon Avenue Yucca Valley Fault Study	**Project No.:** 1664-01	**Drawing Title:** Trench Log 1 of 3	**Plate No.:** 2a
	Date: August 2024		

Trench Log 2 of 3
Trend N36E



Legend

Approximate Soil Stratigraphy Contacts	Qal	Young Alluvium	Qoa3 Old Alluvium Unit 3
Trench Benching	Qal2	Young Alluvium Unit 2	Qoa7 Old Alluvium Unit 7
	Qoa	Old Alluvium	

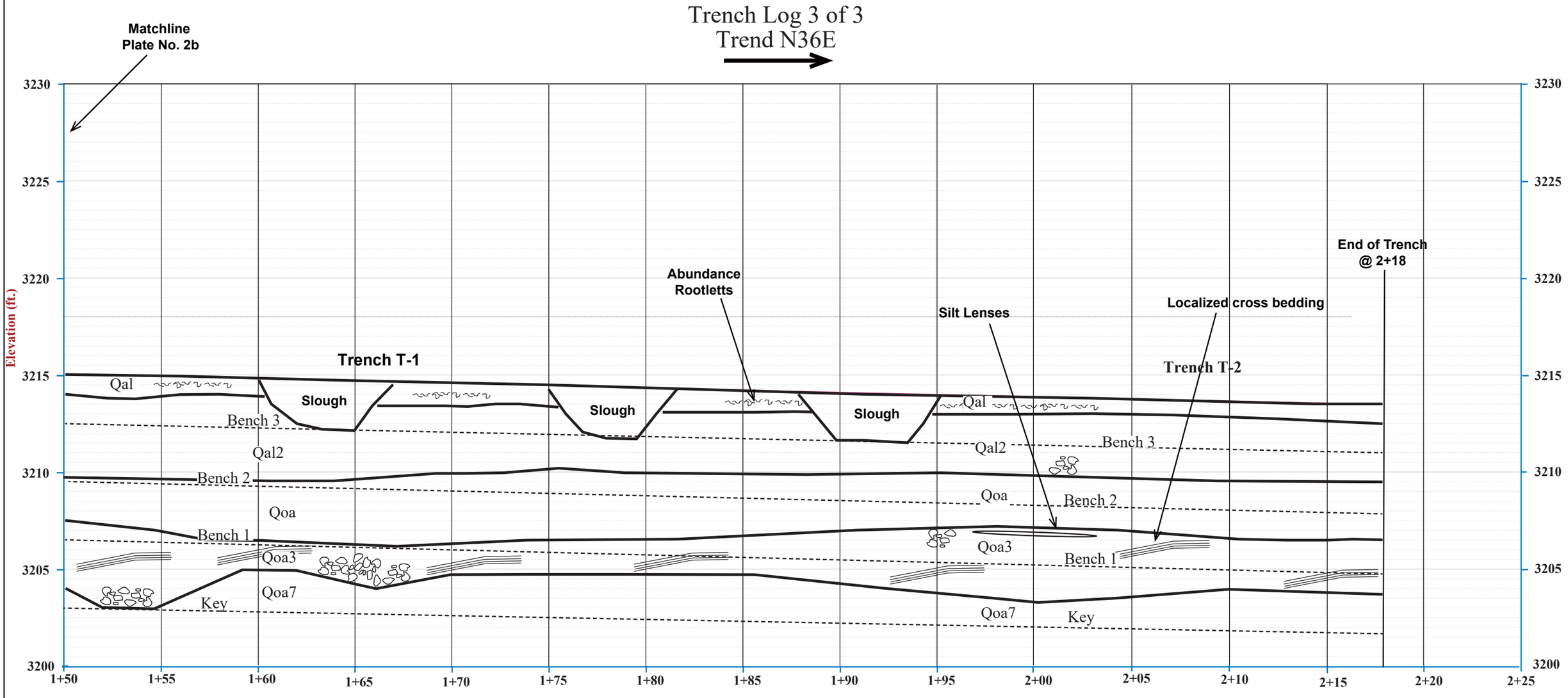
0 5 10’

SCALE: 1”=5’
HORIZONTAL = VERTICAL

Note: Elevations are approximate and based on data obtained by plan sheets provided by client.
(for illustration purpose only).

HILLTOP GEOTECHNICAL INCORPORATED	**Project Name:** Avalon Avenue Yucca Valley Fault Study	**Project No.:** 1664-01 **Date:** August 2024	**Drawing Title:** Trench Log 2 of 3	**Plate No.** 2b

Trench Log 3 of 3
Trend N36E



Legend

- Approximate Soil Stratigraphy Contacts
- Trench Benching

Qal	Young Alluvium	Qoa3	Old Alluvium Unit 3
Qal2	Young Alluvium Unit 2	Qoa7	Old Alluvium Unit 7
Qoa	Old Alluvium		

0 5 10’

SCALE: 1”=5’
HORIZONTAL = VERTICAL

Note: Elevations are approximate and based on data obtained by plan sheets provided by client.
(for illustration purpose only).

HILLTOP GEOTECHNICAL INCORPORATED

Project Name: Avalon Avenue Yucca Valley Fault Study	Project No.: 1664-01	Drawing Title: Trench Log 3 of 3	Plate No.: 2c
	Date: August 2024		

APPENDIX A TECHNICAL REFERNCENCES

Bader, J.S. and Moyle, W.R., Jr., 1960, *Data on water wells and springs in the Yucca Valley-Twentynine Palms Area, San Bernardino and Riverside Counties, California: California Department of Water Resources Bulletin 91-2.*

Bryant, W. A., 1992, *Fault Evaluation Report FER-234, Surface Rupture along the Johnson Valley, Homestead Valley, and Related Faults Associated with the Mw 7.5 28 June 1992 landers Earthquake; California Division of Mines and Geology. Scale 1:24,000.*

California Department of Water Resources, 2004, *California's Groundwater Bulletin 118, Colorado River Hydrologic Region, Warren Valley Groundwater Basin, Groundwater Basin Number 7-12.*

Cohrt, M. J. and **Anderson, B. L.**, January 20, 2017, *Fault Investigation, Proposed Retail Center, NWC Avalon Avenue and Palisade Drive, APN 601-201-41, Yucca Valley, California.*

Dibblee, T. W., 1967, *Geologic Map of the Joshua Tree quadrangle, San Bernardino and Riverside Counties, California: U. S. Geological Survey Miscellaneous Geologic Investigations Map I-516, Scale 1:62,500.*

Dibblee, T.W., Jr., 1968, *Evidence of major lateral displacement on the Pinto Mountain fault, southeastern California: Geological Society of America Special Paper 115, p. 322.*

GoogleEarth.com, 2024, *Historical Aerial Photograph for the Yucca Valley Area, Variable Scale. Reviewed at googleearth.com.*

Hauksson, E., Hutton, K., and Jones, L.M., 1992, *Preliminary report on the 1992 Landers earthquake sequence in southern California, in Landers earthquake of June 28, 1992, San Bernardino County, California, Field Trip Guidebook: Southern California Section of the Association of Engineering Geologists, p.23-31.*

Rasmussen, G. S., November 1, 1977, *Engineering geology investigation of Sunset Village, Joshua Tree, California, Project No. 1265.*

Rasmussen, G. S., October 2, 1990, *Subsurface engineering geology investigation of approximatey 120 acres, northwest of Gorgonio Drive and Canyon Road, Twentynine Palms, California, Project No. 2930.*

Rasmussen, G. S., 1992, *M7.5 Landers Earthquake, in Ebersold, D. B. (ed.), Landers Earthquake of June 28, 1992, San Bernardino County, California:*

Association of Engineering Geologists Annual Meeting, Field Trip Guidebook, p. 11-14.

Treiman, J.A., 1992, *Eureka Peak and Burnt Mountain faults - two "new" faults in Yucca Valley, San Bernardino County, California, in Landers earthquake of June 28, 1992, San Bernardino County, California, Field Trip Guidebook: Southern California Section of the Association of Engineering Geologists, p. 19-22.*

Treiman, J.A., 1992, *Fault Evaluation Report 230, Eureka Peak and Related Faults, Joshua Tree South and Yucca Valley South Quadrangles, San Bernardino and Riverside Counties, California Division of Mines and Geology, December 15, 1992.*

Treiman, J.A., 1993, *Fault Evaluation Report 230, Supplement No. 1, Eureka Peak and Related Faults, San Bernardino and Riverside Counties, California Division of Mines and Geology, May 20, 1993.*

APPENDIX F

COMPACTION TEST REPORT FOR BACKFILL OF FAULT TRENCH

HILLTOP GEOTECHNICAL
INCORPORATED

786 S. GIFFORD AVENUE • SAN BERNARDINO • CA 92408
Phone **909-890-9079** • FAX 909-890-9055
hilltopg@hgeotech.com

August 27, 2024

Moore Twining Associates, Inc.
2527 Fresno Street
Fresno, CA 93721

Project No.: 1664-01
Report No.: 2

Attention: Mr. Read Andersen

Subject: **Compaction Report for Backfilling the Exploratory Trench, APN: 0601-543-01, Avalon Avenue, Yucca Valley, California.**

Gentleman:

According to your request, **Hilltop Geotechnical, Inc.** (HGI) has completed a limited fault hazard evaluation for design and construction of the proposed convenience store and fuel canopies at the subject site. Per our signed agreement, the exploratory trenches for the fault study have been backfilled from 12 feet below the existing ground surface to the existing ground surface with compacted fill. Submitted herein are the summary of the field observations and the compaction test results along the laboratory data.

GRADING PROCEDURES

In general, the grading was performed as follows:

a) The bottoms of excavated trenches were scarified and compacted and approved by a representative of HGI.

b) A John Deere 85G with sheep foot roller, a John Deere 544 front loader, and water tank truck were used for soil moistening and compaction. Onsite soils

were used as compacted fill, which consists of yellowish brown to brown silty sand. All backfilling materials were spread in 6-to-8-inch lift, moistened to near the optimum moisture content, placed in layers not exceeding eight inches, compacted, and tested in intervals no more than two feet to the finished grades.

SPECIFICATION AND TEST RESULTS

On-site soil cuttings were utilized for backfilling the exploratory trenches. The specified minimum degree of compaction is 90 percent of the maximum dry density, as determined by the latest version of ASTM D1557. Field density tests were performed in accordance with the latest version of ASTM D 6938 (nuclear gauge). The test locations are shown in **Plate No. 1.** The test results are tabulated in **Plate Nos. 2** and **3**. Soils used in compacted fills are classified and tabulated in **Plate No. 4**.

Wherever field density tests failed, the soils were scarified, re-mixed to near optimum moisture and re-compacted to achieve the required density.

SCOPE OF WORK

TIME OF BACKFILLING

Backfilling operations were conducted from August 6 to August 23, 2024.

CONTRACTOR AND EQUIPMENT

The backfilling operations were performed by **Norfork, Inc.** A John Deere 85G with sheep foot roller, a John Deere 544 front loader, and water tank truck were used for soil moistening and compaction.

CONCLUSIONS

Based on the observations and tests performed during backfilling, the exploratory trenches at the subject site have been completed and meet the required minimum of 90 percent of the maximum dry density in accordance with the latest version of ASTM D1557.

CLOSURE

This report has been prepared for use by the parties or project named or described above. It may or may not contain sufficient information for other parties or purposes. The findings and conclusions expressed in this report are based on field and laboratory testing performed during the grading operation and on generally accepted engineering practices and principles. No further warranties are implied or expressed beyond the direct representations of this report.

Thank you for the opportunity to provide these services. If you should have any questions regarding this report, please do not hesitate to contact this office at your convenience.

Respectfully Submitted,
HILLTOP GEOTECHNICAL, INC.

S. Mack Chen, P.E. C76834
Principal Engineer

Distribution: Addressee
pdf Copy Via E-Mail:
Mr. Read Andersen: ReadA@mooretwining.com

CIRCLE K STORE
SITE PLAN
NWC 29 PALMS & AVALON AVENUE, YUCCA VALLEY, CA

REMAINING PARCEL AREA
203,829 SF
± 4.679 AC

C-STORE
5200 S.F.

FUEL CANOPY
82' 0" x 24'
1968 SQ.FT.

AVALON AVENUE
(SECONDARY)

29 PALMS HIGHWAY

EXISTING R-O-W

PROPOSED DIRECTIONAL SIGN

PROPOSED CURB CUT

PROPOSED MONUMENT SIGN

PROPOSED ADA PATH

FUTURE ACCESSIBLE EVCS

PROPOSED EV TRANSFORMER

PROPOSED EV SWITCHGEAR

PROPOSED GURB, GUTTER, & SIDEWALK

ASSUMED R-O-W VACATION

PROPOSED AIR & WATER

PROPOSED MONUMENT SIGN

PROPOSED TRASH ENCLOSURE

PROPOSED TRANSFORMER

PROPOSED BIKE RACK

PROPOSED FREESTANDING UST VENT RISER

PROPOSED R-O-W DEDICATION

PROPOSED CURB, GUTTER, & SIDEWALK

EXISTING R-O-W

PROPOSED R-O-W

PROPOSED CURB CUT

PROPOSED DIRECTIONAL SIGN

PROPOSED PROPERTY LINES

±65.4'
±307.1'
±229.8'
±200.8'
±291.4'
150.6'
230.2'
53'
52'
113'

Legend
- ●1 Test Location and Test No.
- T-1 Exploratory Trench Location

source: revised from site plan prepared by Greenberg Farrow

HILLTOP GEOTECHNICAL INCORPORATED

Site Plan and Exploratory Trench Locations
APN 0601-543-01, Avalon Avenue, Yucca Valley

By: MC	Date: 8/2024
Project No.: 1664-01.1	**Plate No. 1**

SUMMARY OF FIELD IN-PLACE DENSITY TEST RESULTS

MOORE TWINING: AVALON AVE. YUCCA VALLEY

TRENCH BACKFILL

TEST #	TEST DATE	TEST LOCATION	TEST ELEVATION	DRY DENSITY (P.C.F)	MOISTURE CONTENT (%)	SOIL TYPE	MAX DRY DENSITY (P.C.F)	RELATIVE COMP. (%)	SPEC REQ. (%)	TEST TYPE (N OR S)	TEST STATUS
1	08/06/24	Sta. 0+25	SG-11.5'	114.6	6.7	1	126.5	91	90	N	P
2	08/06/24	Sta. 0+50	SG-11.5'	115.3	7.2	1	126.5	91	90	N	P
3	08/07/24	Sta. 0+75	SG-11.5'	115.1	7.4	1	126.5	91	90	N	P
4	08/07/24	Sta. 0+100	SG-11.5'	114.8	6.9	1	126.5	91	90	N	P
5	08/08/24	Sta. 1+50	SG-11.5'	115.5	8.1	1	126.5	91	90	N	P
6	08/08/24	Sta. 2+00	SG-11.5'	116.2	9.3	1	126.5	92	90	N	P
7	08/09/24	Sta. 0+25	SG-11.0	117.3	6.2	1	126.5	93	90	N	P
8	08/09/24	Sta. 1+00	SG-11.0	118.3	5.7	1	126.5	94	90	N	P
9	08/09/24	Sta. 1+75	SG-11.0	117.7	6.1	1	126.5	93	90	N	P
10	08/09/24	Sta. 2+00	SG-10.0	117.3	7.9	1	126.5	93	90	N	P
11	08/09/24	Sta. 1+95	SG-10.0	120.8	8.5	1	126.5	95	90	N	P
12	08/09/24	Sta. 1+60	SG-10.0	118.4	8.1	1	126.5	94	90	N	P
13	08/12/24	Front Trench	SG-10.0	117.3	7.3	1	126.5	93	90	N	P
14	08/12/24	Front Trench	SG-8.0	119.1	9.1	1	126.5	94	90	N	P
15	08/13/24	Front Trench	SG-6.0	119.4	7.4	1	126.5	94	90	N	P
16	08/13/24	Front Trench	SG-4.0	121.6	7.7	1	126.5	96	90	N	P
17	08/13/24	Front Trench	SG-.2.0	123.1	7.9	1	126.5	97	90	N	P
18	08/13/24	Front Trench	SG	122.7	8.1	1	126.5	97	90	N	P
19	08/14/24	Sta. 0+00	SG-8	122.1	8.1	1	126.5	97	90	N	P
20	08/14/24	Sta. 1+00	SG-8	121.7	7.9	1	126.5	96	90	N	P

SUMMARY OF FIELD IN-PLACE DENSITY TEST RESULTS
MOORE TWINING: AVALON AVE. YUCCA VALLEY
TRENCH BACKFILL

TEST #	TEST DATE	TEST LOCATION	TEST ELEVATION	DRY DENSITY (P.C.F)	MOISTURE CONTENT (%)	SOIL TYPE	MAX DRY DENSITY (P.C.F)	RELATIVE COMP. (%)	SPEC REQ. (%)	TEST TYPE (N OR S)	TEST STATUS
21	08/14/24	Sta. 2+00	SG-8	122.9	8.3	1	126.5	97	90	N	P
22	08/15/24	Sta. 0+40	SG-6	113.5	6.6	1	126.5	90	90	N	P
23	08/15/24	Sta. 1+00	SG-6	114.9	7.8	1	126.5	91	90	N	P
24	08/15/24	Sta. 2+00	SG-6	114.1	7.0	1	126.5	90	90	N	P
25	08/16/24	Sta. 0+50	SG-4	119.7	6.7	1	126.5	95	90	N	P
26	08/16/24	Sta. 1+00	SG-4	121.8	5.7	1	126.5	96	90	N	P
27	08/19/24	Sta. 0+50	SG-2	123.3	7.7	1	126.5	97	90	N	P
28	08/19/24	Sta. 0+85	SG-2	123.7	5.1	1	126.5	98	90	N	P
29	08/20/24	Sta. 0+50	SG-1	123.6	7.1	1	126.5	98	90	N	P
30	08/20/24	Sta. 1+00	SG-1	124.1	6.9	1	126.5	98	90	N	P
31	08/20/24	Sta. 1+60	SG-2	120.9	8.0	1	126.5	96	90	N	P
32	08/21/24	Sta. 1+60	SG-1	122.2	8.8	1	126.5	97	90	N	P
33	08/21/24	Sta. 0+90	SG-1	119.9	8.8	1	126.5	95	90	N	P
34	08/22/24	Sta. 1+93	SG-1	120.0	6.2	1	126.5	95	90	N	P
35	08/23/24	Sta 01+60	SG	120.1	7.0	1	126.5	95	90	N	P
36	08/23/24	Sta 01+80	SG	120.8	7.2	1	126.5	95	90	N	P
37	08/23/24	Sta 0+90	SG	118.9	6.9	1	126.5	94	90	N	P

S.G. - Surface ground elevation.
N - Nuclear gauge test.

SUMMARY OF LABORATORY TESTING
APN: 0601-543-01, AVALON AVE,
YUCCA VALLEY, CA

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE CONTENT RELATIONSHIP TEST RESULTS (ASTM D1557 Test Method)

SAMPLE	EARTH MATERIAL DESCRIPTION	PROCEDURE	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
1 Sampled 7-24-24	Silty, Fine to Coarse Sand, Tr. Gvl.-1", Brown	B	126.5	7.5