APPENDIX E

Initial Geotechnical Engineering Inspection Report

INITIAL GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

PROPOSED RETAIL TENANT STORE #2077-06

SOUTHWEST CORNER OF CENTRAL AVENUE (ROUTE 74)

AND CAMBERN AVENUE

LAKE ELSINORE, CALIFORNIA

Project Number: D82109.01

For:

Greenberg Farrow
1920 Main Street, Suite 1150
Irvine, California 92614

July 8, 2011

MOORE TWINING ASSOCIATES, INC.

July 8, 2011 D82109.01-01

Greenberg Farrow
1920 Main Street, Suite 1150
Irvine, California 92614

Attention: Mr. Farman Shir

Subject: **Initial Geotechnical Engineering Investigation**
Proposed Retail Tenant Store 2077-06
Southwest Corner of Central Avenue (State Route 74) and Cambern Avenue
Lake Elsinore, California

Dear Mr. Shir:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed Retail Tenant store to be located at the southwest corner of Central Avenue (State Route 74) and Cambern Avenue, Lake Elsinore, 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.

We appreciate the opportunity to be of service to Greenberg Farrow. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience at 800-268-7021.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

Dean B. Ledgerwood II, PG
Project Geologist
Geotechnical Engineering Division

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for the proposed Retail Tenant Store #2077-66 to be located at the southwest corner of Central Avenue (Route 74) and Cambern Avenue in Lake Elsinore, California.

It is our understanding that the proposed development will include construction of a new Retail Tenant store comprising approximately 155,410 square feet in plan view dimension. It is anticipated that the proposed construction will consist of concrete masonry unit (CMU) bearing walls, interior steel columns and a steel frame roof structure with a concrete slab-on-grade floor.

The proposed development will include asphaltic concrete parking and drive aisles, Portland cement concrete pavements, underground utilities, and isolated landscape areas. Also, it is our understanding that a 96 inch diameter stormwater culvert is proposed east of the proposed Retail Tenant Store, extending south along the eastern site boundary to be tied into the proposed stormwater detention basin to be located near the southern property line.

At the time of our field exploration, the subject site comprised a combination of vacant field and developed property. In general, the north to northwestern two-thirds of the property is vacant, undeveloped land and is covered by grasses, weeds, brush and some barren areas with exposed gravelly soils.

The south to southeastern one-third of the property were occupied by four (4) residential properties and vacant land covered by eucalyptus trees. The four (4) residential properties have access from the northwest side of Third Street. Four (4) single story, single family homes were noted throughout the existing residential development. In addition, several old trailers, trash debris, old tires, fencing debris were noted throughout the residential areas. The remainder of the residential properties are covered by gravelly soils, weeds, and mature tall eucalyptus trees.

Based on our review of the preliminary site topographic map/grading plan provided by Greenberg Farrow, the site gently slopes descending from north to the south, with site elevations ranging from approximately 1,317 feet above mean sea level (AMSL) in the northern portion of the site to about 1,298 feet AMSL in the southern portion.

Based on an approximate finish floor elevation of 1,306 feet AMSL and our review of the topographic survey plan provided, it is anticipated that cuts up to 3 feet and fills up to 7 feet would be required within the proposed building pad to achieve final pad grade elevations.

From June 27, 2007 through June 30, 2011, a total of sixty six (66) soil borings were drilled throughout the proposed building pad, parking, and outlot pad areas. The test borings were drilled under the direction of a Moore Twining Professional Geologist using a truck mounted, CME-75 drill rig equipped with 6 5/8 outside diameter hollow stem augers. Twelve (12) of the sixty six (66) test borings were drilled throughout the building pad area to depths ranging from about 20 feet to 50 feet below existing site grade elevations (a minimum of 20 feet below final anticipated pad grade). The remaining fifty four (54) test borings were drilled at an approximate 100 foot grid spacing, throughout the pavement areas, outlot pad area, and storm water detention basin areas to the depths ranging from 10 feet to 15 feet BSG (a minimum of 10 feet below final anticipated site grades).

EXECUTIVE SUMMARY (Continued)

The near surface soils encountered generally consisted of near surface silty sands from the surface to depths ranging from 1 foot to about 8½ feet BSG. Below the near surface sands, interbedded layers of sandy silty clays, lean clays, clayey sands, sandy silts, poorly graded sands with silt, and silty sands were encountered to the depth of approximately 45 feet BSG. At the depth of 45 feet BSG, Claystone Rock material was encountered to the depth of 50 feet BSG. Below the Claystone Rock material, Sandstone Rock material was encountered to the maximum depth explored of 51½ feet BSG. It should be noted that poorly graded gravels were encountered in two (2) test borings, B-7 and B-14 at the depths of 13½ and 18½ feet BSG. In addition, the native lean clays encountered within the upper 5 feet BSG were generally described as "slightly cemented" hardpan soils.

In addition, three (3) of the test borings conducted within the proposed building pad encountered undocumented fill soils from the surface to depths ranging from 1 to 3½ feet BSG. Also, two (2) test borings conducted outside of the limits of the proposed building pad encountered undocumented fills from the surface to depths of 2½ and 8½ feet BSG. The undocumented fills generally consisted of silty sand soils with trash and concrete debris.

At the time of our field exploration, groundwater was encountered in eight (8) of the sixty six (66) test borings at depths ranging from 13⅓ to 25 feet BSG, drilled during our June 2011 field exploration. Test borings B-16 was left open for a period of 24 hours. After a period of 24 hours, groundwater was measured at a depth of 23½ feet BSG in test boring B-16.

In order to reduce the potential for excessive differential static settlement of foundations, it is recommended foundations be supported on engineered fill established by over-excavation and compaction. Foundations supported on engineered fill as recommended in this report would reduce estimated static total and differential static settlements to meet the Retail Tenant requirements.

The near surface soils tested exhibited low to high compressibility, low plasticity and excellent pavement support characteristics.

The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the Elsinore (Glen Ivy) Fault, located about 3.2 miles (5.1 kilometers) west of the site. Therefore, the potential for fault rupture at the site is considered low.

Based on the boring data, total seismic settlements are estimated to range from about 1.5 inches to 2.5 inches based on the design earthquake. Thus, a differential seismic settlement of up to 1.25 inches in 40 feet was estimated.

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

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

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INITIAL GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

PROPOSED RETAIL TENANT STORE #2077-06

SOUTHWEST CORNER OF CENTRAL AVENUE (ROUTE 74)

AND CAMBERN AVENUE

LAKE ELSINORE, CALIFORNIA

Project Number: D82109.01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed Retail Tenant Store to be located at the southwest corner of Central Avenue (Route 74) and Cambern Avenue in Lake Elsinore, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Greenberg Farrow 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); Photographic Log (Appendix D), the Geotechnical Fact Sheet and Foundation Subsurface Preparation Note (Appendix E), Dewatering Specifications (Appendix F), and Pavement Design Calculations (Appendix G).

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

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of the investigation was to conduct field exploration and laboratory testing programs, 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 slabs-on-grade;

2.1.2 Recommendations for the design and construction of asphaltic concrete and Portland cement concrete pavements;

2.1.3 Recommendations for 2010 California Building Code (CBC) seismic design parameters;

2.1.4 Evaluation of the potential for liquefaction and seismic settlement;

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

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

2.1.7 Recommendations for temporary excavations and trench backfill; and

2.1.8 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed improvements referenced in the Anticipated Construction section of this report. The recommendations provided herein are not intended for use for off-site improvements.

This investigation did not include a geologic/seismic hazards evaluation, flood plain investigation, environmental investigation, environmental audit, nor in-place density tests.

2.2 **Scope:** Our proposal, dated December 6, 2010 and contract amendment dated June 17, 2011, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A site plan, undated, untitled, provided by Greenberg Farrow, was reviewed and is referred to in this report as the site plan.

2.2.2 A preliminary topographic survey plan, undated, provided by Greenberg Farrow, was reviewed.

2.2.3 The Retail Tenant Geotechnical Investigation Specifications and Report Requirements, dated February 7, 2011, was reviewed.

2.2.4 A visual site reconnaissance and subsurface exploration were conducted.

2.2.5 A recent online aerial photograph of the site was reviewed.

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

2.2.7 Mr. Farman Shir (Greenberg Farrow) were consulted during the investigation.

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

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

3.0 BACKGROUND INFORMATION

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

3.1 **Site History:** Based on review of available on-line aerial photographs, the subject site appears generally unchanged from 1994 to present day. However, based on our review of a November 2009 aerial photograph, numerous debris/soil piles were noted near the northeast corner of the site. The approximate location of the former debris piles are depicted on Drawing No. 2 in Appendix A of this report.

No information relative to the Site History had been provided to Moore Twining at the time of this investigation.

3.2 **Previous Studies:** At the time of this report, it is our understanding that the Draft Phase I report is not available for review. When available, Moore Twining should be provided a copy for consideration of the final geotechnical report.

No previous engineering, geological, or environmental studies conducted for this site were not provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

3.3 **Site Description:** The project site comprises approximately 16.85 acres and is located at the southwest corner of Central Avenue (State Route 74) and Cambern Avenue in Lake Elsinore, California (see Site Location Map, Drawing No. 1, Appendix A). The subject site is bound to the northwest by Central Avenue; to the northeast by Cambern Avenue; to the southeast by Third Street;

and to the southwest by a combination of existing single family residential and commercial properties and vacant land with Dexter Avenue beyond. It should be noted that Cambern Avenue is paved with asphaltic concrete from the intersection of Central Avenue southeast for approximately 275 feet, beyond approximately 275 feet southeast of Central Avenue, Cambern continues as an unpaved, dirt road. Also, Third Street was noted to be an unpaved, dirt road.

At the time of our field exploration, the subject site comprised a combination of a vacant field and developed property. In general, the northwestern, approximately two-thirds of the property is vacant, undeveloped land covered by grasses, weeds, brush and some barren areas with exposed gravelly soils. Two unpaved, graded dirt access roads cross the northwestern two-thirds of the property in a general east-west and a northeast-southwest orientation. A two-foot high soil and rock berm was noted from the intersection of Central Avenue and Cambern, extending southeast adjacent to Cambern Avenue for approximately 800 feet. In addition, an approximate 320 foot long, 1-foot high soil berm was noted along the undeveloped portion of the southwestern property boundary. Also, a small rock pile with boulders embedded in the ground surface were noted near the northeast corner of the site. It appears that these may be associated with the former debris piles noted in our review of available aerial photographs. The approximate location of the existing soil berms and former debris piles are depicted on Drawing No. 2 in Appendix A of this report.

The southeastern, approximately one-third of the property was occupied by four (4) residential properties and vacant land with numerous mature eucalyptus trees. A total of four (4) single story, single family homes were noted throughout the existing residential development. Based on our discussions with the property owners, the existing residential properties include on-site septic systems. An existing swimming pool with a maximum depth of about 8 feet was noted within the residential property west of the intersection of Cambern Avenue and Third Avenue. Numerous cars, semi-trucks, and equipment such as fork lifts and a skip loader, etc., were noted throughout the existing residential properties. In addition, several old trailers, trash, debris, old tires, and fencing materials were noted throughout the residential areas. The remainder of the residential properties are covered by gravelly soils, weeds, and mature, tall eucalyptus trees with heights estimated to be up to 100 feet. Also, based on our site observations the single family residence appear to be serviced by underground water and gas utilities and over-head electric and telephone utilities.

Underground water and gas utilities were noted along Cambern Avenue and Central Avenues. In addition, overhead power lines were noted along Cambern Avenue and Third Street. Existing fire hydrants were noted at the intersection of Crane Street with the southwestern property boundary and along Cambern Avenue.

Two (2) drainage features were noted within the limits of the subject site. An erosional drainage feature located near the central portion of the subject site in a north-south orientation was observed to be dry and appeared to extend to a depth of about 18 inches. Also, a meandering erosional drainage feature is located near the southern portion of the subject site, in a northeast-southwest orientation starting from Cambern Avenue through the center of the proposed Retail Tenant building pad area. The southern most erosional feature was noted to be dry and appeared to extend to about 24 inches in depth. The approximate location of the drainage features noted are depicted on Drawing No. 2 in Appendix A of this report.

Based on our review of the preliminary site topographic map/grading plan provided by Greenberg Farrow, the site gently slopes descending from north to the south, with site elevations ranging from approximately 1,317 feet above mean sea level (AMSL) in the northern portion of the site to about 1,298 feet AMSL in the southern portion.

A photographic log of the observed conditions, debris and drainage features, etc. observed at the time of this investigation are located in Appendix D of this report.

3.4 **Anticipated Construction:** It is our understanding that the proposed development will include construction of a new Retail Tenant store comprising approximately 155,410 square feet in plan view dimension. It is anticipated that the proposed construction will consist of concrete masonry unit (CMU) bearing walls, interior steel columns and a steel frame roof structure with a concrete slab-on-grade floor. Basements are not anticipated as part of the proposed construction, however, a depressed loading dock is anticipated.

The proposed development will include asphaltic concrete parking and drive aisles, Portland cement concrete pavements, underground utilities, and isolated landscape areas. In addition, it is our understanding that three (3) proposed storm water detention basins are planned to be constructed along the southern site boundary and within outlot pad 1 and outlot pad 2. Also, it is our understanding that a 96 inch diameter stormwater culvert is proposed east of the Retail Tenant Store, extending south along the eastern site boundary which will drain to a proposed stormwater detention basin to be located near the southern property line.

Also, based on our review of the site plan provided, it is our understanding that a 1.15 acre Outlot Pad and a 1.67 Outlot Pad for future development has been designated along the northern site boundary.

Based on our review of the Retail Tenant Geotechnical Investigation Specifications and Report Requirements (February 7, 2011), it is our understanding that the proposed store additions will have maximum foundation loads for column foundations of 50 kips for exterior columns and 85 kips for interior columns. Maximum wall loads range from 4.0 to 6.0 kips per linear foot and the estimated maximum uniform floor slab live load is 125 pounds per square foot.

According to the Retail Tenant geotechnical report requirements, the maximum allowable total movements are ¾ inch and maximum allowable differential settlements are ½ inch (0.53 inches) in 40 feet horizontal, for wall foundations. The maximum allowable differential settlement for interior slabs or interior isolated footings is listed as L/500, where L is the horizontal distance in feet between two points, such as 0.96 inches in 40 feet. An allowable potential vertical rise of ¾ of an inch is also specified. For the purpose of this report, seismic settlements are considered separately in addition to the above settlements.

At the time of preparation of this report, a grading plan was not available. It is our understanding the finished floor elevation of the structure is anticipated to be about 1,306 feet AMSL. Based on an approximate finish floor elevation of 1,306 feet AMSL and our review of the preliminary topographic survey plan provided, it is anticipated that cuts up to 3 feet and fills up to 8 feet would be required within the proposed building pad to achieve final pad grade elevations. In addition, it is anticipated that cuts and fills of approximately 3 feet and 1 feet, respectively, are required in the proposed parking areas to achieve final 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 site reconnaissance, drilling test borings, soil sampling and standard penetration tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by a Moore Twining Professional Geologist on June 27, 2011. The features noted are described in the background information section of this report.

4.1.2 Drilling Test Borings: The depths and locations of the test borings were selected based on the size of the proposed building, type of construction, anticipated grading, and the subsurface soil conditions encountered with respect to Retail Tenant's "Geotechnical Investigation Specifications and Report Requirements," dated February 7, 2011. Based on the subsurface conditions and our familiarity with the subsurface soils in the Lake Elsinore area, a 100-foot deep boring was not required to assess the subsurface conditions and develop recommendations for the proposed improvements.

Prior to drilling the test borings at the site, the area of the field exploration was marked for Underground Service Alert. In addition, a private utility locator was contracted to determine the location of any underground utilities in the vicinity of the existing residences.

From June 27, 2007 through June 30, 2011, a total of sixty-six (66) soil borings were drilled throughout the proposed building pad, parking, stormwater basin and outlot pad areas. The test borings were drilled under the direction of a Moore Twining Professional Geologist using a truck mounted, CME-75 drill rig equipped with 6 5/8 outside diameter hollow stem augers. Twelve (12) of the sixty-six (66) test borings were drilled throughout the building pad area to depths ranging from about 20 feet to 50 feet below existing site grade elevations (a minimum of 20 feet below final anticipated pad grade). The remaining fifty-four (54) test borings were drilled at an approximate 100 foot grid spacing, throughout the pavement areas, outlot pad area, and storm water detention basin areas to the depths ranging from 10 feet to 15 feet BSG (a minimum of 10 feet below final anticipated site grades).

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

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of the borings. One (1) test boring (B-16) was left open for a minimum period of 24 hours to record static groundwater elevations.

The test boring locations were determined based on existing site features shown on the Site Plan provided. The test boring locations are shown on Drawing No. 2 in Appendix A. The coordinates for the test borings are provided in a table located in Appendix B of this report. The elevations of the test borings were estimated based on the elevations provided on the referenced topographic plan provided. The test borings were backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated at the bore hole locations.

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

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

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in closefitting, 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.

Sampling using thin wall Shelby tube samplers was not conducted due to the relatively low plasticity of the clay soils encountered, the presence of gravels and the stiff nature of the materials.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and relatively undisturbed samples representative of the subsurface soils. At the completion of the testing, the soil samples were retained and will be stored of a minimum of 6 months.

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

4.3 Field Hydraulic Conductivity Testing: To estimate the hydraulic conductivity (permeability) of the near surface soils, three (3) Guelph Permeameter tests were conducted in the native soils at depths of approximately 3 feet BSG. The testing was conducted on June 29, 2011. The field data are presented in Table 1 in section 6.2, "Permeability Characteristics of In-Place Soils" of this report.

The testing was performed using a Guelph Permeameter manufactured by SoilMoisture Equipment Corporation as described in ASTM test method D5126, "Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone." In general, the tests consisted of excavating three (3) shallow hand auger borings within the area of the proposed detention basins to depths of approximately 3 feet below site grades. The apparatus consisted of a Guelph Permeameter device. The purpose of the Guelph Permeameter was to estimate the in place hydraulic conductivity of the near surface soils under constant head conditions. The locations of the tests are identified on Drawing No. 2 in Appendix A.

5.0 FINDINGS AND RESULTS

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

5.1 Surface Conditions: At the time of the June 2011 field exploration, the northern two-thirds of the subject site was vacant undeveloped property. As discussed in the Site Description section of this report, an erosional drainage feature was noted in the central portion of the site. In addition, near the northeast corner of the site, scattered rock fragments were noted at the surface and boulders were noted to be embedded in the ground surface. Based on our review of available aerial photographs, numerous small debris/soil piles were noted near the northeast corner of the site. It is anticipated that the rock fragments and boulders observed are related to the previous fill piles. Therefore, it is anticipated that shallow fills extending below the ground surface may be present within these areas. The surface of the undeveloped portion of the site was noted to generally be covered with dry native grasses.

At the time of this investigation, the southern two-thirds of the site was a developed area with approximately four (4) residential properties. The residential properties included single story structures, animal pens, hardscaping, an in-ground swimming pool, underground and over-head utilities, scattered trash, and numerous cars, trucks, trailers, etc. In addition, the developed residential portion of the site was occupied by tall, mature trees. Also, a drainage erosional feature was noted within the southern portion of the site.

5.2 **Soil Profile:** The near surface soils generally consisted of silty sands from the surface to depths ranging from 1 foot to about 8½ feet BSG. Below the near surface sands, interbedded layers of sandy silty clays, lean clays, sandy lean clays, clayey sands, sandy silts, poorly graded sands with silt, and silty sands were encountered to a depth of approximately 45 feet BSG. These soils were underlain by sedimentary rock which was encountered to the maximum depth explored of 51½ feet BSG. It should be noted that poorly graded gravels were encountered in two (2) test borings, B-7 and B-14 at the depths of 13½ and 18½ feet BSG. In addition, the native lean clays and sandy lean clays, where encountered, within the upper 5 feet BSG were generally described as "slightly cemented" hardpan soils.

In addition, three (3) of the test borings conducted within the proposed building pad encountered undocumented fill soils from the surface to depths ranging from 1 to 3½ feet BSG. Also, two (2) test borings conducted outside of the limits of the proposed building pad encountered undocumented fills from the surface to depths of 2½ and 8½ feet BSG. The undocumented fills encountered generally consisted of silty sand soils with trash and concrete debris.

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

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

Undocumented Silty Sand Fills: The undocumented silty sand fill soils encountered were described as very loose to medium dense, as determined by standard penetration resistance, N-values, ranging from 1 to 20 blows per foot. The silty sand fills tested had a moisture content of 2 percent.

Native Silty Sands: These soils were described as very loose to dense, as determined by standard penetration resistance, N-values, ranging from 2 to 38 blows per foot. Two (2) relatively undisturbed samples indicated these soils tested have an in-place dry density of 114.3 and 116.2 pounds per cubic foot. These soils tested had a moisture content ranging from 2 to 11 percent.

Sandy Silty Clay: These soils encountered were described as very soft to very stiff, as determined by standard penetration resistance, N-values, ranging from 2 to 20 blows per foot. Two (2) relatively undisturbed samples indicated in-place dry densities of 107.2 and 121 pounds per cubic foot. The moisture content of these soil samples tested were 8 and 10 percent. A direct shear test conducted on these soils resulted in an internal angle of friction of 32 degrees, with 110 pounds per square foot of cohesion. Two (2) consolidation tests indicated that these soils had moderate to high compressibility characteristics (about 8.5 and 16 percent consolidation under a load of 16 kips per square foot). Upon inundation (wetting), the samples exhibited slight collapse potential (0.1 and 1.6 percent under a load of 0.5 kips per square foot). Atterberg tests conducted on three (3) samples indicated plasticity indexes of 5, 6, and 7 with liquid limits of 22, 22, and 24, respectively.

Sandy Lean Clays/Lean Clays: These soils were described as soft to hard, as determined by standard penetration resistance, N-values, ranging from 3 to 57 blows per foot. Five (5) relatively undisturbed samples indicated in-place dry densities ranging from 111.7 to 130.4 pounds per cubic foot. The moisture content of these soils tested ranged from 5 to 15 percent. Two (2) consolidation tests indicated low to high compressibility characteristics (about 4.5 and 18.4 percent consolidation under a load of 16 kips per square foot). Upon inundation (wetting), the samples exhibited moderate collapse potential (4.1 percent under a load of 0.5 kips per square foot) and a swell potential (2.5 percent under a load of 0.5 kips per square foot). An atterberg limits test conducted on one (1) near surface sample indicated theses soils tested have a plasticity index of 15 and a liquid limit of 28. An expansion index test conducted indicated a low expansion potential (EI=34).

Clayey Sands: These soils encountered were described as loose to very dense, as determined by standard penetration resistance, N-values, ranging from 4 to 50 blows per foot. The moisture content of these soils tested were 5 percent.

Poorly Graded Gravels: Theses soils encountered were described as loose to medium dense, as determined by standard penetration resistance, N-values, ranging from 8 to 14 blows per foot. The moisture content of the soils tested were 12 percent.

Poorly Graded Sands with Silt: These soils encountered were described as medium dense, as determined by standard penetration resistance, N-values, ranging from 11 to 12. Two (2) atterberg limits tests conducted on these soils resulted in non plastic and no liquid limit value.

R-Value Tests: The results of R-value tests conducted on four (4) near surface bulk samples of near surface soils sampled from the surface to depths raging from 3½ to 5 feet BSG indicated R-values of 40, 41, 59, and 58.

Maximum Density/Optimum Moisture: One (1) maximum density/optimum moisture test (ASTM D1557) conducted on a near surface sample indicated a maximum dry density of 132.4 pounds per cubic foot with an optimum moisture content of 7.5 percent.

Chemical Tests: Chemical tests performed on three (3) near surface soil samples indicated pH values of 7.3, 7.1, and 7.2; minimum resistivity values of 5,600, 2,400, and 6,100 ohm-centimeters; 0.0021, 0.0054, and "none detect" percent by weight concentrations of sulfate; and 0.0012, 0.0097, and "none detect"percent by weight concentrations of chloride, respectively.

At the time of preparation of this report, the results from the top soil analysis were not available. When complete, the top soil analysis tests will be provided.

5.4 **<u>Groundwater Conditions</u>:** At the time of our field exploration, groundwater was encountered in eight (8) of the sixty six (66) test borings at depths ranging from 13⅓ to 25 feet BSG, drilled during our June 2011 field exploration. Test boring B-16, where groundwater was initially encountered at a depth of about 25 feet BSG, was left open for a period of 24 hours. After a period of 24 hours, groundwater was measured at a depth of 23½ feet BSG in test boring B-16.

In general, groundwater was encountered at deeper depths in the north corner of the building pad (approximately 23½ feet BSG) when compared to the southern and eastern building pad corners (17.5 and 17.7 feet BSG, respectively). In addition, two (2) test borings (B-7 and B-10) drilled along the south-southwest property boundary, adjacent to Third Street, encountered groundwater at depths ranging from 12.8 to 13.3 feet BSG. Therefore, based on test borings where groundwater was encountered, groundwater appears to be deeper in the north and west when compared to the eastern and southern portions of the site.

Water well records were reviewed on the Department of Water Resources' On-Line Water Well Database website and the USGS Groundwater Watch website, however, no groundwater data could be located for wells in the vicinity of the site.

It should be recognized that water table 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.

It should be noted that shallow cemented hardpan type soils (where present) will increase the potential for perched water from surface irrigation and stormwater that infiltrates the near surface soils. Due to the anticipated low permeability of the hardpan material, water may infiltrate the surface soils and result in a perched condition on top of the hardpan materials, ponding and migrating laterally over the top of the hardpan materials. Perched water may reduce the drainage capacity of the soils, increasing the potential for moisture-related problems. Therefore, control of surface runoff and irrigation water will be an important aspect of site development.

5.5 Climatic Conditions: According to the Climatic Atlas of the United States, published by the U. S. Department of Commerce, the project area receives approximately 12.06 inches of precipitation annually with the majority of the precipitation occurring during the months of December through March. These months typically receive between 2.02 and 2.55 inches of precipitation per month. In addition, the average daily temperature is reported to be above 32 degrees Fahrenheit.

Based on the information provided in the Climatic Atlas of the United States, there is a high potential that during the wet period of the year, the moisture content of surface soils will be greater than optimum. It is anticipated that the surface soils will become unstable during compactive effort under these conditions. Therefore, it is anticipated that wet soils could have a significant impact on site grading/earthwork operations during the wet period of the year.

6.0 EVALUATION

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

6.1 Surface Conditions and Existing Improvements to be Removed: At the time of the field exploration in June 2011, the subject site was generally vacant in the north to northwestern two-thirds and occupied by a total of four (4) residential properties in the south to southeastern third of the site. The area occupied by residences were located within a dense grove of mature eucalyptus trees.

Existing single family homes and associated improvements, including swimming pools, underground utilities, septic systems, foundations, etc. are present within the southern third of the proposed development. All existing surface and subsurface improvements associated with the single family homes should be removed entirely prior to development of the site, and not buried or crushed in place. Existing improvements (i.e. foundations, utilities, etc.,) shall be removed and the excavations backfilled with engineered fill.

Also, existing underground water and gas utilities were noted along Cambern Avenue and Central Avenues and existing fire hydrants were noted at the intersection of Crane Street with the southwestern property boundary and along Cambern Avenue. It is anticipated that these utilities may be located in pavement areas, drive entrances, and landscape areas. The compaction characteristics of existing public, underground utility trenches located within the limits of proposed improvements are not known. Supporting proposed improvements over trench backfill soils which were not sufficiently compacted may result in unwanted settlement and distress to proposed improvements (i.e. settlement of pavements, curbs, etc). In the event records documenting the compaction of the existing trenches are available, this information should be provided to our firm for review. In the event documentation of the backfill compaction is not available, in-place density testing within of the existing backfill could be conducted. In the event the backfill is not adequately compacted and presents a potential for future settlement and damage to the new improvements, existing underground utilities located in areas of proposed pavements, drive entrances, etc., which are planned to remain in service, should be excavated and backfilled as engineered fill.

Undocumented fills were encountered in five (5) test borings drilled during our June 2011 investigation. The undocumented fill soils were encountered from the surface to depths ranging from 1 to 8½ feet BSG. The undocumented fill soils generally consisted of silty sand soils with trash, wood, and concrete debris. The test borings which encountered undocumented fills (B-1, B-4, B-8, B-9, and B-19) are within the limits of the eucalyptus trees and near the single family homes located in the southern third of the site. Thus, undocumented fills should be anticipated during site grading and should be removed and replaced with compacted engineered fill. Prior to reuse of these soils as engineered fill, the debris would need to be removed, which may require special procedures such as screening or hand picking.

In addition to the undocumented fill soils encountered during this investigation, cobbles and boulders were noted at the surface near the northeastern site corner. Based on our review of available historical aerial photographs, numerous debris/soil piles were located near or adjacent to the area where the embedded cobbles and boulders were noted. Therefore, it is anticipated that the partially buried cobbles and boulders may be associated with the former debris piles. It is unknown whether the former debris piles were removed from the site or spread over the native surface soils. Therefore, the Contractor shall anticipate the potential to encounter undocumented fills, debris, over-sized rock material, etc., in the areas where the former debris piles were present. The approximate location of the former debris piles are depicted on Drawing No. 2 in Appendix A of this report.

Erosional features were noted in the central and southern portions of the site. The erosional features extended east to west from Cambern Avenue to the central portion of the site and were noted to be approximately 2 to 4 feet wide and appeared to extend to a depth of about 24 inches below adjacent grade. Over-excavation to the depth required to remove loose soils within the drainage features are recommended in the Site Preparation section of this report. It is estimated that loose soils may extend to approximately 1 foot below the bottom of the existing drainage feature. The approximate location of the erosional drainage feature are depicted on Drawing No. 2 in Appendix A of this report.

Removal of the eucalyptus trees and associated root systems will be an integral part of site preparation for the proposed construction. Based on our discussions with Mr. Carter Pierce with J.M. Lord, Inc., it is our understanding that the root systems for eucalyptus trees may extend to depths of 4 to 5 feet BSG and may extend laterally throughout the canopy area. Removal of the trees and associated root systems will be required prior to site grading. Roots greater than 1/4 inch will not be considered suitable for use within the on-site fill soils. This may require special procedures, such as screening the soils to remove roots prior to use as engineered fill and segregating and offhauling organic materials. The approximate limits of the areas where eucalyptus trees were noted during our investigation are depicted on Drawing No. 2 in Appendix A of this report.

6.2 **Permeability Characteristics of Near Surface Soils:** The in-place soils were tested at three (3) locations to estimate hydraulic conductivity for the native soils. The tests were conducted using a Guelph Permeameter as described in ASTM D5126, "Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone."

The field tests are used to estimate the rate of intake of water into soils under a constant water head to determine a field saturated hydraulic conductivity. However, it should be noted the tests do not take into account the long term effects of subgrade saturation, silt accumulation, groundwater influence, nor vegetation. Accordingly, an appropriate safety factor should be applied to the test results for use in the drainage system design for the proposed detention basins. In addition, the designer should evaluate the site conditions and determine that the data is appropriate for the desired application before use. Based on the presence of shallow stiff to hard clays, including cemented zones within the upper 10 feet encountered within the majority of the soil borings, and the associated poor drainage characteristics of these materials, the near surface soil characteristics are not well-suited for long term infiltration of stormwater. The results included in Table No. 1 below would indicate the drainage characteristics based on the coefficients of permeability would generally be described as "poor" (Holtz and Kovacs, an Introduction to Geotechnical Engineering, 1981).

The results of the field tests are summarized in Table No. 1 below.

TABLE No. 1

Results of the Hydraulic Conductivity Tests

Location	Depth (Feet)	Hydraulic Conductivity (cm/sec)
P-1	3	5.1×10^{-04}

Location	Depth (Feet)	Hydraulic Conductivity (cm/sec)
P-2	3	9.5×10^{-04}
P-3	3	3.2×10^{-04}

Based on review of the Natural Resources Conservation Service (NRCS) web soil survey maps, the subject site is identified to include soils identified as Arbuckle Gravelly Loam for the majority of the site and Garretson Gravelly Very Fine Sandy Loam for a portion of the site. The NRCS soil survey report indicates the drainage capacity of the limiting layer to be 0.20 inches per hour. The NRCS also lists estimated hydraulic conductivities of about 5×10^{-04} cm/sec to 9×10^{-04} cm/sec for the primary surface soils. Based on the results of our field hydraulic conductivity testing, the average of three (3) tests conducted at 3 feet BSG indicated an average hydraulic conductivity of 5.9×10^{-04} cm/sec.

Due to the poor drainage characteristics of the near surface soils, the presence of shallow groundwater, and the cementation noted within the near surface soils in the majority of the borings, it would not be recommended to rely on a long term infiltration capacity for the near surface soils for disposal of large quantities of stormwater.

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

In evaluation of the expansive soils, expansion index testing was performed on representative samples of the near surface soils. The expansion index testing was performed in accordance with ASTM D4829. The testing conducted by our firm indicated that the near surface soils had a low expansion potential, with an expansion index value of 39. Based on the low expansive potential of the soils tested, recommendations for use of an imported non-expansive engineered fill below interior and exterior slabs on grade are provided to limit the heave to the 3/4 inch vertical rise specified by Retail Tenant.

6.4 Slope Grading and Slope Setbacks: The existing soils encountered during our investigation consisted primarily of silty sands, lean clays, and sandy lean clays, which based on our field observations, exhibit a potential for excessive erosion as evidenced by erosional features extending up to a depth of about 2 feet BSG. Given the desert-type climate at the site, establishing and maintaining adequate surface vegetation for slopes to reduce the potential for erosion would be difficult. The recommendations in this report have been prepared assuming the proposed slopes for the project as part of the detention basin will not be irrigated to establish permanent rooting surface vegetation cover.

The depths of the proposed basins were unknown at the time of this investigation. In order to reduce the potential for excessive soil erosion, slope movements and related maintenance of the detention basin side slopes, the basin slopes shall not be greater than 2½H to 1V. If steeper basin slopes are desired, alternative recommendations for use of selectively graded on-site soils for fill material less susceptible to erosion, cement treating the on-site soils to be used in the exposed slope, or the use of a protective liner could be considered.

Foundations should be setback from cut, fill, and native slopes to provide adequate foundation support and protection against erosion and shallow slope movements. Structures should be setback a minimum of 15 feet from the top of descending slopes (foundations at the top of the cut, fill, or native slopes exceeding 2½H:1V) or by a distance of 1/3 the slope height, whichever is greater. Pavements, exterior flatwork and landscaping improvements may be placed within the setback area, however, these improvements may be subject to damage from future slope movement or erosion.

6.5 Static Settlement and Bearing Capacity of Shallow Foundations: The increases in effective stress to underlying soils which can occur from new foundations and structures, placement of fill, withdrawal of groundwater, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structure and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Based on the subsurface data and laboratory testing performed as part of this report, static settlement was estimated. In order to reduce the potential for excessive total static and differential static settlement of new foundations, it is recommended new foundations be supported on engineered fill established by over-excavation and compaction. New foundations supported on engineered fill as recommended in this report would reduce the estimated static total and differential settlement to ¾ inch and ½ inch in 40 feet, respectively. The static settlement estimates are based upon a net allowable soil bearing pressure of 2,000 pounds per square foot for footings supported on engineered fill (for dead-plus-live loads).

The site preparation recommendations for the proposed foundations are provided in this report to reduce static settlements to meet the Retail Tenant Geotechnical Investigation Specifications and Report Requirements. The subgrade soils for support of the foundations should be prepared by over-excavation and compaction to provide engineered fill below foundations and floor slab areas as recommended in the site preparation section of this report to achieve the Retail Tenant static settlement requirements.

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 the weight of the concrete may be neglected in design. The net allowable soil bearing pressure was selected to satisfy the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A minimum factor of safety of 3 was used to determine the allowable bearing capacity based on the Terzaghi equations.

For the purpose of design, seismic settlements should be considered in addition to the static settlements. Estimates of potential seismic settlement are provided in the section of this report entitled "Liquefaction and Seismic Settlement" of this report.

6.6 **Interior Slab-on-Grade Construction:** Several issues need to be considered to limit the potential for damage to slabs during construction. These issues include: 1) differential slab movement at interior columns (if any); 2) aggregate base sections below the slabs, and 3) construction equipment loads on the slabs.

The method of slab construction at interior columns can potentially damage the overlying slabs. In some cases, the subgrade preparation for the slab is not continuous across the top of spread footings. Often, the zone above the top of footings is backfilled with concrete during slab placement. This results in a differential slab support condition and increases differential concrete shrinkage which often causes cracking at the soil/base-to-concrete transition. This crack appears as an outline of the underlying footing at the floor surface. The potential for this type of slab cracking can be reduced by backfilling the zone above the top of the footing and below the bottom of slabs with an approved backfill material and/or an aggregate base section below the floor slab. This procedure will provide more uniform support for the slabs which should reduce the potential for cracking.

Compacted subgrade can experience instability under construction traffic loads resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. A layer of aggregate base (AB) can reduce the potential for instability under the construction traffic. Also, the improved support characteristics of the AB can be used in the design of the slab sections. Based on the soils encountered and the recommendations provided herein for placement of aggregate base below interior slabs, a design modulus of subgrade reaction of 150 pounds per square inch per inch may be used for design of interior slabs on grade.

It should be noted that cranes and heavy construction equipment can impart intense loads on slabs and pavements. The loads from cranes and/or heavy construction equipment that may operate on slabs or pavements should be assessed by the contractor prior to placing equipment on the slab.

6.7 Faulting and Seismic Design Coefficients: The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the Elsinore (Glen Ivy) Fault, located about 3.2 miles (5.1 kilometers) west of the site. Therefore, the potential for fault rupture at the site is considered low.

It is our understanding that the 2010 CBC will be used for structural design. Based on the 2010 CBC, the site is classified as a stiff soil (S_D) site with standard penetration resistance, N-values averaging between 15 and 50 blows per foot for the upper 100 feet BSG. Considering a five percent damped design spectral response acceleration for short period (SD_S) of 1.007, the peak horizontal ground acceleration as defined in the CBC for liquefaction analyses was estimated to be 0.40g.

A table providing the recommended site coefficients and earthquake spectral response acceleration values for the project site is included in the "Foundations" recommendations section of this report.

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

Based on our review of The Riverside County Land Information System website, the subject site is mapped in an area shown to have a very high potential for liquefaction. Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by test borings and the results of laboratory testing. Liquefaction and seismic settlement analyses were conducted using the computer program LiquefyPro, developed by CivilTech Software. A horizontal ground acceleration of 0.40g (based on the 2010 California Building Code requirements) and a predominant maximum considered earthquake magnitude of 6.8 were used for the evaluation. The N-values generated based on the test borings were used in the analysis. At the time of this investigation, groundwater was encountered at depths ranging from 13⅓ to 25 feet BSG. Historical groundwater data was not available for the site at the time of this investigation, therefore, a groundwater depth of 10 feet BSG was utilized for the liquefaction/seismic settlement analysis.

The results of the seismic settlement analysis indicates total seismic settlement ranging from about 1.5 inches to 2.5 inches and a differential seismic settlement of up to 1.25 inches in 40 feet.

In the event the estimated seismic settlements are considered excessive for design of conventional shallow foundation systems, a supplemental investigation would be recommended using CPT soundings to conduct a more detailed assessment of liquefaction potential and to develop final recommendations for design and preparation. In the event the supplemental investigation indicated the seismic settlement estimates were greater than allowed for a conventional foundation system, recommendations for ground improvement, or alternate foundation types would be prepared.

It has been our experience that additional field testing in the form of cone penetration tests (CPT) may generate data which would support refined estimates of seismic settlement. In contrast to the drilled borings, CPT testing provides nearly continuous penetration resistance (soil density) data and thus a more detailed soil density and penetration resistance profile. Since soil density strongly impacts seismic settlement, evaluation of CPT test results generally produce refined seismic settlement estimates when compared with data from hollow stem auger borings.

6.9 **Asphaltic Concrete (AC) Pavements:** Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation (Caltrans) Highway Design Manual. The traffic loading data for Retail Tenant Supercenter stores were obtained from the Retail Tenant Geotechnical Investigation Specifications and Report Requirements (February 7, 2011). The "standard duty" pavement for the store should be designed for a life of 20 years and an ESAL (18 kips) of 15 axles per day. This equates to an ESAL of 109,500 for the design life of the pavement and a Traffic Index of 7.0. The "heavy duty" pavement should be designed for a life of 20 years and an ESAL (18 kip) of 46 axles per day. This equates to an ESAL of 335,800 and a Traffic Index of 8.0. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

The subgrade support characteristics of the native soils were evaluated using Resistance (R)-value tests. The results of the tests on samples collected indicated R-values of 40, 41, 59, and 58. The recommendations for asphaltic concrete pavements were prepared using an R-value of 40.

Asphaltic concrete pavement section details are described in the Recommendations Section and are presented on Drawing No. 4 in Appendix A.

6.10 **Portland Cement Concrete (PCC) Pavements:** Recommendations for Portland Cement Concrete pavement structural sections are presented in the "Recommendation" section of this report. The structural section was based primarily on the Portland Cement Association's "Thickness Design of Highway and Street Pavements."

The traffic loading data for Retail Tenant Supercenter stores were obtained from the Retail Tenant Geotechnical Investigation Specifications and Report Requirements (February 7, 2011). The "standard duty" pavement for the store should be designed for a life of 20 years and an ESAL (18 kip) of 15 axles per day. This equates to an ESAL of 109,500 for the design life of the pavement. The "heavy duty" pavement should be designed for a life of 20 years and an ESAL (18 kip) of 46 axles per day. This equates to an ESAL of 335,800 for the design life of the pavement. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

The PCC pavement sections were designed for a life of 20 years, a load safety factor of 1.1, an ESAL (18 kips) of about 15 axles per day (this equates to 109,500 ESALs for the design life of the standard duty pavements), and an ESAL (18 kips) of about 46 axles per day (this equates to 335,800 ESALs for the design life of the heavy duty pavements, and a modulus of rupture of 550 pounds per square inch (compressive strength of 3,500 psi) at 28 days for concrete. Tests performed on the native soils indicated a correlated k-value of 175 psi/in. A higher k-value than the subgrade k-value is provided for this pavement section, since the concrete will be underlain by a 6-inch layer of Class 2 aggregate base material (minimum R-value of 78). Therefore, a k-value of 215 psi/in at the top of the aggregate base was used in design. PCC pavement section details are presented on Drawing No. 5 in Appendix A.

6.11 <u>Corrosion Protection</u>: 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. Corrosion can eventually damage or destroy a metallic object.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on 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 soil tested are included in the Conclusions section of this report. 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.

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

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 geotechnically suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report are followed.

7.2 The near surface soils encountered generally consisted of silty sands from the surface to depths ranging from 1 foot to about 8½ feet BSG. Below the near surface sands, interbedded layers of sandy silty clays, sandy lean clays, lean clays, clayey sands, sandy silts, poorly graded sands with silt, and silty sands were encountered to the depth of approximately 45 feet BSG. In addition, the near surface lean clays and sandy lean clays, where encountered, within the upper 5 feet BSG were generally described as "slightly cemented" hardpan soils. Three (3) of the test borings conducted within the proposed building pad encountered undocumented fill soils from the surface to depths ranging from 1 to 3½ feet BSG. Also, two (2) test borings conducted outside of the limits of the proposed building pad encountered undocumented fills from the surface to depths of 2½ and 8½ feet BSG. The undocumented fills generally consisted of silty sand soils with trash and concrete debris.

7.3 The results of the expansion index testing indicated that the near surface sandy lean clay soils have a low expansion potential.

7.4 Groundwater was encountered in eight (8) borings at depths ranging from 13⅓ to 25 feet BSG, drilled during our June 2011 field exploration. Test boring B-16 was left open for a period of 24 hours. After a period of 24 hours, groundwater was measured at a depth of 23½ feet BSG in test boring B-16.

7.5 Weakly cemented "hard pan" type soils were encountered within the upper 5 feet BSG. Based on the presence of shallow hardpan soils, perched water due to infiltration of surface water could result.

7.6 The majority of the building pad and the southern third of the site was occupied by many large, mature eucalyptus trees. Removal of the trees, root structures, root balls and organics will be a critical part of site preparation. It is anticipated that the root system will be extensive and may extend to the minimum depths of 4 to 5 feet BSG. Site preparation will require removal of all existing trees, root balls roots and organics and disposal of these materials offsite.

7.7 Site preparation recommendations are provided in this report to reduce the static settlement of new foundations to ¾ inch total and ½ inch differential in 40 feet in accordance with the Retail Tenant Geotechnical Investigation Specifications and Report Requirements by supporting new foundations on engineered fill.

7.8 Based on our evaluations from the boring data, total liquefaction seismic settlements are estimated to range from 1.5 inches to 2.5 inches based on the design earthquake. It is further estimated that differential seismic settlements up to 1.25 inches in 40 feet may result. Provided the foundations may be designed to tolerate the anticipated total and differential settlements provided in this report, the use of shallow spread foundations supported on engineered fill is considered the most economical foundation type for the proposed construction.

In the event the estimated seismic settlements are considered excessive for design of conventional shallow foundation systems, a supplemental investigation would be recommended using CPT soundings to conduct a more detailed assessment of liquefaction potential and to develop final recommendations for design and preparation. In the event the supplemental investigation indicated the seismic settlement estimates were greater than allowed for a conventional foundation system, recommendations for ground improvement, or alternate foundation types would be prepared.

7.9 The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, the potential for fault rupture at the site is considered low.

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

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, we present the following recommendations 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 by Moore Twining are integral to the development of construction documents.

8.1 General

8.1.1 This report has been prepared based on anticipated site grading described in the Background Information section of this report. When available, Moore Twining should be provided a grading plan to verify the recommendations provided herein are appropriate.

8.1.2 The landscape, civil, and foundation plans, when available, should be provided to Moore Twining for review. The recommendations presented in this report could change depending on the proposed site improvements, grading, etc. Therefore, it is critical that improvement plans, when available, be provided to Moore Twining for review.

8.1.3 A plan should be developed to identify existing improvements which will require removal. As a minimum, the plans should show the existing improvements planned for removal such as the existing structures, foundations, septic tanks, underground utilities, etc. These elements should be removed in their entirety and the resulting excavations backfilled with engineered fill.

8.1.4 A preconstruction meeting including, as a minimum, the owner, general contractor, foundation and paving subcontractors, civil engineer, and the construction testing laboratory should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.

8.1.5 The contractor is responsible for including in the base bid the costs to perform the work required by the project plans, the project specifications, the City of Lake Elsinore, and Riverside County, whichever is most stringent. After review of the geotechnical report and the aforementioned documents, the contractor(s) bidding on this project should determine if the data are sufficient for accurate bid purposes. If the data are not sufficient, the contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect more data as required to prepare accurate bids.

8.1.6 The contractor should monitor the existing improvements to remain which are within influence of the work, such as existing improvements along property lines. The contractor will be required to take precautions to prevent damage to the existing improvements and adjacent properties during excavation. Any damage to existing improvements should be repaired at no cost to the owner.

8.1.7 Contractors should anticipate that subgrade instability may occur if grading is conducted during periods of inclement weather. Where wet, unstable soil conditions are experienced, stabilization may be achieved by use of a geotextile fabric and compacted aggregate base, or chemical (i.e., lime, cement, etc.) treatment. The contractor should include in the bid the costs for stabilization of all wet, unstable areas in accordance with the project specifications. No change orders will be allowed for wet weather conditions, wet soil, soil instability, etc. or mitigation measures such as chemical treatment, geotextile fabric, rock, soil import, etc.

8.1.8 The Contractor should anticipate the potential for groundwater to be encountered in excavations extending greater than 10 feet BSG. Therefore, it should be anticipated that shallow groundwater may be encountered during grading and utility construction. Contractors should anticipate the need to provide dewatering from areas of grading, utility trench construction, etc. No change orders will be allowed for shallow water conditions. A dewatering specification is included in Appendix F of this report.

8.1.9 The Contractor should use appropriate equipment such as low pressure equipment, steel tracks, etc. to achieve the required excavation, compaction and site preparation to minimize rutting and subgrade instability.

8.1.10 In the event the prepared subgrade moisture conditions dry below the recommended moisture contents, additional moisture conditioning, or scarification, moisture conditioning and compaction would be necessary to re-establish the required moisture contents prior to placement of aggregate base and concrete slabs, etc.

8.2 Site Grading and Surface 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. If any grading is conducted around the exterior of the building, adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least five feet away from the structure, 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 Shallow cemented "hardpan" type soils were encountered at the site which will increase the potential for perched water from surface irrigation and stormwater sources that infiltrate the near surface soils. Due to the low permeability of the "hardpan" material, water may infiltrate and result in a perched condition, ponding and migrating laterally over the top of the hardpan materials. Perched water will reduce the drainage capacity of the soils, increasing the potential for moisture-related problems. Therefore, providing and maintaining positive drainage collection systems, using thickened slab edges and deepened curbs and control of surface runoff and irrigation water will be important aspects of design and site development to reduce the potential for moisture-related problems.

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

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

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

8.2.6 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.

8.2.7 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 structure and the resulting runoff directed away from the structure at a minimum of 2 percent.

8.3 Proposed Detention Basin

It is our understanding that a detention basin is planned east to southeast of the proposed Retail Tenant store. The following recommendations have been prepared for the proposed basins. At the time of this report, exact details relative to the depth of the proposed basins were not provided. When available, these details should be provided to Moore Twining for review and consideration in preparation of the final geotechnical report.

8.3.1 In order to reduce the potential for soil erosion, slope movements and related maintenance of the detention basin side slopes, the basin slopes shall not be greater than 2½H to 1V and positive rooting vegetation should be established for exposed slopes. If steeper basin slopes are required, Moore Twining should be contacted to provide alternative recommendations.

8.3.2 The sidewalls of the basin will require periodic maintenance due to erosion. Thus, erosion control should also be implemented for the side slopes of the basin. In addition, the exposed slope surface should be conditioned and compacted to a minimum of 95 percent relative compaction.

8.3.3 The top of the side slopes of the basin should be graded to prevent runoff from flowing over the top of the slopes. Ramps (maintenance and sediment removal) into the basins should be protected against erosion where concentrated runoff may drain along the ramp.

8.3.4 As a minimum, setbacks from the top of the detention basin slopes shall conform to the requirements of the 2010 California Building Code. This includes setting structures and improvements a minimum distance away from the top of the slope equal to one-third of the height (H/3) of the slope but need not exceed 40 feet maximum. Site work improvements such as paving and curbs may be planned within the setback zone; however, improvements within the setback zone may be susceptible to distress due to shallow slope movements.

8.4 <u>Site Preparation</u>

8.4.1 The Contractor should locate all on-site water wells (if any) and septic system components. All wells and septic systems encountered during construction should be abandoned per state and local requirements and the project specifications under the observation of the CTL. The Contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of abandonment to the owner upon completion. At a minimum, it is recommended that the well casings be removed to a minimum depth of 60 inches below the finished grade and the excavations backfilled with engineered fill.

8.4.2 Subsurface improvements such as irrigation lines (if any), former foundations, utilities, former septic systems, pools, underground tanks, etc., should be removed entirely and not crushed or buried in place. The resulting excavations should be cleaned of all loose, organic or disturbed soils, the exposed native soils should be scarified to a depth of 8 inches then compacted as engineered fill. The excavation should be backfilled with compacted engineered fill.

8.4.3 The proper removal of existing trees and their associated root structures is an important aspect of this project and should be properly planned and monitored. Excavation of tree roots, root balls, etc. may require excavation to depths of 4 to 5 feet below existing site grades. Therefore, the contractor should anticipate excavations to minimum depths of about 5 feet to remove all root systems greater than about 1/4 inch in diameter and concentrations of organics greater than 3 percent. Soils containing organic matter such as root clumps, roots exceeding ¼ inch in diameter, with organic contents above 3 percent should be completely removed and not used as engineered fill. Limbs, tree branches, roots, etc. should not be disced into the soils. These materials should be screened or raked and hand-picked, as necessary, to ensure proper removal of all roots and organics prior to use of the onsite soils as engineered fill. A tree and root removal plan should be developed by the Contractor. The plan should specify how the Contractor proposes to remove the trees, roots, and organic matter generated during the removal process. The plan should also specify how the excavations and loose soils disturbed during this process will be addressed. The bottom of the excavation should extend a minimum of 12 inches below the bottom of the tree root ball and root systems to be removed. Upon removal, the resulting soils should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations. The Retail Tenant CTL should be contacted to observe removal of the tree roots and organics as part of the site preparation.

8.4.4 Stripping shall be conducted in all areas of existing surface vegetation and where root systems are present. The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. A minimum stripping depth of about 4 inches is recommended. The actual depth of stripping should be reviewed by the CTL at the time of construction. Deeper stripping and excavation will be required to remove the root systems of the existing trees as described in Section 8.4.3 of this report. Stripping and clearing of debris should extend throughout the areas of the proposed improvements. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner.

8.4.5 After excavation and removal of existing trees, root systems, organics and stripping of surface vegetation and root structures, the subgrade soils in the proposed building area and over-build zone, shall be prepared by over-excavation to a depth of 12 inches below improvements to be removed (i.e. foundations, septic systems, utilities, etc.), to the depth to remove undocumented fills (encountered to depths ranging from 1 to 8½ feet BSG), to the depth required to remove the tree root systems (estimated to extend to depths ranging from 4 to 5 feet BSG), to a minimum depth of 48 inches below preconstruction site grades, to a minimum of 12 inches below the bottom of the existing drainage features identified in this report, or to 2½ feet below foundations, whichever provides the deeper excavation.

Over-excavation should include the entire building footprint and overbuild zone. The building pad is defined as the areas to be occupied by the building, adjacent sidewalks, garden center, porches, ramps, stoops, truck wells/docks, concrete aprons, compactor pad, etc., and to a minimum horizontal distance of five (5) feet beyond these areas, or to a horizontal distance equal to the depth of engineered fill at the outside edge of these improvements, whichever is greater. The project civil engineer should show the overbuild line on the plans. Slot cutting only below foundations will not be allowed. Upon approval of the over-excavation by the Retail Tenant CTL, the subgrade soils at the bottom of the over-excavation should be scarified to a depth of 12 inches, moisture conditioned between optimum and three (3) percent above optimum moisture content and compacted as engineered fill.

8.4.6 For the purpose of this report, interior slabs should be underlain by 6 inches of non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) over a minimum of 6 inches of imported non-expansive engineered fill over the depth of engineered fill recommended above. As an alternative to the use of an imported non-expansive engineered fill, the interior slabs on grade may be supported on a minimum of 10 inches of non-recycled Class 2 aggregate base or Crushed Aggregate Base (CAB) over the depth of engineered fill recommended below foundations.

8.4.7 It is recommended that extra care be taken by the Contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. This is the sole responsibility of the Contractor. The Contractor shall verify in writing to the owner 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). This verification shall be performed by a licensed surveyor. The licensed surveyor shall provide a plan and cross-sections that demonstrate that the horizontal and vertical extent of the over-excavation required by this report was achieved. The surveyor shall also provide a written report that states the over-excavation was performed in accordance with the project geotechnical engineering report. This verification should be provided prior to requesting pad certification or excavating for foundations.

8.4.8 After excavation and removal of existing trees, root systems, organics and stripping of surface vegetation and root structures, the subgrade soils in areas of proposed pavements, exterior concrete slabs or areas to receive fill outside the building pad should be prepared by excavation to a minimum of 12 inches below preconstruction site grades, 12 inches below the bottom of the aggregate base, or to the depth required to remove the tree root systems (estimated to extend to depths ranging from 4 to 5 feet BSG), to the depth to remove undocumented fills (encountered to depths ranging from 1 to 8½ feet BSG), whichever is greater. Following excavation, the exposed subgrade soils shall be scarified to a minimum depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill. The subgrade preparation performed for the new pavement and exterior slab areas should extend a minimum of 2 feet beyond the edge of the planned improvements, or by a distance equal to the depth of fill, whichever is greater.

8.4.9 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all onsite soils excavated or disturbed should be compacted as engineered fill.

8.4.10 Open graded gravel or rock material such as ¾-inch crushed rock or ½-inch crushed rock shall not be used on this project, including trench backfill. In the event an agency having jurisdiction requires the use of an open graded rock, the open-graded rock shall be fully encapsulated in a geotextile fabric.

8.4.11 Final grading shall produce a pad ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad or portion thereof 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. In addition, the moisture content of the final grading should be maintained within optimum to three (3) percent above optimum moisture content prior to placement of AB and excavations for footings.

8.4.12 The Contractor is 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 is responsible for performing the tests, assessments, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

8.5 Engineered Fill

8.5.1 The on-site near surface soils encountered are predominantly silty sands with interbedded layers of lean clay and sandy lean clay soils which exhibit expansive characteristics. The existing near surface soils that are free of organics (less than 3 percent by weight), free of roots and debris, and oversized materials greater than 3 inches in largest dimension will be suitable for use as fill material below a depth of 6 inches below the recommended aggregate base within the building and over-build zone and directly below the aggregate base outside of the building pad preparation limits, provided they are properly moisture conditioned and compacted. The upper 12 inches below interior slabs on grade should consist of 6 inches of Class 2 aggregate base or crushed aggregate base (CAB), over 6 inches of imported, non-expansive engineered fill. Additional mechanical effort should be anticipated to remove or pulverize the excavated cemented "hardpan" material and blend it with onsite soils prior to use as engineered fill. Oversized material should be removed from the fill soils as necessary to establish a well-graded fill material with a maximum particle size of 3 inches in largest dimension. The

Contractor should be aware that the on-site undocumented fill soils will require processes such as hand picking or screening to remove debris and oversized material prior to reuse as engineered fill. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

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

8.5.3 Import fill soil (if needed) should be non-expansive, free of recycled aggregate base materials 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	15 - 40
Plasticity Index	Less than 15
Expansion Index (ASTM D4829)	Less than 15
Organics	Less than 3 percent by weight
R-Value	Minimum 40
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 10,000 ohms-cm

Prior to importing fill, the Contractor shall submit test data that demonstrates that the proposed import complies with the recommended criteria for both geotechnical and environmental criteria. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, analytical data specific to the source of the import material in accordance with the Department of Toxic Substances Control, "Information Advisory, Clean Imported Fill Material," dated October 2001.

8.5.4 Dock walls and retaining walls should be constructed with imported, non-recycled, non-expansive granular free draining backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. The onsite soils are not allowed within a 1 horizontal to 1 vertical plane from the back of the retaining wall footings. This requirement should be detailed on the construction drawings. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	5 - 35
Internal Angle of Friction	30 degrees

8.5.5 Imported, non-expansive soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum to three (3) percent above optimum moisture content, and compacted to a non-yielding condition at a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.5.6 On-site soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to between optimum and three (3) percent above optimum moisture content, and compacted to a non-yielding condition at a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.5.7 In-place density tests should be conducted in accordance with ASTM D6938 (nuclear methods) at a frequency of at least:

Area	**Minimum Test Frequency**
Building Area	1 test per 2,500 square feet per lift
Pavements or Areas of Mass Grading	1 test per 10,000 square feet per lift
Utility Lines	1 test per 200 feet per lift

The above testing frequencies are suggested rates for tests. Testing frequency should be adjusted by the field technician and the engineer as needed based on earthwork observation considering the methods used for compaction and the soil conditions; however, the testing frequency should not be less than listed above.

8.5.8 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material.

8.5.9 Aggregate base below the interior building slab on grade shall be non-recycled and comply with Class 2 aggregate base (AB) per State of California Standard Specifications or comply with Crushed Aggregate Base (CAB) per the Greenbook. Aggregate base used for pavement construction should comply with Class 2 aggregate base in accordance with the State of California Standard Specifications, Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) per the Greenbook. Aggregate base shall be compacted to a minimum relative compaction of 95 percent in accordance with ASTM D1557 standards. Documentation, including laboratory test data, should be provided to the Retail Tenant CEC prior to delivery of the aggregate base to the site indicating that the aggregate base meets project specifications and is non-recycled, where applicable.

8.5.10 Recycled materials cannot be used in the building pad unless approved by Retail Tenant.

8.6 Foundations

8.6.1 Spread and continuous footings supported on engineered fill prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.

8.6.2 All footings should have a minimum width of 15 inches, regardless of load. All perimeter footings should be supported at a minimum depth of 24 inches below the top of the slab, or 18 inches below the lowest adjacent grade, whichever is deeper. Interior foundations should extend a minimum of 18 inches below the bottom of the interior floor slab.

8.6.3 The foundations should be designed and reinforced for the anticipated differential settlements. A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations and slabs on grade based on: a total static settlement of ¾ inches; a differential static settlement of ½ inch in 40 linear feet of continuous footings for footings supporting masonry walls; a differential settlement of L/500 between column foundations, where L is the horizontal distance between column foundations; a total swell of ¾ inch and a differential seismic settlement of up to 1.25 inches in 40 feet.

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

8.6.5 Structural loads for miscellaneous foundations (such as sound walls, trash enclosures, screen walls, monument signs, etc.) should be evaluated on a case by case basis to develop supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, if native soils are used as engineered fill, miscellaneous foundations may be supported on spread or continuous footings placed entirely on at least 1 foot of engineered fill, engineered fill to a depth of at least 3 feet below preconstruction site grades, or to the depth required to remove existing root systems, whichever provides the deeper fill. The zone of engineered fill shall extend at least 5 feet beyond the edges of foundations on all sides. Upon approval of the over-excavation limits, the soils at the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted as engineered fill to a minimum of 95 percent relative compaction. The resulting excavation should be backfilled to finished grades with engineered fill. Footings extending to a minimum depth of 12 inches below the lowest adjacent finished grade and a minimum width of 15 inches for these improvements may be designed for a maximum allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads for footings. This value may be increased by one-third for short duration wind or seismic loads.

8.6.6 The following values were developed using the Ground Motion Parameter Calculator provided by the United States Geological Survey (http://earthquake.usgs.gov/) in accordance with the 2010 CBC.

Seismic Factor	2010 CBC Value
Site Class	D
Spectral Response At Short Period (0.2 Second), Ss	1.511
Spectral Response At 1-Second Period, S_1	0.600
Site Coefficient (based on Spectral Response At Short Period), Fa	1.000
Site Coefficient (based on spectral response at 1-second period) Fv	1.500
Maximum considered earthquake spectral response acceleration for short period, SM_S	1.511
Maximum considered earthquake spectral response acceleration at 1 second, SM_1	0.900
Five percent damped design spectral response accelerations for short period, SDs	1.007
Five percent damped design spectral response accelerations at 1-second period, SD_1	0.600

8.6.7 The moisture content of the footing excavations should be maintained by the contractor until placement of concrete. If the excavations are allowed to dry, conditioning and remedial measures should be conducted to establish moisture contents of at least optimum moisture.

8.6.8 Foundation excavations should be observed by the Retail Tenant Construction Testing Laboratory (CTL) 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 for scheduling the observations and receipt of written confirmation of this observation prior to placement of steel reinforcement.

8.7 Retaining Walls

8.7.1 When available, the locations, types and heights of proposed retaining walls should be reviewed by Moore Twining to evaluate the actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters. The following recommendations were based on general assumptions for conventional, cantilever type retaining walls.

8.7.2 Retaining wall foundations should be supported on a minimum of 18 inches of engineered fill, engineered fill extending a minimum of 3 feet BSG, or to the depth required to remove existing tree root systems, whichever is greater. Retaining wall foundations should extend to the minimum depth of 12 inches below lowest adjacent grade and shall have a minimum width of 15 inches

8.7.3 Retaining wall foundations supported on engineered fill prepared as recommended in this report may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.7.4 Retaining walls should be constructed with imported, non-expansive, granular free-draining backfill placed within the zone extending from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. The onsite soils are not allowed within a 1 horizontal to 1 vertical plane from the back of wall footing (See Drawing No. 3). This requirement should be detailed on the construction drawings. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	5 - 35
Internal Angle of Friction	30 degrees

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

8.7.6 Segmented wall design (mechanically stabilized walls) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for segmented wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. In addition, the designer shall perform sufficient inspections to certify that the wall was constructed per the approved plans and specifications.

8.7.7 Retaining walls may be subject to lateral loading from pressures exerted from the soils, groundwater, slabs-on-grade, and pavement traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

8.7.8 All retaining walls should be constructed with a drain system including perforated drain pipes surrounded by at least 1.5 cubic foot (per lineal foot) of Caltrans Class 2 permeable material, as depicted on Drawing No. 3 in Appendix A. Class 2 permeable material should be compacted to 95 percent relative compaction in accordance with ASTM D1557. Drain pipes should be placed near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system.

8.7.9 For loading dock area retaining walls only, as an alternative to using drain pipes behind the wall to reduce the potential for hydrostatic pressures behind the wall, weep holes may be used, provided that a continuous crushed rock (minimum 1 cubic foot per lineal foot of 3/4 inch rock) and filter fabric section is provided directly behind the wall. The weep holes cannot have the potential for clogging. The weep holes should discharge directly to an approved drainage. Details regarding the loading dock drain system are shown on Drawing No. 3 in Appendix A.

8.7.10 If open graded materials such as crushed rock are used as drain material, these materials should be fully encased in filter fabric and compacted to a non-yielding condition under the observation of the Retail Tenant CTL. These materials, if used, must be placed in maximum lifts of 6 inches and compacted using vibratory equipment. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric.

8.7.11 The Contractor should use light hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The Contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

8.7.12 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., waterproofing measures such as manufactured drainage boards (i.e., Miradrain 6000 or 6200 or approved alternative) should be applied to moisture proof the exterior of the walls. Waterproofing should also be used if effervescence (discoloration of wall face) is not acceptable. The waterproofing system should be designed by a qualified professional.

8.8 Frictional Coefficient and Earth Pressures

8.8.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.34 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.

8.8.2 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot for level soil conditions. A minimum factor of safety of 1.5 should be used when combining the frictional and passive resistance of the soil to determine the total lateral resistance. The upper 6 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance. The passive pressure was calculated based on a passive earth pressure coefficient of 2.75 and a minimum soil unit weight of 100 pounds per cubic foot.

8.8.3 The onsite soils are not considered suitable for use as backfill of vertical walls, such as the loading dock area retaining walls. Backfill of these features extending within a zone defined by a 1 Horizontal to 1 Vertical plane from the back of the wall foundation to the ground surface should consist of an imported, granular fill meeting the requirements of section 8.6.3 of this report. This requirement should be depicted on the project plans. The active and at-rest pressures of imported, granular backfill placed in accordance with this report, may be assumed to be equal to the pressures developed by a fluid with a density of 43 and 65 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.8.4 The active and at-rest pressures for the imported wall backfill soils were calculated based on an active earth pressure coefficient of 0.33, an at rest coefficient of 0.50, and a soil unit weight of 130 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 130 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 130 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of over 130 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind retaining walls.

8.8.5 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

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

8.8.7 Since the pressures recommended in this section do not include vehicle surcharges, it is recommended to use lighter hand operated or walk behind compaction equipment to avoid wall damage during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure.

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

8.9 Interior Slabs-on-Grade

The slabs-on-grade on the project that should be prepared as interior slabs include: the floor slab of the building and any concrete flatwork directly adjacent to the building. 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 trucks, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

8.9.1 The interior floor slab should be reinforced for the anticipated temperature and shrinkage stresses, settlement and swell. Provided the interior floor slab is supported on the recommended 6 inches of Class 2 aggregate base or non-recycled crushed aggregate base (CAB) over a minimum of 6 inches of imported non-expansive engineered fill, a modulus of subgrade reaction of 150 pounds per square inch per inch, may be used for design. In addition, a structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slab-on-grade for a total static settlement of ¾ inch, a differential settlement of L/500 (where L is the horizontal distance in feet between any two points on the floor slab) and a differential static settlement of ½ inch between new slabs and adjacent existing foundations. A heave of up to ¾ inches should also be anticipated in design.

8.9.2 It is recommended new concrete slabs-on-grade be supported on a minimum of 6 inches of non-recycled, Class 2 aggregate base or non-recycled crushed aggregate base (CAB) over a minimum of 6 inches of imported non-expansive engineered fill over the depth of moisture conditioned, compacted subgrade soils recommended in the section entitled, "Site Preparation." The minimum 6 inches of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction purposes. As an alternative to importing a non-expansive engineered fill, interior floor slabs may be supported on a minimum of 10 inches of non-recycled Class 2 aggregate base or non-recycled crushed aggregate base (CAB) over the depth of engineered fill recommended below foundations. The aggregate base should be moisture conditioned to within optimum to three (3) percent over optimum moisture content, and compacted to a minimum of 95 percent relative compaction (ASTM D1557) to a non-yielding condition.

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

8.9.4 The subgrade soils should be tested to verify that the in-situ moisture content is at least optimum just prior to placement of the aggregate base, the vapor retarding membrane, and construction of the slab. If the moisture is below optimum, the dry soils should be moisture conditioned to slightly above the optimum moisture content and maintained until vapor retarding membrane or concrete placement. The moisture content of the subgrade soils should be tested and proper moisture verified by the Retail Tenant CTL within 48 hours of placement of the vapor retarding membrane or the concrete for the slab-on-grade if a vapor retarding membrane is not used. If necessary to achieve the recommended moisture content, the native subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill

8.9.5 ACI recommends that the interior slab-on-grade be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is our understanding that the exposed slab-on-grade will be covered with vinyl ceramic tile. It is recommended that Stegowrap 15 or equivalent should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The layer of Stegowrap 15 should overlay a minimum of 6 inches of non-recycled, compacted AB. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C., the Stegowrap can be placed directly on the AB and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with ASTM C 755-02, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to ASTM E 154-99 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R-96), Addendum, Vapor Retarder Location and ASTM E 1643-98, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

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

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

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

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

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

8.9.11 To reduce the potential for damaging slabs during construction the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to perimeter foundations; 2) provide aggregate base below the slabs; and 3). the loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be considered in the design of the slabs and evaluated by the contractor prior to loading the slab. If cranes are to be used, the contractor should provide slab loading information to the slab design engineer to determine if the slab is adequate.

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

8.9.13 If construction traffic will be traveling over the aggregate base material or the aggregate base will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The proposed compacted subgrade can experience instability under high frequency concrete truck loads during slab construction resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. Often the aggregate base can reduce the potential for instability under the construction traffic.

8.10 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic. These recommendations are intended for rather lightly loaded sidewalks outside the building overbuild zone (i.e. outside the building pad limits). Recommendations for asphaltic concrete pavements and Portland cement concrete slabs subjected to vehicular traffic are included in later sections of this report.

8.10.1 Site preparation for exterior slabs-on-grade should be prepared in accordance with the recommendations section entitled, "Site Preparation." Exterior concrete slabs-on-grade (such as sidewalks, etc.) should be supported on a minimum of 6 inches of aggregate base or non-recycled crushed aggregate base (CAB) over subgrade soils prepared as recommended in the Site Preparation section of this report.

8.10.2 The exterior slabs-on-grade should be designed with thickened edges which extend to the bottom of the aggregate base below the slabs, or deeper as determined by the designer. This should reduce the potential for infiltration of water into the aggregate base below exterior slabs.

8.10.3 Since exterior sidewalks and flatwork 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. Written test results indicating passing density and moisture tests should be in the general Contractor's possession prior to placing concrete for exterior flatwork.

8.10.4 The exposed subgrade soils to receive imported engineered fills below exterior slabs should be tested to verify adequate compaction and moisture conditions prior to placing the imported engineered fills. If adequate compaction and moisture is not verified, the disturbed subgrade should be scarified, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.10.5 The exposed imported engineered fill soils to receive aggregate base below exterior slabs should be tested to verify adequate compaction and moisture conditions prior to placing the aggregate base section, and also within 48 hours of placement of the concrete for the slab-on-grade. If adequate compaction and moisture is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557 at between optimum and three (3) percent above optimum moisture content.

8.11 Asphaltic Concrete Pavements

8.11.1 The subgrade soils below asphaltic concrete pavements should be prepared as recommended in the Site Preparation section of this report.

8.11.2 The exposed subgrade soils to receive aggregate base below pavements should be tested to verify adequate compaction and moisture conditions prior to placing the aggregate base section, and also within 48 hours of placement of the pavements. If adequate compaction and moisture is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557 at between optimum and three (3) percent above optimum moisture content.

8.11.3 The contractor shall proof roll the subgrade of the areas to receive pavements prior to placement and compaction of the aggregate base (AB). All unstable areas should be removed, stabilized, and replaced with engineered fill to achieve a stable condition under the observation of the Retail Tenant CTL.

8.11.4 Prior to placement of asphaltic concrete adjacent to slabs-on-grade, curbs, and gutters, the Contractor shall compact the area immediately adjacent to these features with equipment that can provide adequate compactive effort to the aggregate base adjacent to the vertical face of the concrete to achieve a dense, non-yielding condition.

8.11.5 The following two-layer asphaltic concrete pavement sections are based on an R-value of 40, a traffic index of 7.0 for the "Standard Duty Pavements," and a traffic index of 8.0 for the "Heavy Duty Pavements." The gravel factors for the asphalt concrete pavement layers were applied in accordance with the requirements of the Caltrans Highway Design Manual. Gravel factors of 2.14 and 2.0 were used for standard and heavy duty paving, respectively, and a gravel factor of 1.1 was applied to the aggregate base in accordance with Caltrans requirements. Cross sections depicting the pavement sections are included on Drawing No. 4 in Appendix A of this report.

Table No. 2
Recommended Two-Layer Asphaltic Concrete Pavement Sections

Pavement Type	Traffic Index	AC Thickness (inches)	AB Thickness (inches)	Compacted Subgrade (inches)
Standard Duty	7.0	4.0	7.0	12

Pavement Type	Traffic Index	AC Thickness (inches)	AB Thickness (inches)	Compacted Subgrade (inches)
Heavy Duty	8.0	4.5	8.5	12

AC - Asphaltic Concrete compacted to an average relative compaction of 93 percent with no single test value below a relative compaction of 91 percent and not single test value being above a relative compaction of 97 percent of the reference laboratory design according to AASHTO T209 or ASTM D2041.

AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D-1557)

Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

Note: According to the Guide for Designing Geotextiles, dated April 28, 2009 (California Department of Transportation), the pavement thickness may not be reduced using a geotextile where the R-value is 20 or greater. Thus, an alternative pavement section is not provided for the use of a geotextile for the above sections based on a design R-value of 20.

8.11.6 The Contractor shall verify the thickness of the aggregate base prior to paving. The thickness shall be verified at a minimum frequency of 1 test location per 20,000 square feet of paved area. The test shall consist of excavating the aggregate base and measuring the thickness. The Contractor shall arrange for the Retail Tenant Construction Testing Laboratory (CTL) to observe these tests. The Contractor shall provide these measurements and locations in writing to Retail Tenant. The Contractor shall repair the areas indicating deficiencies at no cost to Retail Tenant.

8.11.7 The completed pavements should be cored at the locations and frequency required by the specifications to verify proper thickness

8.11.8 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.

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

8.11.10 The asphaltic concrete, including 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 AASHTO T209 or ASTM D2041.

8.11.11 At a minimum, it is recommended the asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. The Contractor shall provide mix designs, prepared and signed by a registered civil engineer in the State of California, to the Retail Tenant CEC for review and approval prior to placement of the asphaltic concrete.

8.11.12 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.

8.12 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 550 psi. The design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

8.12.1 The subgrade below Portland cement concrete pavement sections should be prepared as recommended in the Site Preparation section of these recommendations. Any soft or unstable areas identified during compaction should be removed and compacted as engineered fill.

8.12.2 The moisture content of the upper 12 inches of the subgrade soils should be tested and confirmed to be between optimum and three (3) percent above optimum prior to placement of the base section.

8.12.3 The following pavement section thicknesses were determined based on the minimum 20 year design life and the ESAL specified by Retail Tenant for "Standard" and "Heavy Duty" pavements. A design modulus of subgrade reaction k-value of 215 psi/in, was used, considering a recommended 6-inch layer of aggregate base material (minimum R-Value of 78) over the on-site compacted soils. The designs are based on a design life of 20 years. 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 of 3,500 psi concrete, 2) load transfer by aggregate interlock or dowels, 3) with a concrete shoulder or thickened edge, 4) a load safety factor of 1.1, and 5) truck loading consisting of 18 kip equivalent single axle loading as specified in the Retail Tenant criteria. Cross sections depicting the pavement sections are included on Drawing No. 5 in Appendix A of this report.

Pavement Type	PCC Layer Thickness (inches)	AB Layer Thickness (inches)	Compacted Subgrade (inches)
Standard Duty	6.5	6.0	12.0
Heavy Duty	6.5	6.0	12.0

PCC - Portland Cement Concrete (minimum Modulus of Rupture=550 psi)
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

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

8.12.5 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 550 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC slabs.

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

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

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

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

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

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

8.12.12Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

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

8.12.14 Pavement construction should conform to Sections 40 and 90 of the State of California Standard Specifications.

8.13 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 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 1.5H to 1V zone below existing improvements which are to remain after construction. Excavations which are required to be advanced below the 1.5H to 1V envelope should be shored to support the soils, foundations, and slabs.

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

8.13.5 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, Retail Tenant should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.14 Utility Trenches

8.14.1 Based on the shallow depth to groundwater encountered, the Contractor should be aware that dewatering may be required for excavations near or below the depth of groundwater (anticipated to be as high as approximately 10 feet BSG). The contractor should anticipate the need for stabilization and dewatering for utility trench construction near groundwater. A dewatering specification is included in Appendix F of this report.

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 95 percent relative compaction prior to placement of bedding material. 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 recommendations provided in this report include placement of an imported non-expansive engineered fill section below interior and exterior slabs on grade. Thus, utility trench backfill below the slabs will be required to selectively excavate, stockpile and backfill the non-expansive fill such that the non-expansive fill material is replaced in the upper section of the trench backfill to match the thickness of the recommended non-expansive fill. The onsite clay soils or mixtures of the imported, non-expansive fill and clay soils should not be used as fill within 12 inches of interior slabs or 6 inches of exterior slabs.

8.14.4 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The contractor is responsible for contacting the governing agency to determine the requirements for pipe bedding, pipe zone and final backfill. The contractor is responsible for notifying Retail Tenant 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 (95 percent relative compaction) select sand with a minimum sand equivalent of 30 and meeting the following requirements: 100 percent passing the 1 inch sieve, and not more than 10 percent passing the No. 200 sieve. The bottom of the trench should be compacted as engineered fill prior to placement of the pipe bedding. 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 95 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be on-site or imported soils placed as engineered fill. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.14.5 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 95 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (95 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321-00 listed in Table No. 3 (minimum manufacturer requirements). As an alternative to the trench width recommended above and the use of the select sand bedding, a lesser trench width for HDPE pipes may be used if the trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to 1 foot above the top of the pipe.

Table No. 3
Minimum Trench Widths for HDPE Pipe with Select Sand Bedding as Initial Backfill

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

8.14.6 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.

8.14.7 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to within optimum to 3 percent above the optimum moisture content and compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.14.8 Trench backfill should be placed in 8 inch lifts, moisture conditioned to within optimum to three (3) percent above optimum and compacted to achieve the minimum relative compaction requirements. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved.

8.14.9 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. However, rocks greater than 3 inches in any dimension will not be permitted in backfill placed between 1 foot above the top of any pipe and subgrade.

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

8.14.11 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.

8.14.12 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 heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended and the Contractor should be required to video inspect and/or pressure test the pipelines 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 video tape and a written summary of the pipe conditions prepared by the video inspection firm. The Contractor is required to repair all noted deficiencies at no cost to the owner. The Contractor should confirm, in writing, that the deflection of the HDPE pipe is within the requirements of the manufacturer after the pipe is installed and backfilled.

8.14.13 The plans should note that all utility trenches, including electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 95 percent per ASTM D-1557.

8.14.14 Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 1.5 horizontal to 1 vertical downward from the bottom of building foundations.

8.15 Corrosion Protection

8.15.1 Based on the ASTM Special Technical Publication 741 and the analytical results of sample analyses indicate the sample had a resistivity values of 7.3, 7.1, and 7.2; minimum resistivity values of 5,600, 2,400, and 6,100 ohm-centimeters, respectively. Based on the resistivity value, the soils exhibit a "corrosive" to "moderately corrosive" corrosion potential. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.15.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (0.0021, 0.0054, and "none detect" percent by dry weight concentrations of sulfate). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Thus, Type I or II cement may be used for concrete mixes in contact with the subsurface soils.

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 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 provided the opportunity 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.

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 afforded the opportunity 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 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, pavements, etc. 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.

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.

11.2 The nature and extent of subsurface variations between borings may not become evident until construction.

11.3 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.

11.4 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.

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

11.6 The conclusions and recommendations contained in this report are valid only for the project discussed in Section 3.4, 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 Section 3.3, Site Description is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure 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.

11.7 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.

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

11.9 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.

11.10 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 Greenberg Farrow. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

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Dean B. Ledgerwood II
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STATE OF CALIFORNIA
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