DRAFT

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED WALMART STORE NO. 85313

SOUTHWEST CORNER OF

GENTIAN AVENUE AND PERRIS BOULEVARD

MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

Project Number: E40505.01-01

For:

Kimley-Horn & Associates, Inc.
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November 30, 2012

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November 30, 2012 E40505.01-01

Kimley-Horn & Associates, Inc.
765 The City Drive, Suite 200
Orange, CA 92868

Attention: Ms. Jennifer Steen

Subject: **Geotechnical Engineering Investigation**
Proposed Walmart Store No. 85313
Southwest Corner of Gentian Avenue and Perris Boulevard
Moreno Valley, Riverside County, California

Dear Ms. Steen:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed Walmart store to be located at the southwest corner of Gentian Avenue and Perris Boulevard in Moreno Valley, Riverside County, California.

It is our understanding that the following report will be provided to Contractors for informational purposes only; however, the report will not be included as part of the construction contract (construction bid) documents.

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 Kimley-Horn & Associates, Inc. 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.
Geotechnical Engineering Division

DRAFT

Read L. Andersen, RGE
Manager

EXECUTIVE SUMMARY

Moore Twining Associates, Inc. conducted a geotechnical engineering investigation for the proposed Walmart Store #85313 to be located at the southwest corner of Gentian Avenue and Perris Boulevard in Moreno Valley, Riverside County, California.

The project site comprises approximately 21.47 acres. The Walmart parcel includes 20.34 acres of the total site area and Outparcel 1 comprises 1.13 acres. The proposed construction will consist of a Walmart retail store comprising approximately 186,944 square feet in plan area. It is anticipated that the store will consist of a single-story structure with concrete masonry unit (CMU) walls, a combination of steel and wood frame roofing, with concrete slab-on-grade floors. Appurtenant construction is anticipated to include paving, concrete flatwork and underground utilities.

Fills of between about 1½ and 4½ feet are expected to achieve a level building pad for the Walmart store as part of site development.

The project also includes an approximately one-half (½) acre storm water basin and offsite storm drain improvements. The offsite storm drain is anticipated to extend approximately 2,930 lineal feet below Perris Boulevard and Iris Avenue.

At the time of our field exploration, the site was mostly covered by low lying weeds and brush.

Between October 15 and 21, 2012, eighty-five (85) test borings were drilled in the onsite portion of the proposed project in the areas proposed for the Walmart building, garden center, outparcel area, detention basin area and parking and drive areas to depths ranging from 10 feet to 51½ feet below site grade (BSG). Eleven (11) borings were drilled within the footprint of the proposed Walmart store building and garden center to depths ranging from 20 to 51½ feet BSG, and seven (7) Cone Penetration Test (CPT) soundings were conducted to depths of approximately 50 feet. Seventy-four (74) borings were drilled within the proposed parking and drive areas, detention basin and outlot pad to depths ranging from 10 to 31½ feet BSG.

On November 6, 2012, six (6) borings (B-86 through B-91) were drilled within Perris Boulevard and Iris Avenue for the proposed storm drain improvements to depths ranging from 13 to 16½ feet BSG.

The near surface soils encountered generally consisted of medium dense clayey sands extending from the ground surface to depths of about 3½ feet BSG. Loose to medium dense silty sands with varying amounts of gravel generally extended from about 3½ feet BSG to depths of about 5 to 13½ feet BSG. The silty sands were underlain by interbedded layers of poorly graded sands, clayey sands, sandy lean clays, and silty sands with occasional layers of sandy silts extending to the maximum depth explored of 51½ feet BSG. Groundwater was not encountered during our October 2012 investigation.

The near-surface soils exhibit low shear strength, high compressibility characteristics, moderate collapse potential, very low expansion potential and poor to excellent support characteristics for pavements when compacted as engineered fill.

EXECUTIVE SUMMARY

Field and laboratory tests indicated that the near surface silty sands and clayey sands within the upper approximately 8 to 10 feet are loose and have a moderate collapse potential. Accordingly, improvements supported on the existing native soils would be subject to potential excessive collapse settlement. In order to reduce the potential for excessive settlement of foundations due to collapse type soil movements to meet the Walmart foundation settlement requirements, over-excavation and compaction of the upper 8 feet of the near surface soils are recommended in the building pad areas to support foundations on engineered fill.

Based on the results of R-value testing included in this report, an asphaltic concrete pavement section of 4.0 inches of asphalt concrete over 13 inches of aggregate base is recommended for the "standard duty" Walmart pavement section; and 4.5 inches of asphalt concrete over 16.0 inches of aggregate base is recommended for the "heavy duty" Walmart pavement section.

Given that no mapped faults were identified trending through the site, the potential for fault rupture at the site is considered low.

Due to the historic depth to groundwater at the site, liquefaction is not considered a concern. Based on our evaluations, a dry seismic settlement of ¼ inch total and ⅛ inch differential in 40 feet were estimated.

Based on the resistivity values, the soils exhibit a “mildly corrosive” corrosion potential.

According to the 2010 California Building Code, the soluble sulfate concentrations fall in the negligible category. Thus, sulfate attack of concrete in contact with the onsite soils is not anticipated to be a concern.

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

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PURPOSE AND SCOPE OF INVESTIGATION 1
- 2.1 Purpose 1
- 2.2 Scope 2

3.0 BACKGROUND INFORMATION 4
- 3.1 Site Description 4
- 3.2 Site History 5
- 3.3 Previous Studies 5
- 3.4 Anticipated Construction 6

4.0 INVESTIGATIVE PROCEDURES 7
- 4.1 Field Exploration 7
  - 4.1.1 Site Reconnaissance 7
  - 4.1.2 Drilling Test Borings 7
  - 4.1.3 Soil Sampling 9
  - 4.1.4 Cone Penetrometer Testing (CPT) 9
  - 4.1.5 Double-Ring Infiltrometer Testing 10
- 4.2 Laboratory Testing 10

5.0 FINDINGS AND RESULTS 11
- 5.1 Existing Pavement Sections Along Perris Boulevard and Iris Avenue 11
- 5.2 Soil Profile 12
- 53 Soil Engineering Properties 13
- 5.4 Groundwater Conditions 15
- 5.5 Double-Ring Infiltrometer Test Results 16

6.0 EVALUATIONS 17
- 6.1 Surface Conditions 17
- 6.2 Stormwater Infiltration 18
- 6.3 Expansive Soils 19
- 6.4 Collapsible Soils 19
- 6.5 Static Settlement and Bearing Capacity of Shallow Foundations 20
- 6.6 Interior Slab-on-Grade 21
- 6.7 Climatic Conditions 22
- 6.8 Faulting and Seismicity 22
- 6.9 Liquefaction and Seismic Settlement 22
- 6.10 Asphaltic Concrete (AC) Pavements 23
- 6.11 Portland Cement Concrete (PCC) Pavements 24
- 6.12 Corrosion Protection 24
- 6.13 Sulfate Attack of Concrete 25

7.0 CONCLUSIONS 25

TABLE OF CONTENTS

Page

8.0 RECOMMENDATIONS 26
- 8.1 General 26
- 8.2 Site Grading, Landscaping and Drainage 29
- 8.3 Stormwater Basin 29
- 8.4 Site Preparation 31
- 8.5 Engineered Fill 34
- 8.6 Foundations 37
- 8.7 Frictional Coefficient and Earth Pressures 39
- 8.8 Retaining Walls/Screen Walls 40
- 8.9 Interior Slabs-on-Grade 42
- 8.10 Exterior Slabs-on-Grade 45
- 8.11 Asphaltic Concrete (AC) Pavements 46
- 8.12 Portland Cement Concrete (PCC) Pavements 49
- 8.13 Temporary Excavations 52
- 8.14 Utility Trenches 53
- 8.15 Corrosion Protection 56

9.0 DESIGN CONSULTATION 57

10.0 CONSTRUCTION MONITORING 57

11.0 NOTIFICATION AND LIMITATIONS 57

APPENDICES

APPENDIX A - Drawings A-1

- Drawing No. 1 - Site Location Map
- Drawing No. 2 - Test Boring Location Map (for Walmart Site)
- Drawing No. 3 - Test Boring Location Map (for Storm Drain Improvements)
- Drawing No. 4 - Asphaltic Concrete Pavement Sections
- Drawing No. 5 - Portland Cement Concrete Pavement Sections
- Drawing No. 6 - Retaining Wall Drain Detail

APPENDIX B - Logs of Borings, Cone Penetrometer Tests, and Backhoe Pits B-1

APPENDIX C - Results of Laboratory Tests C-1

TABLE OF CONTENTS

Page

APPENDIX D - Results of Double Ring Infiltrometer Testing D-1

APPENDIX E - Summary of Geotechnical Design Parameters;
Geotechnical Investigation Fact Sheet; Foundation Design Criteria;
and Foundation Subsurface Preparation E-1

APPENDIX F - Pavement Design Calculations F-1

APPENDIX G - Photographs G-1

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED WALMART STORE NO. 85313

SOUTHWEST CORNER OF

GENTIAN AVENUE AND PERRIS BOULEVARD

MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

Project Number: E40505.01-01

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed Walmart store No. 85313 to be located at the southwest corner of Gentian Avenue and Perris Boulevard in Moreno Valley, Riverside County, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Kimley-Horn & Associates, Inc. to conduct this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided for this project. The site history, previous studies, existing site features, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices include the drawings (Appendix A), the logs of borings, cone penetration tests and backhoe pits (Appendix B), the results of laboratory tests (Appendix C), the results of double-ring infiltrometer tests (Appendix D), the Geotechnical Investigation Fact Sheet; Foundation Design Criteria; and Foundation Subsurface Preparation (Appendix E), Pavement Design Calculations (Appendix F), and Photographs (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 geotechnical engineering investigation was to conduct a field exploration, a laboratory testing program, evaluate the data collected during the field and laboratory portions of the investigation, and provide the following:

2.1.1 Evaluation of the near surface soils in the areas of the proposed improvements based on the Walmart's Geotechnical Investigation Specifications and Report Requirements, dated September 25, 2012;

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

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

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

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

2.1.6 Assessment of the infiltration characteristics of the soils in the area of the proposed detention basin based on subsurface investigation and two (2) double-ring infiltrometer tests;

2.1.7 Recommendations for the proposed offsite storm drain;

2.1.8 Recommendations for temporary excavations and trench backfill; and

2.1.9 Conclusions regarding soil corrosion potential.

This investigation did not include a floodplain investigation, in-place density tests, environmental investigation, or environmental audit. This investigation did not include recommendations for offsite improvements other than the offsite storm drain described in this report.

2.2 **Scope:** Our proposal, dated September 12, 2012, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A site plan (CP-5) for the proposed project, prepared by Nasland Engineering, dated August 30, 2012, was reviewed. This plan is referred to, hereinafter, as the Site Plan.

2.2.2 An undated topographic map, prepared by Canyon Consulting emailed to our firm on November 1, 2012, was reviewed. This plan is referred to, hereinafter, as the topographic plan.

2.2.3 An ALTA/ACSM Land Title Survey, dated July 31, 2010, prepared by O.K.O. Engineering, Inc., was reviewed.

2.2.4 The Walmart Geotechnical Investigation Specifications and Report Requirements, dated September 25, 2012, was reviewed.

2.2.5 The 2003 Riverside County General Plan and the Riverside County Land Information System were reviewed.

2.2.6 A site reconnaissance, subsurface exploration and infiltration tests were conducted.

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

2.2.8 Ms. Kathleen Caldwell and Ms. Jennifer Steen (Kimley-Horn & Associates, Inc.) were consulted during the investigation.

2.2.9 The following representatives: Mr. Leon Alkire (Eastern Municipal Water District), Mr. Bruno Chavez (Utiliquest) and Mr. John Robinson (Department of Water Resources) were consulted regarding the existing utility services on and near the site.

2.2.10 An encroachment permit (Encroachment Permit No. EP2012-0275) was obtained from the City of Moreno Valley for the offsite investigation work.

2.2.11 A report entitled, "Preliminary Geotechnical Investigation, Proposed 104-Acre Residential Development, Northwest of Perris Boulevard and Iris Avenue, City of Moreno Valley, California, dated June 9, 2004, prepared by Leighton and Associates, Inc., was reviewed. In addition, a geotechnical engineering investigation report for the commercial development located about 500 feet south of the subject site, prepared by our firm, dated May 15, 2003, was reviewed.

2.2.12 A report entitled, "Phase I Environmental Site Assessment, Walmart Store No. 4059-00, Moreno Valley, Riverside County, California," dated November 8, 2012, prepared by Kimley-Horn and Associates, Inc. was reviewed.

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

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

3.1 Site Description: The project site comprises approximately 21.47 acres located northwest of the intersection of Perris Boulevard and Santiago Drive in Moreno Valley, Riverside County, California. Based on our review of the site plan (CP-5) for the proposed project, prepared by Nasland Engineering, dated August 30, 2012, the Walmart parcel includes 20.34 acres of the total site area, and Outparcel 1 comprises 1.13 acres. The site is bound to the north by proposed Gentian Avenue with vacant land and residential development beyond, to the east by Perris Boulevard and residential property beyond, to the south by Santiago Drive and vacant property and a Home Depot store development beyond, and to the west by vacant land and residential development beyond. Overhead power lines were noted along the eastern and southern property boundaries. A site location map is presented on Drawing No. 1 in Appendix A.

At the time of our field exploration, the site was mostly covered by low lying weeds and brush. However, an area, approximately 60 to 80 feet in width trending in an approximately north-south or east-west orientation, had been cleared of most of the vegetation. During our field exploration, an adjacent road improvement project including widening of Perris Boulevard, which borders the east side of the site, was under construction.

A pad mounted, high voltage vault for S&C Electric Company was noted in the southeast corner of the site near the new curb and gutter for Perris Boulevard. A pile of concrete and brick debris, approximately 15 feet by 15 feet in plan dimensions, was noted in the southeast portion of the site. In addition, just south of this debris pile was an excavated area that appeared to be a concrete washout pit that was about 1 to 2 feet deep and 15 feet by 15 feet in plan dimensions. The apparent concrete washout pit included a cementitous material over a layer of black plastic.

A 100-foot wide easement for the California Aqueduct trends in a northwest-southeast orientation and lies beyond and parallel to the southwestern property boundary. Based on our discussions with Mr. John Robinson with the Department of Water Resources, the easement contains a 9-foot diameter, high pressure, steel water line with concrete encasement which is about 6 to 8 feet below site grades within the middle of the 100-foot wide easement. At the time of our field exploration, the approximate location of the alignment of the underground aqueduct was delineated with blue flagging along the ground surface. Based on our site observations and our review of the ALTA/ACSM Land Title Survey, dated July 31, 2010, prepared by O.K.O. Engineering, Inc., three (3) manholes are located within the eastern portion of the 100-foot wide easement.

The site slopes gently to the southeast. An undated topographic map, prepared by Canyon Consulting, indicates that ground surface elevations generally range from about 1,509 feet above mean sea level (AMSL) in the southeastern portion of the site to about 1,519 feet AMSL in the northwestern portion of the site.

3.2 <u>Site History</u>: A report entitled, "Phase I Environmental Site Assessment, Walmart Store No. 4059-00, Moreno Valley, Riverside County, California," dated November 8, 2012, prepared by Kimley-Horn and Associates, Inc., was reviewed. Based on our review of the Phase I Environmental Site Assessment (Phase I ESA), the site was used for agricultural purposes as early as 1938. The Phase I ESA also reported that aerial photographs suggested that agricultural activities were suspended sometime between 1977 and 1989, and the site has remained undeveloped since that time. Aerial photographs for the years 1938, 1953, 1967, 1977, 1989, 2002, 2005, and 2006 were included in the Phase I ESA. Review of the 1953 aerial photograph indicated a rectangular shaped depression, approximately 10,000 to 15,000 square feet in plan dimension, located in the southwestern-most portion of the site (adjacent to Santiago Drive) which appeared to be used for water storage. The approximate location of this feature is depicted on Drawing No. 2 in Appendix A of this report. The depression does not appear in the 1977 aerial photograph. Other than a possible structure, trailer or equipment near the east property line present in the 1989 aerial photograph, no significant changes were noted in aerial photographs since the 1977 photograph.

3.3 <u>Previous Studies</u>: A report entitled, "Preliminary Geotechnical Investigation, Proposed 104-Acre Residential Development, Northwest of Perris Boulevard and Iris Avenue, City of Moreno Valley, California, dated June 9, 2004, prepared by Leighton and Associates, Inc. (Leighton), was reviewed. The investigation was conducted for a 104-acre site that included the subject site and additional areas west and south of the subject site. The investigation included eight (8) borings that extended to depths ranging from 21½ to 51½ feet below site grades (BSG) and eight (8) backhoe test pits that extended to a maximum depth of 5½ feet BSG. The alluvial soil deposits encountered by Leighton were described as loose to medium dense silty sand to gravelly sand and soft to stiff sandy silt. Below a depth of 15 feet, the soil was generally described as consisting of stiff to very stiff sandy silt to clay. Groundwater was not encountered in any of Leighton's borings to the maximum depth explored, about 51½ feet BSG. The report indicated that the upper alluvial soils were slightly to moderately compressible, have negligible to minor hydrocollapse potential (1 percent or less), and had very low expansion potential. Leighton's analysis indicated that the potential for liquefaction appears to be low; however, seismic settlements of up to about 1½ inches total and ¾ inch differential over 40 feet were estimated. The report recommended over-excavation and recompaction of the site soils to a minimum of 3 feet below the bottom of proposed foundations for residential structures. Static foundation settlement estimates of 1 inch total and ½ inch differential over a horizontal distance of 30 feet were listed in the report.

A geotechnical engineering investigation report for the commercial development located about 500 feet south of the subject site, prepared by Moore Twining Associates, Inc. (formerly The Twining Laboratories, Inc.), dated May 15, 2003, was also reviewed. The native near surface soils predominantly consisted of silty sands and sandy silts with layers of poorly graded sands extending to depths of 13 to18 feet BSG. The silty sands exhibited moderate to high compressibility characteristics with the addition of moisture and low to high collapse potential. The soils with collapse potential appeared to extend to depths of about 7 feet BSG. One (1) expansion index test on a near surface sample indicated a very low expansion potential. Based on the collapse potential of the near surface soils, the report recommended over-excavation to at least 7 feet below preconstruction site grades. Static settlements of 1 inch total and ½ inch differential over 50 linear feet were estimated based on the recommended site preparation and an allowable bearing pressure

of 2,500 pounds per square foot for foundations. Groundwater was not encountered in test borings drilled to a maximum depth of exploration of about 50 feet BSG at the time of the field investigation (January and February 2003). Seismic settlements of ½ inch total and ¼ inch differential were estimated.

Moore Twining was not provided any other previous geotechnical engineering or environmental reports for the subject site to review during this investigation. If other reports for geotechnical engineering, geological, or environmental studies conducted for this site become available, these reports should be provided for review and consideration for this project.

3.4 **Anticipated Construction**: Based on the information shown on site plan CP-5, prepared by Nasland Engineering, dated August 30, 2012, it our understanding that the proposed construction will consist of a Walmart retail store comprising approximately 186,944 square feet in plan area. The Walmart store is proposed to be located in the northern portion of the site, at the southwest corner of Gentian Avenue and Perris Boulevard. It is anticipated that the proposed construction will consist of a single-story structure with concrete masonry unit (CMU) walls, a combination of steel and wood frame roofing, with concrete slab-on-grade floors. A garden center is planned on the west side of the store. Depressed loading docks, a trash compactor, bale & pallet recycling area, and an organic bin area are planned on the north side of the store. A detention basin comprising approximately one-half (½) acre is planned in the southern portion of the site, which is anticipated to extend to depths of between approximately 6 and 10 feet below existing site grades. Outparcel 1 is to be located in the southeast portion of the site. The improvements planned in Outparcel 1 are not known at this time. For the purpose of this report, it was assumed that Outparcel 1 would be developed with a lightly loaded single-story structure such as a restaurant or a small retail building and associated paving. No retaining walls are identified on the plan. However, it is our understanding that a screen wall is planned along the north and northeast edges of the property boundary. Appurtenant construction is anticipated to include asphaltic and concrete pavements, underground utilities, and isolated landscape areas.

Building construction information and structural loads are summarized below based on the Walmart Geotechnical Investigation Specifications and Report Requirements. It is anticipated that the store building will be a combination of load bearing concrete masonry unit walls and steel columns supporting roof loads by means of steel joist girders and steel bar joists. A maximum temporary column load of 150 kips is anticipated due to severe live loading; however, interior and exterior column loads of 85 kips and 50 kips, respectively, are typical. The typical bay spacing between columns and walls is 48 feet by 55 feet. The maximum column uplift force from wind is estimated to be 30 kips. Gravity wall loads ranging from 1.5 to 2.0 kips per lineal foot are anticipated for non load-bearing walls, and loads of 4.0 to 6.0 kips per lineal foot are anticipated for load bearing walls. The maximum uniform floor slab load will be 125 pounds per square foot, and the maximum concentrated floor slab load will be 5 kips. The maximum allowable total movement is 1 inch. The maximum allowable differential settlement for masonry walls is 0.53 inch in 40 lineal feet of wall. The maximum allowable differential settlement for interior slabs or interior isolated footings is 0.96 inches in 40 feet. Seismic settlements are assumed to be in addition to the allowable (static) settlements previously listed in this report.

A grading plan was not available at the time this report was prepared. The site plan indicates the Walmart store will have a finished floor elevation of 1,518 feet above mean sea level. Based on our review of the topographic map, elevations in the Walmart building pad area range from about 1,513½ feet AMSL in the south-central portion of the proposed building footprint to about 1,516½ feet AMSL in the north-central portion of the proposed building footprint. Thus, it is anticipated that fills ranging from about 1½ to 4½ feet may be required to construct a level building pad as part of site development. Cuts of up to about 6 to 10 feet below site grades are anticipated for the detention basin.

It is our understanding that offsite storm drain improvements associated with the proposed project are also planned. The planned storm drain improvements include a new storm drain line below Perris Boulevard and Iris Avenue extending a total length of approximately 2,930 lineal feet. The new storm drain will extend approximately 1,360 lineal feet from the intersection of Perris Boulevard and Santiago Drive to the intersection of Perris Boulevard and Iris Avenue, then continuing along Iris Avenue approximately 1,570 lineal feet from the intersection of Perris Boulevard and Iris Avenue to the intersection of Iris Avenue and Wedow Drive. At the time of preparation of this report, the depth of the storm drain was not known.

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 performed at the subject site consisted of a site reconnaissance, drilling test borings, soil sampling, conducting standard penetration tests, conducting cone penetration tests and performing double ring infiltrometer tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The site reconnaissance was conducted by Mr. Allen Harker (Moore Twining project geologist) on October 18, 2011. The features noted at the subject site are described in the background information section of this report.

4.1.2 Drilling Test Borings: The depths and locations of test borings drilled for the Walmart store and associated on-site improvements were selected based on the size of the structure, type of construction, anticipated foundation loads, subsurface soil conditions, the past uses of the site which were known at the time of the investigation and Walmart's Geotechnical Investigation Specifications and Report Requirements. The depths and locations of the test borings drilled for the offsite storm drain were provided by Kimley-Horn and Associates, Inc.

Between October 15 and 21, 2012, eighty-five (85) test borings were drilled for this investigation in the areas proposed for the Walmart building, garden center, outparcel area, detention basin area and parking and drive areas to depths ranging from 10 feet to 51½ feet below site grade (BSG). Eleven (11) borings were drilled within the footprint of the proposed Walmart store building and garden center to depths ranging from 20 to 51½ feet BSG, and Cone Penetration Test (CPT) soundings were conducted in place of the remaining soil borings planned for the building area (see Section 4.1.4 of this report). Six (6) borings were drilled within the outparcel area to depths ranging

from 10 to 15 feet BSG, six (6) borings were drilled in the proposed detention basin area to depths ranging from 10 to 31½ feet BSG, and sixty-two (62) borings were drilled within the proposed parking and drive areas on a 100-foot grid spacing to depths of 10 feet BSG. It should be noted that a few of the parking lot borings were relocated slightly to stay at least 50 feet outside the easement for the California Aqueduct as requested by the Department of Water Resources.

Based on our geotechnical experience in the site area and the subsurface soil conditions encountered, a 100-foot deep boring was not considered necessary to determined the site class.

On November 6, 2012, six (6) borings (B-86 through B-91) were drilled within Perris Boulevard, between Santiago Drive and Iris Avenue, and within Iris Avenue, between Perris Boulevard and Wedow Drive for the proposed storm drain improvements. An encroachment permit (EP2012-0275) was obtained from the City of Moreno Valley to drill the borings within Perris Boulevard and Iris Avenue. Prior to drilling the soil borings, the asphalt concrete was cored using a coring barrel with diamond impregnated cutting teeth. The borings were drilled to depths ranging from 13 to 16½ feet BSG.

The test boring and bulk sample locations are shown on Drawing No. 2 in Appendix A. The test borings were drilled using a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow-stem augers. The test borings were drilled under the direction of a Moore Twining project geologist. The soils encountered in the test borings were logged during drilling. The field soil classification was in general 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 borings. In addition, boring B-35 that was drilled to 51½ feet BSG was dry after being left open for 24 hours.

Test boring locations were determined by measuring wheel and by pacing with reference to the property corners and building corner reference stakes set by Canyon Consulting. The locations of the borings for the Walmart store, outparcel area, and detention basin area are shown on Drawing No. 2 in Appendix A. The locations of the borings drilled along Perris Boulevard and Iris Avenue for the proposed storm drain improvements are shown on Drawing No. 3 in Appendix A. In addition, the latitude and longitude of the test borings are noted in Appendix B. The elevations were estimated from topographic data provided by Canyon Consulting. Test boring locations should be considered accurate to within about 10 feet.

The test borings on the Walmart site were loosely backfilled with soil cuttings; thus, some settlement should be anticipated. The test borings along Perris Boulevard and Iris Avenue were backfilled with neat cement to the bottom of the asphaltic concrete pavement sections, the asphalt cores were placed back to cap the holes based on the requirements of the City of Moreno Valley. The annular space around the cores was filled with emulsified asphalt. The backfill of the boreholes along Perris Boulevard and Iris Avenue were observed by an inspector with the City of Moreno Valley Department of Public Works.

4.1.3 Soil Sampling: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained during drilling. In each boring, four (4) samples were collected in the upper 10 feet at approximate 2.5-foot intervals and at 5-foot intervals thereafter. Shelby tube samples were not collected during the field exploration since the soils encountered were primarily granular in nature and due to the stiff to hard nature of the lean clays encountered at depths of 8 ½ feet BSG or deeper.

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 portions of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing. Soil samples from this investigation will be stored at our laboratory for up to six (6) months.

4.1.4 Cone Penetration Testing (CPT): CPTs were conducted as part of the field exploration for use in analysis of dry seismic settlement in the Walmart building area. On October 17, 2012, seven (7) CPTs were advanced within the proposed Walmart building to depths of about 50 feet BSG. Five (5) of the seven (7) CPT soundings were conducted in place of soil borings within the Walmart building pad area, and two (2) of the seven (7) CPT soundings were conducted adjacent to soil borings in the Walmart building pad area for correlation with physical soil samples. The CPTs were conducted under the direction of a Moore Twining project geologist. CPT methods were used to obtain generally continuous soil behavior type and penetration resistance information.

The CPTs were performed by Middle Earth Geo Testing, Inc. using an electronic piezocone with a 60-degree apex angle and a diameter of 35.7 millimeters (about 1½ inches). The CPTs were hydraulically advanced using a 30-ton CPT rig in accordance with ASTM Test Method D3441. CPT measurements of cone tip resistance, sleeve friction, and N-values data were recorded at 1/6 foot intervals during penetration to provide nearly continuous logs of the soil behavior types. The CPT logs are presented in Appendix B following the boring logs and test pit logs.

The CPTs were located by pacing from building corner stakes set by Canyon Consulting. The CPT locations shown on Drawing No. 2 (Appendix A) should be considered accurate to within about 10 feet.

4.1.5 Double-Ring Infiltrometer Testing: Two (2) double-ring infiltrometer tests were conducted in the area of the proposed detention basin in the southwest portion of the site. It is our understanding that the detention basin is anticipated to be excavated to depths of about 6 to 10 feet below site grades. In order to conduct the double-ring infiltrometer tests, backhoe pits were excavated on October 19, 2012 to depths of 8.8 feet and 10.5 feet BSG and pre-soaked with water. The locations of the test pits and double-ring infiltrometer tests (I-1 and I-2) are shown on Drawing No. 2 in Appendix A. The soils encountered in the backhoe pit excavations were logged. The field soil classification was in general accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The testing was performed in general accordance with ASTM D3385 "Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometers" on October 20, 2012. The test apparatus consisted of a one-foot diameter inner ring and a two-foot diameter outer ring which was seated into the soils at the bottom of the excavation. The purpose of the double ring configuration was to measure the infiltration rate in the vertical direction under a virtually constant water head. Therefore, the test apparatus was configured so the infiltration of the outer ring would constrain the infiltration of the inner ring to the vertical component. The infiltration rate was determined by recording the volume of water added to the inner ring to maintain a constant head. The results of the double-ring infiltrometer tests are summarized in Section 5.5 of this report.

Following the completion of the double-ring infiltrometer testing, the test pits were loosely backfilled with the excavated soils.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of the soils encountered. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface soils encountered. Unconfined compressive strength tests were not conducted due to the predominantly granular nature of the near surface soils and since sampling the stiff to hard lean clay soils were not practical with a Shelby tube. In addition, given that the on site near surface soils are granular in nature and graded slopes are not planned (other than the detention basin), strength testing by direct shear test methods were conducted for the project and triaxial compression tests were not conducted to assess the strength characteristics of the soils.

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 of this report.

5.0 FINDINGS AND RESULTS

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

5.1 Existing Pavement Sections Along Perris Boulevard and Iris Avenue: The existing pavement section thicknesses encountered along Perris Boulevard and Iris Avenue varied. The pavement section thicknesses encountered are summarized in Table No. 1.

Table No. 1
Pavement Sections Encountered Along Perris Boulevard and Iris Avenue

Boring Number and Street	Total Asphaltic Concrete (AC) Thickness[5]	Aggregate Base (AB) Thickness[6]	Subgrade Soils
B-86 Iris Avenue	8 inches[1]	14 inches	Silty Sand with trace gravel
B-87 Iris Avenue	4¾ inches	18 inches	Silty Sand
B-88 Iris Avenue	4 3/4 inches	5 inches	Silty Sand with trace clay
B-89 Perris Boulevard	10⅛ inches[2]	No AB	Silty Sand with trace clay
B-90 Perris Boulevard	6 inches[3]	6 inches	Silty Sand with trace clay
B-91 Perris Boulevard	6 inches[4]	7 inches	Silty Sand with trace clay

Notes:
1. Pavement fabric encountered at a depth of 2½ inches
2. Pavement fabric encountered at a depth of 2⅝ inches
3. Pavement fabric encountered at a depth of 4 inches
4. Pavement fabric encountered at a depth of 2 inches
5. The asphaltic concrete thickness was measured to the nearest ⅛ inch
6. The aggregate base thickness was measured to the nearest ½ inch.

5.2 Soil Profile: The near surface soils encountered generally consisted of medium dense clayey sands extending from the ground surface to about 3½ feet BSG. Loose to medium dense silty sands with varying amounts of gravel generally extended from 3½ feet BSG to depths of about 5 to 13½ feet BSG. The silty sands were underlain by interbedded layers of poorly graded sands, clayey sands, sandy lean clays, and silty sands with occasional layers of sandy silts extending to the maximum depth explored of 51½ feet BSG. A thin (6 inch) fill soil consisting of silty sand with concrete and asphalt debris was encountered in two (2) borings (B-13 and B-27) that were drilled in the northeast portion of the site and the southeast portion of the proposed detention basin.

Based on the five (5) borings drilled in the detention basin area (B-27, B-28, B-46, B-47 and B-48), the near surface soils consisted of silty sands with varying amounts of gravel extending to depths of about 8½ to 9½ feet BSG. In boring B-45, the near surface silty sands were underlain by sandy lean clays from depths of 8½ feet to 30 feet BSG with an interbedded silty sand layer between the depths of 15 and 16 feet BSG. The lean clays were underlain by silty sands to the maximum depth explored, 31½ feet BSG. In boring B-46, the near surface silty sands were underlain by sandy lean clays extending from depths of 8½ feet to 19 feet BSG, which were underlain by silty sands extending from 19 feet to 28½ feet BSG, which were underlain by sandy lean clays extending to the maximum depth explored, about 30 feet BSG.

The test pits excavated for the double-ring infiltrometer tests generally indicated a similar soil profile as noted above (see test pits logs following the boring logs in Appendix B). Test Pit TP-1 excavated for infiltration test I-1 encountered clayey sands extending to a depth of about 4 feet BSG, which were underlain by silty sands extending to a depth of about 10½ feet BSG. Following the infiltration test, the bottom of the test pit was hand augered down to 14 feet BSG and revealed the clayey sand soils extending from 10½ to 13½ feet BSG and a sandy lean clay layer was encountered between depths of 13½ and 14 feet BSG.

Test pit TP-2 excavated for infiltration test I-2 encountered silty sand fill soils with some metal debris extending to a depth of about 1 foot BSG. The native soils consisted of clayey sands extending from depths of about 1 foot to 3 feet BSG, which were underlain by silty sands extending from 3 to 7½ feet BSG. A poorly graded sand layer with trace fine gravel was encountered between the depths of about 7½ and 9 feet BSG. A sandy lean clay layer was encountered from 9 to 9½ feet BSG.

The soils encountered below the existing pavements within Iris Avenue, between Perris Boulevard and Wedow Drive (see boring logs B-86 through B-88 in Appendix B) generally consisted of loose silty sands with varying amounts of clay and gravel over interbedded layers of loose to medium dense poorly graded sands, medium dense silty sands, medium stiff sandy silts, and stiff to very stiff lean clays to sandy lean clays extending to the maximum depth explored, about 16½ feet BSG.

The soils encountered below the existing pavement sections within Perris Boulevard, between Santiago Drive and Iris Avenue (see boring logs B-89 through B-91 in Appendix B) generally consisted of very loose to loose silty sands with trace clay overlying medium dense poorly graded sands or medium stiff to very stiff lean clays to sandy lean clays extending to the maximum depth explored, about 15 feet BSG.

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

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

Silty Sands: The silty sands encountered in the borings were very loose to dense, as indicated by standard penetration resistance, N-values, ranging from 3 to 43 blows per foot. The silty sands were generally loose between the depths of about 3½ and 8½ feet BSG. Below depths of about 8½ feet BSG, the silty sands were generally medium dense to dense. The moisture contents of the soils tested in the upper 6½ feet ranged from 2 to 5 percent. Nine (9) in-place densities determined from relatively undisturbed samples revealed dry densities of 99.6, 104.5,105.7, 108.1, 109.5, 110.2, 110.4, 111.2, and 117.0 pounds per cubic foot. Sieve analyses of the near surface silty sands indicated a fines content (silt and clay content) ranging from19.8 to 42.5 percent passing the No. 200 sieve. Deeper silty sand soils tested indicated 33 to 34.8 percent passing the No. 200 sieve. Four (4) consolidation tests within the upper 10 feet BSG indicated that the silty sands have high compressibility characteristics. Upon inundation (wetting), the samples exhibited moderate collapse potential (2.6, 2.7, 2.8 and 3.8 percent collapse under a load of 2 kips per square foot). Two (2) direct shear tests conducted on near surface silty sand samples indicated internal angles of friction of 25 and 27 degrees with 120 and 190 pounds per square foot of cohesion, respectively.

Poorly Graded Sands and Poorly Graded Sands with Silt: The poorly graded sands and poorly graded sands with silt encountered were loose to medium dense as indicated by standard penetration resistance, N-values, ranging from 6 to 24 blows per foot. The moisture contents of the poorly graded sands and poorly graded sands with silt ranged from about 1 to 2 percent. A sieve analysis conducted on a poorly graded sand with silt sample indicated 5.3 percent passing the No. 200 sieve.

Clayey Sands: The surface clayey sands encountered were generally medium dense, as indicated by standard penetration resistance, N-values, ranging from 11 to 30 blows per foot. One (1) near surface clayey sand sample was loose, as indicated by standard penetration resistance, N-value, of 9 blows per foot. The moisture contents of the clayey sand samples ranged from about 2 to 8 percent. Two sieve analyses and four (4) wash analyses conducted on clayey sand samples indicated

from 28 to 48 percent passing the No. 200 sieve. Atterberg Limits tests conducted on the same six (6) clayey sand samples indicated liquid limits ranging from 28 to 46 and plasticity indices ranging from 15 to 26. Two (2) expansion index (E.I.) tests indicated a very low expansion potential (E.I. of 14 and 17). One consolidation test conducted on a clayey sand sample collected at depths of between 8½ and 10 feet BSG indicated high compressibility characteristics. Upon inundation (wetting), the clayey sand sample exhibited a moderate collapse potential (2.7 percent collapse under a load of 2 kips per square foot).

Sandy Lean Clays, Lean Clays with Sand and Lean Clays: The sandy lean clays, lean clays with sand, and lean clays were stiff to hard as indicated by standard penetration resistance, N-values, ranging from 11 to 44 blows per foot. The moisture contents of the sandy lean clays and lean clays with sand ranged from 8 to 12 percent. Two (2) in-place densities determined from relatively undisturbed samples revealed dry densities of 101.4 and 105.2 pounds per cubic foot. Atterberg Limits tests conducted on a lean clay sample with sand and a lean clay sample indicated liquid limits of 47 and 41 and a plasticity indices of 28 and 22, respectively. Another Atterberg Limits test conducted on a sandy lean clay sample indicated a liquid limit of 42 and a plasticity index of 24.

R-Value Tests: R-value tests were conducted on five (5) near surface soil samples for the parcel for retail development. The clayey sand samples tested indicated R-values of 14 and 18. The results of tests on silty sands with clay revealed R-values of 18 and 34. The silty sand samples tested indicated R-values of 57 and 63.

R-value tests were also conducted on four (4) near surface soil samples from borings drilled along Perris Boulevard and Iris Avenue. A silty sand sample with fine gravel collected from a boring along Iris Avenue (near Perris Boulevard) indicated an R-value of 75. The other three (3) samples, consisting of either lean clays or sandy lean clays, from borings drilled along the eastern portion of Iris Avenue (near Wedow Drive) or along Perris Boulevard, and indicated R-values of 11, 12 and 24.

Moisture/Density Relationships: One (1) maximum density/optimum moisture determination test was conducted on a clayey sand sample collected from the ground surface to a depth of about 3½ feet BSG. The results of the tests indicated a maximum dry density of 135.8 pounds per cubic foot and an optimum moisture of 6.5 percent.

Chemical Tests: The results of chemical analyses conducted on three (3) silty sand samples indicated resistivity values of 21,000; 19,000; 13,000 ohm-centimeters, and pH values of 7.6, 7.6 and 7.4, respectively. In addition, the results of soil sample analyses indicated none-detected percent by dry weight concentrations of chloride and none-detected, 0.00073, and 0.00068 percent by weight concentrations of sulfate.

Topsoil Analysis: Topsoil tests were performed in accordance with the Walmart requirements on a composite of five (5) samples collected throughout various portions of the site between the ground surface and a depth of about 4 inches below site grades. The particle size analysis was reported based upon the methodology described in ASTM D5268, "Standard Specification for Topsoil Used for Landscaping Purposes." Based on the results of the tests, the materials do not meet the requirements of Table No. 2 of ASTM D5268. The results of the topsoil tests are summarized in Table No. 2.

Table No. 2
Results of Topsoil Testing

Topsoil Test	Test Result
Percentages of sand, silt, clay and organic matter passing the No. 10 sieve	SAND = 68.2% SILT = 23.5% CLAY = 6.6% ORGANICS = 1.7%
Percentage of deleterious material (Percent greater than No. 10 Sieve as defined by ASTM D5268)	13.5%
pH	6.7
Mineral content	98.3%
Micro and macro nutrient content	See specific results in Appendix C

Note: Percentages of sand, silt and clay are reported in accordance with ASTM D5268.

5.4 <u>Groundwater Conditions</u>: Groundwater was not encountered in the test borings drilled during our October 2012 investigation. Boring B-35, which was drilled to a depth of 51½ feet BSG, was free of groundwater after being left open for 24 hours. The Department of Water Resources website indicates a well about 2.5 miles northeast of the site had groundwater levels generally ranging from about 100 feet below the ground surface to about 160 feet below the ground surface between the years 1951 and 1986.

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

5.5 Double-Ring Infiltrometer Test Results: The results of the double-ring infiltrometer tests are summarized in Table No. 3. The double-ring infiltrometer tests were conducted at depths of 10.5 feet and 8.8 feet, where clayey sands and sandy lean clays were encountered, respectively.

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

Table No. 3

Results of Double-Ring Infiltrometer Testing		
Test Location / Depth	Field Infiltration Rate [1] (in./hr)	Soil Type
I-1 at 10.5 feet BSG	2	Clayey Sand
I-2 at 8.8 feet BSG	Less than 0.25	Sandy Lean Clay

BSG - Below site grade
1 - Includes no factor of safety

Laboratory tests conducted on samples collected at depths of between 10½ and 11 feet BSG within test pit TP-1 (depth where soils were tested for infiltration at location I-1) included a sieve analysis and Atterberg Limits. The results of the tests indicated 33 percent passing the No. 200 sieve, a liquid limit of 30 and a plasticity index of 15. The test results classify the soil as a clayey sand.

Laboratory tests on samples collected at depths of between 9 and 9½ feet BSG within test pit TP-2 (depth where soils were tested for infiltration at location I-2) included a sieve analysis and Atterberg Limits. The results of the tests indicated 62 percent passing the No. 200 sieve, a liquid limit of 42 and a plasticity index of 24. The test results classify the soil as a sandy lean clay.

6.0 EVALUATIONS

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 conditions determined from the field 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. Moore Twining should review the conceptual and final grading plans (when available). Proposed grading which varies from that assumed for the purpose of this investigation could significantly impact our evaluations and recommendations for this site. The evaluations performed for this investigation are summarized below.

Based on the findings of this investigation, the primary geotechnical concerns at this site are the potential for excessive total and differential settlements of shallow foundations due to the loose, collapsible nature of the near surface soils.

6.1 Surface Conditions: At the time of our October 2012 field investigation, the site was covered with weeds and other low lying vegetation. Removal of the vegetation and organics will be an important part of site preparation.

In general, stripping and removal of vegetation and roots should be conducted to remove all root structures about 1/4 inch in diameter and greater and all soils with greater than 3 percent organics by dry weight. Based on the conditions encountered, a stripping depth of 4 inches would be anticipated to remove the root structures.

A thin (6 inch) fill soil consisting of silty sand with concrete and asphalt debris was encountered in two (2) borings (B-13 and B-27) that were drilled in the northeast portion of the site and the southeast portion of the proposed detention basin. Test pit TP-2, excavated in the southeast portion of the proposed detention basin area, encountered about 1 foot of fill soil consisting of silty sand. As part of site preparation, existing fills should be over-excavated and compacted as engineered fill.

A pile of concrete and brick debris, approximately 15 feet by 15 feet in plan dimensions, was noted in the southeast portion of the site. In addition, just south of this debris pile was an excavation about 1 to 2 feet deep and 15 feet by 15 feet in plan dimension that appeared to be a concrete washout pit. The apparent concrete washout pit was partially filled with a cementitious material over a layer of plastic. In addition, undocumented fills may be present in an area of an apparent former pond/water storage area within the southwest portion of the site. The approximate location of the washout pit and the apparent former pond shown in the 1953 aerial photograph are depicted on Drawing No. 2 in Appendix A of this report. Due to the historic agricultural use of the site, burn pits, debris pits, undocumented fills, etc. may be encountered during grading and construction. If encountered, these features should be removed and the excavations backfilled as engineered fill.

As part of the site preparation, all undocumented fills should be over-excavated and compacted as engineered fill. In addition, all trash and debris, including the washout pit materials, should be removed from the site in a legal manner.

6.2 <u>Stormwater Infiltration</u>: The infiltration tests conducted in the proposed detention basin area indicated infiltration rates of 2 inches per hour within the clayey sand soils and less than ¼ inch per hour within the sandy lean clay soils. The test results and the soil types encountered are not considered favorable for infiltration of stormwater.

Group "C" type soils are defined by the United States Department of Agriculture, Natural Resources Conservation Service, as soils typically having between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam and silty clay loam textures. Group "D" type soils are defined as soils typically having greater than 40 percent clay, less than 50 percent sand, and have clayey textures. The soils tested at the location of double-ring infiltrometer test I-2 classify the soil as a sandy lean clay, which would categorize these soils as either a Group "C" or "D" type soil. Similar soils were encountered at the location of infiltrometer test I-1 below the clayey sand layer tested. The Riverside County Stormwater Quality Best Management Practice Design Handbook, dated July 21, 2006, indicates that the use of infiltration basins are not appropriate for sites with "C" or "D" type soils. Accordingly, the underlying sandy lean clays do not meet the criteria for infiltration basins listed in the referenced Riverside County document.

The Riverside County Stormwater Quality Best Management Practice Design Handbook also lists a required minimum infiltration rate of 0.5 inches per hour as one of the design criteria for infiltration basins and a minimum infiltration rate of 0.27 inches per hour as one of the criteria for infiltration trenches. The infiltration rate of double-ring infiltrometer test I-2 was less than 0.25 inches and included no factor of safety. Accordingly, the infiltration characteristics of the underlying sandy lean clays do not meet the minimum infiltration rate requirements of the referenced Riverside County document for infiltration basins or infiltration trenches.

The Riverside County Stormwater Quality Best Management Practice Design Handbook also indicates that the infiltration rate can be estimated by using the Riverside County Flood Control and Water Conservation District Hydrology Manual. Accordingly, this data was reviewed for comparison with our field and laboratory test results. Based on our review of the Hydrology Manual, dated April 1978, prepared by the Riverside County Flood Control and Water Conservation District, a hydrologic soils group map for Sunnymead (Plate C-1.17) indicates that the surface soils in the area of the subject site are shown as consisting of Group "B" type soils. Group "B" type soils are defined by the United States Department of Agriculture, Natural Resources Conservation Service, as soils typically having between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Using a Group "B" type soil from Plate C-1.17 and a fallow agricultural type cover ("land plowed but not tilled or seeded") from Plate D-5.5 of the Hydrology Manual, the runoff index number would be 85. Based on Plate E-6.2 from the Hydrology Manual, a runoff index of 85 would indicate an infiltration rate of 0.19 inches per hour for antecedent moisture condition (AMC) type II. This infiltration rate is less than the minimum 0.5 inch per hour rate for an infiltration basin and it is less than the minimum 0.27 inch per hour criteria for infiltration trenches listed in the Riverside County Stormwater Quality Best Management Practice Design Handbook. Accordingly, based on our subsurface investigation, infiltrometer testing and review of the above-referenced Riverside County guidelines, the subsurface soil conditions indicate that an infiltration system would not be feasible at the site. Furthermore, from a geotechnical engineering perspective, due to the presence of collapsible soils, infiltration systems which result in saturation of the near surface soils are not recommended for this site. However, this

report includes general recommendations for grading and preparation of a basin area in the event stormwater storage is required at the site.

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.

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on representative samples of the near surface soils. The expansion index testing was performed in accordance with ASTM D4829 "Standard Test Method for Expansion Index of Soils." The soils tested were classified by expansion potential in accordance with Table 1 of ASTM D4829 and are summarized in Appendix C of this report. The results of two (2) expansion index tests, conducted on samples collected from the ground surface to a depth of about 3½ feet BSG, indicated a very low expansion potential. Thus, there are no special recommendations included in this report to mitigate an excessive Potential Vertical Rise (PVR) due to expansive near surface soils.

However, due to the expansive nature of the deeper clay soils which were generally encountered at depths of approximately 8½ feet BSG and deeper, these soils are not recommended for use as engineered fill within the upper 4 feet BSG in the area of the building pad, nor within the upper 2 feet below pavement areas.

6.4 **Collapsible Soils:** Collapsible soils typically have low moisture contents, are loosely consolidated, and are deposited by aeolian (wind driven) or debris flow processes. Collapse type settlements in soils can occur where densification of the soils result from wetting of the soils due to rainfall or other water sources.

The potential for collapse type soil settlement was evaluated based on the results of consolidation tests. Consolidation tests indicated that the near surface silty sands and clayey sands within the upper approximately 10 feet have a moderate collapse potential. The near surface soils within the upper approximately 10 feet across the site are generally anticipated to possess similar collapse potential characteristics. Accordingly, improvements supported on the existing native soils would be subject to potential excessive collapse settlement. In order to reduce the potential for excessive settlement of foundations due to collapse type soil movements and meet the Walmart foundation settlement requirements, over-excavation and compaction of the upper 8 feet of the near surface soils are recommended in the building pad areas to support foundations on engineered fill.

In order to limit the potential static settlements to not more than 1 inch in areas of lightly loaded miscellaneous foundations, such as screen walls, over-excavation and compaction of the upper 5 feet of the near surface soils is recommended.

A potential collapse settlement up to about 1½ inches was estimated for improvements such as concrete flatwork or pavements without significant fill surcharge loading. If it is desired to reduce the settlement potential to less than 1 inch in these areas, over-excavation would need to extend to a depth of approximately 4 feet below preconstruction site grades. As an alternative, to reduce the potential for excessive settlement due to collapsible soils, an over-excavation of 12 inches could be conducted and a waterproof liner could be used in areas where saturation of the soils may occur, such as landscape planters in the pavement areas. Alternatively, a limited over-excavation depth of 12 inches could be used if collapse settlement up to 1½ inches is acceptable for these areas. It is recommended the method of preparation selected for the pavement subgrade areas be determined based on the tolerable settlements, the owner's tolerance for risk, expected maintenance and the estimated costs.

Due to the presence of collapsible soils, drought tolerant landscaping and low flow irrigation systems are recommended for the site. In addition, infiltration of stormwater into the near surface soils is not recommended at this site.

6.5 **Static Settlement and Bearing Capacity of Shallow Foundations:** The potential for excessive total and differential static settlement of foundations and slabs-on-grade were evaluated based on the anticipated structural loading specified in Walmart's Geotechnical Investigation Specifications and Report Requirements. 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(s) and improvements. The differential component of settlement is often the most damaging. In addition, the allowable bearing pressures for the soils supporting the foundations were evaluated for shear, or punching type failure due to the anticipated foundation loads.

The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and weight of the footing may be neglected. The net allowable soil bearing pressure presented was selected using the Terzaghi bearing capacity equations for spread foundations considering a minimum factor of safety of 3.0 and based on the anticipated static settlements noted in this report. To reduce the estimated static settlements to meet the Walmart requirements of 1 inch total and 0.53 inches differential in 40 feet, spread foundations supporting column loads would need to be supported on at least 5 feet of engineered fill, loading dock wall foundations would need to be supported on a minimum of 2 feet of engineered fill and the upper 8 feet of collapsible soils would need to be over-excavated and compacted as engineered fill below the entire building pad. Conventional shallow foundations supported on engineered fill were determined to be the most economical foundation type for the allowable differential and total movements.

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

6.6 Interior Slabs-on Grade: The slabs on the project that should be prepared as interior slabs include: appurtenances defined as those items attached to the building proper, typically including, but not limited to, the building, sidewalks, garden center, porches, ramps, stoops, truck wells/docks, concrete aprons, compactor pad, etc. Interior floor slabs should be supported on at least 4 inches of non-recycled Class 2 aggregate base (AB) overlying engineered fill extending to the depth recommended in this report. The 4 inches of non-recycled Class 2 AB may include a ¾-inch choker course (if necessary). The aggregate base material is recommended for structural purposes, to provide a capillary break, and to provide a working surface during construction. The engineered fill placed on the building pad should be tested and the in-place density and moisture content confirmed to meet the compaction and moisture content recommendations of this report immediately prior to placement of the AB section. In addition, the finished compacted subgrade and the aggregate base section should be stable under proofrolling (deflections should not exceed ½ inch under a loaded concrete truck).

Several potential concerns need to be considered to reduce the potential for damaging slabs during construction. These issues include: 1) differential slab movement at interior columns, and 2) aggregate base sections below the slabs.

Often interior column construction methods can result in cracking of the overlying slabs. In some cases, the subgrade or base preparation for the slab is not continuous across the top of spread footings. Often the zone above the top of structural footings is backfilled with concrete during slab placement. This results in a differential slab support/shrinkage condition 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 engineered backfill of 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 for more uniform support for the slabs which should reduce the potential for cracking.

It has been our experience that placing concrete for the concrete slab by the tailgating method can cause subgrade instability due to the high frequency of concrete trucks which travel across the prepared subgrade. Compacted subgrade can experience instability under construction vehicle loads, resulting in heaving and disturbance of the subgrade. This condition becomes more critical during wet winter and spring months. A layer of aggregate base (AB) can reduce the potential for instability under loading of concrete trucks. Also, the improved support characteristics of the AB can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design underlain with at least 4 inches of aggregate base for construction and design purposes.

To reduce the potential for damaging slabs during construction, the following recommendations are presented: 1) delay construction of the slab on grade adjacent to CMU wall locations until the majority of the wall settlement has occurred to reduce the potential for damage to slab-wall connections; 2) design for a differential slab movement relative to interior columns; and 3) provide at least 4 inches of aggregate base below the slabs. In addition, it is expected that erection of roof

steel may require crane and heavy equipment. In addition to the above recommendations, the loaded track and/or pad pressure of cranes and other equipment operating on the pad or slab-on-grade should be considered by the contractor. Where equipment is to be operated on the prepared building pad, rutting and disturbance of the pad can occur. These conditions will require reconditioning, compaction and grading of the building pad prior to placement of the aggregate base and subsequent slab construction. If cranes or other construction equipment are to be used on the slab, the contractor should first analyze whether the slab on grade will be sufficient for the anticipated equipment loads.

6.7 Climatic Conditions: According to the Climatic Atlas of the United States, published by the U. S. Department of Commerce, the project area receives on average about 10 inches of precipitation annually with the majority of the precipitation occurring during the months of November through April. In addition, the average daily temperature does not drop below 32° F for any day of the year.

Based on the information provided in the Climatic Atlas of the United States, the site area receives a fairly low annual rainfall. However, there is a potential that wet periods may result in elevated moisture levels within the near surface soils, resulting in unstable conditions during compactive effort. Therefore, provisions should be made in construction to address the potential for subgrade soil instability.

6.8 Faulting and Seismicity: The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the San Jacinto-San Jacinto Valley fault, located about 5.6 miles (9.0 kilometers) northeast of the site. Given that no mapped faults are known to trend through the site, the potential for fault rupture at the site is considered low.

Based on the 2010 California Building Code (CBC), the site is classified as a class D site (stiff soil profile type) with standard penetration resistance, N-values, averaging between 15 to 50 blows per foot in the upper 100 feet below site grade.

Based on the 2010 CBC and considering a five percent damped design spectral response acceleration for short period (SD_S) of 1.0, the peak horizontal ground acceleration as defined in the CBC was estimated to be 0.40g. Based on the results of hazard deaggregation analysis, a maximum considered earthquake magnitude of 7.6 is estimated for the site.

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

Groundwater was not encountered during drilling of test borings for this investigation and historic data indicates groundwater depths are greater than 50 feet. Thus, the potential for liquefaction to occur at the site is considered low. However, based on the loose condition of some of the granular soils encountered, the potential for dry seismic settlement was evaluated.

The soil samples from borings adjacent to the CPTs were generally consistent with the soil profiles from the CPT logs. Accordingly, seismic settlement analyses were conducted based on soil properties revealed by test borings, CPTs and the results of laboratory testing. The analyses were conducted using the computer program LIQUEFYPRO by Civiltech. A design horizontal ground acceleration of 0.40g and an earthquake magnitude of 7.6 were used in the analysis. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

Based on our evaluations of dry seismic settlement using the CPT data, seismic settlements within the Walmart building area were estimated to be ¼ inch total and ⅛ inch differential in 40 feet.

6.10 <u>Asphaltic Concrete (AC) Pavements</u>: 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 were obtained from the Walmart Geotechnical Investigation Specifications and Report Requirements (September 25, 2012). The "standard duty" pavement 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.

Recommendations for the pavement sections in the outlot areas were based on traffic index values ranging from 5.0 to 7.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual traffic loading.

In general, the anticipated subgrade soils consist of clayey sands or silty sands with varying amounts of clay and gravel. The subgrade support characteristics of the native soils were evaluated using Resistance (R)-value tests. The results of the tests on samples collected from between the ground surface and a depth of about 3½ feet BSG, indicated R-values ranging from 14 to 63. In accordance with the Caltrans Highway Design Manual, the lowest R-value (an R-value of 14) was used for design of the pavement sections.

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

6.11 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 were obtained from the Walmart Geotechnical Investigation Specifications and Report Requirements (September 25, 2012). 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 at 28 days for concrete. Tests performed on the native soils indicated a correlated k-value of 120 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 4-inch layer of Class 2 aggregate base material (minimum R-value of 78). Therefore, a k-value of 150 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.

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

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on 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.

6.13 Sulfate Attack of Concrete: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfate concentrations 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 exploration and laboratory testing program, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following conclusions are presented.

7.1 Based on the findings and results of the geotechnical engineering investigation, the site is considered suitable for the proposed construction with regard to support of the structure on shallow foundations and the use of concrete slabs-on-grade, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation by Moore Twining are integral to this conclusion.

7.2 The near-surface soils exhibit low shear strength, high compressibility characteristics, moderate to severe collapse potential, very low expansion potential and poor to excellent support characteristics for pavements when compacted as engineered fill.

7.3 Due to the presence of loose, collapsible near surface soils, in order to limit static settlements of new foundations to 1-inch total and 0.53 inches differential in 40 feet, the on-site soils should be over-excavated and compacted to a depth of approximately 8 feet below preconstruction site grades.

7.4 Based on our site specific evaluation, the risk of liquefaction is considered low. A total seismic settlement of less than ¼ inch and a differential settlement of less than ⅛ inch in 40 feet, were estimated as a result of the design seismic ground shaking.

7.5 Based on our review of the 2003 Riverside County General Plan (Figure S-7 Documented Subsidence Areas) and our review of the Riverside County Land Information System (Subsidence Map), the site is mapped as being located in an area susceptible to subsidence.

7.6 Based on our review of the 2003 Riverside County General Plan, the site is located outside the 100- and 500-Year flood zones and the site is also outside the dam inundation zones.

7.7 The project site is not located in an Alquist-Priolo Earthquake Fault Zone. No known faults are mapped crossing the site. The nearest known active or potentially active fault is the San Jacinto-San Jacinto Valley fault, which is located approximately 5.6 miles northeast of the site. Thus, the potential for ground rupture at the site is considered low.

7.8 The design earthquake horizontal ground acceleration for this site was estimated to be 0.40g.

7.9 Based on our subsurface investigation, the results of field double-ring infiltrometer testing and review of the referenced Riverside County guidelines, the subsurface soil conditions indicate that an infiltration system would not be feasible at the site. In addition, infiltration systems which increase the soil moisture contents are not recommended at this site from a geotechnical engineering perspective due to the presence of collapsible soils and the potential for subsurface infiltration systems to cause settlement and damage to adjacent, or overlying improvements.

7.10 Based on the resistivity values, the soils exhibit a "mildly corrosive" corrosion potential. In addition, the results of soil sample analyses indicated the soils exhibit a "negligible" potential for sulfate exposure to concrete.

8.0 RECOMMENDATIONS

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

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

8.1 General

8.1.1 A grading plan was not available at the time this report was prepared. Moore Twining should be provided the opportunity to review the grading plans so that the recommendations of this report can be reviewed relative to the planned grading and any relevant supplemental recommendations can be presented.

8.1.2 In order to reduce the settlement potential to less than 1 inch in pavement and concrete flatwork areas outside the building pad preparation limits, over-excavation would need to extend to a depth of approximately 4 feet below preconstruction site grades. As an alternative, to reduce the potential for excessive settlement due to collapsible soils, a lesser over-excavation (12 inches) could be conducted and a waterproof liner could be used in areas where saturation of the soils may occur, such as landscape planters in the pavement areas. Alternatively, a limited over-excavation depth of 12 inches could be used in pavement and flatwork areas outside the building pad if collapse settlement up to 1½ inches is acceptable for these areas. The method of preparation selected for the pavement subgrade areas should be determined based on the tolerable settlements, the owner's tolerance for risk, expected maintenance and the estimated costs.

8.1.3 A preconstruction meeting including, as a minimum, the owner, general contractor, the civil engineering consultant (CEC), earthwork subcontractor, foundation and paving subcontractors, and the Construction Testing Laboratory (CTL) 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 the requirements of the plans and this report, critical project issues, concerns and scheduling.

8.1.4 The Contractor should be responsible for including in the base bid the costs to perform the work required by the project plans, the project specifications, and the requirements of the governing agencies, whichever is most stringent. After review of the geotechnical report and the construction 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.

8.1.5 Climatic conditions are discussed in Section 6.7 of the report which indicated that the project area receives an average of about 10 inches of precipitation annually with the majority of the precipitation occurring during the months of November through April. In general, the months of November through April would be the anticipated months of greater precipitation. If wet, unstable soil conditions are experienced, methods such as aeration, mixing wet soils with drier soils, chemical (i.e., cement) treatment of the soil, or over-excavation and placement of a bridge lift of aggregate base and a geotextile stabilization fabric such as Mirafi 600X, may be required to achieve a stable condition. Stabilization of wet soils should be conducted in accordance with the project specifications.

8.1.6 The contractor should be responsible for performing a survey of existing conditions to document the condition of all existing improvements which may be potentially impacted by the proposed construction, including the offsite storm drain improvements within the influence of temporary excavations, equipment vibrations, etc. The survey should be performed by a qualified individual and include, as a minimum, pictures, level survey (as necessary) and include a written summary of the conditions of the adjacent improvements. A copy of the condition survey should be provided to Walmart and the CEC for review and approval prior to the start of work.

8.1.7 The Contractor will be responsible for monitoring existing adjacent improvements during construction and documenting the conditions at the completion of the construction, such as those which may be impacted by the offsite storm drain construction work. The Contractor shall submit a written plan detailing the methods proposed to document and monitor the condition of the existing adjacent improvements. The monitoring should be capable of detecting movements on the order of $^{1}/_{16}$ of an inch. If any existing improvements move more than ⅛ of an inch during the construction operations, the contractor shall notify Walmart and the CEC immediately. The contractor will be required to take remedial actions to prevent damage to adjacent improvements immediately at no cost to the Owner.

8.2 Site Grading, Landscaping and Drainage

8.2.1 Develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least five feet away from the structures, 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 one (1) percent away from the foundations.

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

8.2.3 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. All irrigation lines and sprinklers should be periodically monitored for leaks and to ensure over-irrigation does not occur. All leaks, damage, etc. should be repaired promptly.

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

8.2.5 Perimeter curbs should be extended to the bottom of the aggregate base section, where irrigated landscape areas meet pavements or walkways to reduce the potential for landscape water to migrate into the base section and reduce the life of the pavements.

8.2.6 If it is desired to reduce the potential for collapse type soil movement in adjacent pavement areas, a waterproof liner should be included below landscape areas to prevent infiltration of landscape and surface water in the near surface soils. Refer to the alternatives described in the Recommendation section 8.1.2 of this report.

8.2.7 It is not recommended to place landscape and planted areas adjacent to the structure. Trees should be setback from the proposed structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from the proposed building.

8.3 Storm Water Basin

As described in this report, the subsurface soil conditions indicate that an infiltration system would not be feasible at the site. Furthermore, from a geotechnical engineering perspective, due to the presence of collapsible soils, infiltration systems which result in saturation of the near surface soils are not recommended for this site. Thus, the use of a stormwater infiltration basin is not recommended. In the event alternate systems, such as drywells are considered for use in disposal of storm water, Moore Twining should be requested to evaluate the subsurface soils for consideration of alternate designs. However, it is our understanding a basin area may be needed for collection, storage and filtration of stormwater prior to discharge to the storm drain system. The design may include a media filter. Accordingly, the following general recommendations were prepared for a stormwater collection basin which does not rely on infiltration for design.

8.3.1 The final location and depth of the basin should be provided to Moore Twining for review when available. Due to the potential for settlement and soil movements from wetting of the upper collapsible soils due to lateral infiltration of water from the basin, perimeter cutoffs or liners may be required for the basin depending on the design water elevation and the duration of stormwater storage. When available, this design information should be provided to Moore Twining for review and assessment of perimeter cutoff systems.

8.3.2 The near surface soils encountered are loose and susceptible to erosion. Based on the soils encountered and the results of testing included in this report, basin side slopes should be constructed with a maximum repose of 3H to 1V, or flatter. In order to reduce the potential for erosion of the side slopes, positive rooting vegetation could be established on the slopes. However, vegetation cover would require an irrigation system to maintain. Thus, as an alternative to vegetation, the side slopes could be un-vegetated and more aggressive maintenance measures could be scheduled to repair erosion and shallow soil movements.

8.3.3 If future erosion or instability in the form of slides, debris or earth flow, accelerated erosion, or other forms of slope instability occur on slopes, our firm should be contacted to provide recommendations for repair, and the distressed areas should be repaired as soon as possible under the observation of our firm.

8.3.4 Moore Twining should be afforded the opportunity to review structural design details to further evaluate the setbacks, grading, and drainage. Structures should be setback from the basin side slopes to provide adequate foundation support and protection. Structures should be setback from the top of slopes a distance equal to ⅓ of the height of the slope, or a minimum of 15 feet, whichever is greater. Pavements, exterior flatwork and landscaping improvements may be placed within the setback area, however, these improvements may be subject to movement, distress, etc. from future shallow slope movement or erosion.

8.3.5 The area above the top of the side slopes of the basin should be graded to drain surface runoff away from the basin and prevent runoff from flowing over the top of the basin side slopes. Ramps, such as for maintenance and sediment removal, into the basins (if any) should be protected from erosion where concentrated runoff may flow along the ramp.

8.3.6 Our experience with media filters for storm water filtration is that they have a limited life span that typical construction projects outlive. Thus, regular maintenance should be expected to maximize their useful life.

8.3.7 Periodic maintenance should include repair of erosion features such as shallow soil movement on side slopes and to minimize the reduction in infiltration capacity over time. Periodic maintenance of the basin should also include removal of trash, debris, vegetation, siltation and fine particles (e.g. silts and organic matter), performing drainage improvements to reduce the sediment load in the runoff and conduct periodic ripping/discing of the bottom of the basin. Lightweight equipment should be used for these activities to minimize compaction of the basin surfaces.

8.4 Site Preparation

8.4.1 All surface topsoil, vegetation, organics, surface and subsurface improvements (if any) should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and surface topsoils with more than 3 percent organics by dry weight. For estimating purposes, a minimum stripping depth of 4 inches should be used. The actual depth of stripping should be reviewed by the CTL at the time of construction.

8.4.2 A pile of concrete and brick debris, approximately 15 feet by 15 feet in plan dimensions, was noted in the southeast portion of the site. In addition, just south of this debris pile was an apparent concrete washout pit about 1 to 2 feet deep with a plan area of 15 feet by 15 feet. The concrete washout pit, all trash and debris should be completely removed from the site in a legal manner and not mixed in with soils to be used as engineered fill.

8.4.3 All wells scheduled for demolition and all wells encountered during construction should be abandoned per state and local requirements under the observation of the CTL. An abandonment permit should be obtained from the local environmental health department and certificates of destruction should be provided 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 or to a minimum depth of 60 inches below foundations, whichever is greater. Excavations for well abandonment should be backfilled with engineered fill.

8.4.4 Following stripping and removal of existing surface and subsurface improvements, the building pad areas for the Walmart store and outlot pad should be over-excavated to a minimum of 8 feet below preconstruction site grade, to a minimum of 5 feet below the bottom of foundations other than the depressed loading dock area, to a minimum depth of 2 feet below the deeper foundations in the loading dock area and to the depth required to remove all undocumented fills, whichever is greater. The over-excavation should be conducted throughout the entire building pad limits. Slot cutting only below foundations will not be allowed. Upon approval of the horizontal and vertical limits of over-excavation by the CTL based on the survey data, the bottom of the over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to between optimum and three (3) percent above optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D1557 prior to placement of fill. 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 eight (8) 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 limits of the building pad preparation should be depicted on the plans.

8.4.5 Final grading shall produce a building pad ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. Both the finished pad (before the aggregate base is placed) and the aggregate base section shall not depress more than one-half (½) inch under the wheels of a fully loaded concrete truck. If depressions more than one-half (½) inch occur, the Contractor shall perform remedial grading to achieve this requirement at no cost to Walmart. The proof-rolling with a loaded concrete truck or equivalent shall be observed by the CTL. This observation shall be documented in writing by the CTL.

8.4.6 This report recommends that all concrete slabs-on-grade within the building pad limits be constructed over minimum of 4 inches of non-recycled Class 2 aggregate base over the depth of engineered fill recommended below foundations. The aggregate base should be compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D1557.

8.4.7 The CTL should observe the bottoms of all areas of over-excavation, and observe and conduct density testing during placement of engineered fill as grading progresses.

8.4.8 Extra care should be taken by the Contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation requirements. The measurement and verification of the horizontal and vertical extent of over-excavation and compaction is the sole responsibility of the Contractor. The Contractor shall verify in writing to Walmart and the CTL that the horizontal and vertical over-excavation limits were completed in conformance with the requirements of the Foundation Subsurface Preparation note (see Appendix E of this report). 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 were achieved. The surveyor shall also provide a written report that states the over-excavation was performed in accordance with the Foundation Subsurface Preparation note. This verification should be provided prior to requesting pad certification from the CTL or excavating for foundations.

8.4.9 Miscellaneous lightly loaded foundations (such as retaining walls, 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, following stripping and removal of existing surface and subsurface improvements, miscellaneous foundations should be over-excavated to a minimum of 1 foot below foundations, to a minimum of 5 feet below preconstruction site grades, or to the depth required to remove undocumented fills, whichever is greater. The zone of over-excavation shall extend at least 5 feet beyond the edge of foundations on all sides. Upon approval of the over-excavation limits by CTL, the soils at the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to slightly above optimum moisture content and compacted as engineered fill to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D1557.

8.4.10 Following stripping and removal of existing surface and subsurface improvements, areas to receive fill outside the building pad, pavements, and concrete slabs on grade outside the building pad limits should be over-excavated to a depth of at least 12 inches below preconstruction site grades or 12 inches below the bottom of the aggregate base where mitigation of collapsible soils will not be conducted, whichever is deeper; or to a depth of 4 feet below preconstruction site grade where collapsible soils will be mitigated to reduce the potential settlement to less than 1 inch, whichever is deeper. However, if the lesser over-excavation depth is selected, the apparent pond/water feature noted on the 1953 aerial photograph in the southwestern portion of the site (See Drawing No. 2) should be over-excavated to a minimum depth of 36 inches below preconstruction site grade, or to the depth required to remove existing undocumented fills, whichever is greater. Refer to Section 8.1.2 of this report for various alternatives to address potential collapse settlement in the pavement areas and flatwork areas outside the building pad limits. The zone of over-excavation and compaction (overbuild zone) should extend laterally a minimum of 5 feet outside the perimeters of exterior slabs and pavements. The bottom of the over-excavation should be scarified to a depth of 8 inches, moisture conditioned to slightly above optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density determined in accordance with ASTM D1557 prior to placement of fill.

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

8.4.12 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be

notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

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

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

8.4.15 It is anticipated that subgrade instability could occur if earthwork operations are conducted in wet weather. The degree of instability will depend on the actual moisture content of the soils at the time of construction. This condition may require either air drying (including periodic discing), a geotextile fabric, chemical treatment, or a combination of these methods to reduce the potential for instability. The actual method employed to stabilize the bottom of the excavation or pavement subgrade should be selected at the time of construction.

8.5 Engineered Fill

8.5.1 The on-site near surface soils encountered are predominantly clayey sands and silty sands with varying amounts of gravel. These soils are considered suitable for use as engineered fill below the aggregate base section in pavement and building areas, provided they are free of organics (less than 3 percent by weight and no roots larger than 1/4 inch in diameter), irreducible material not greater than 3 inches and deleterious debris, and the moisture content of the soil is within optimum to three (3) percent above moisture content at the time of placement. Expansive, lean clay and sandy lean clay soils were typically encountered at depths of 8½ feet BSG or deeper. The lean clays and sandy lean clay soils encountered will not be suitable for use as engineered fill within the upper 4 feet below the building pad area, nor within the upper 24 inches below the bottom of the aggregate base in pavement areas. If soils other than those considered in this report are encountered, our firm should be notified to provide alternate recommendations.

8.5.2 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this

report; therefore, they should be evaluated by the Contractor during preparation of bids and construction of the project.

8.5.3 Imported fill soil should be non-recycled, non-contaminated and granular in nature with enough fine-grained material to allow cutting "neat" footing trenches. The following acceptance criteria recommended:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	10 - 40
Plasticity Index	Less than 15
Expansion Index (ASTM D4829)	Less than 20
R-Value	Minimum 25
Organics	< 3% by weight
Sulfates	< 0.05 % by weight
Min. Resistivity	> 10,000 ohms-cm

8.5.4 A minimum of 14 days prior to importing fill materials such as soils or aggregate base, the Contractor shall submit a certification signed by a registered geologist or civil engineer with a minimum of five (5) years of experience in environmental sampling and testing of soils (i.e., an environmental specialist), which states: "To the best of my personal knowledge, the import material has been sampled and tested, is not contaminated and does not contain constituents at concentrations that exceed regulatory action levels that would be cause for regulatory action if brought to the attention of the regulatory agency having jurisdiction." The environmental specialist's certification shall include the rationale for the testing protocol, the procedures used to sample the material, the quantity of material to be imported to the site, signed reports from the analytical testing laboratory, chain of custody documents, an analysis of the results of the tests relative to applicable action levels in the State of California, and the methods that will be used to ensure that the material delivered and incorporated into the project is representative of the material that was sampled and tested. As a minimum, the sampling frequency and analytical testing protocol shall comply with the procedures recommended in the Department of Toxic Substances Control, "Information Advisory, Clean Imported Fill Material," dated October 2001. In lieu of sampling and testing aggregate base materials (or bedding sand) from virgin sand and gravel sources, a letter stating that the aggregate base (or bedding sand) comprises materials entirely from natural (virgin) sources and that the aggregate base (or bedding sand) is non-contaminated may be provided by the Contractor. After approval of the Contractor's submittal and prior to being transported to the site, the import fill material shall also be tested and approved by the retail tenant's Construction Testing Laboratory (CTL) for the above geotechnical

characteristics. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested by the CTL for compliance with the geotechnical characteristics specified herein.

8.5.5 Other than recycled aggregate base for pavement areas, contractors should not assume that recycled materials can be used in preparing bids for the project.

8.5.6 Imported, non-expansive soils, or on-site soils should be placed in lifts approximately 8 inches thick (measured loose), moisture conditioned to within optimum to three (3) percent above optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM 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 minimum frequency of at least:

Area	Minimum Test Frequency
Building Pad Subgrade Areas[1]	1 test per 2,500 square feet per compacted lift
Pavement Subgrade[2]	1 test per 10,000 square feet per compacted lift
Utility Trench Backfill	1 test per 200 feet per compacted lift

[1] - Building pad subgrade areas includes the limits defined in the Foundation Subsurface Preparation
[2] - Pavement subgrade includes other areas of construction exclusive of building subgrade.

The above testing frequencies are minimum rates for tests (note that if the type and frequency of testing specified in the approved plans and specifications are different than those specified herein, the most stringent shall apply). Testing frequency should be adjusted by the field technician and the engineer as needed based on the methods used for compaction and the soil conditions; however, the minimum frequencies shall be achieved.

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. Gravel and rock cannot be used without the written approval of Walmart. If required, open graded materials should be vibrated, and mechanically compacted to a dense, non-yielding condition. Maximum lift thickness of 12 inches is recommended. Each lift must be approved by the CTL prior to placing the next lift.

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. Aggregate base used for pavement construction should comply with Class 2 aggregate base in accordance with the State of California Standard Specifications and may include recycled materials. Aggregate base shall be compacted to a minimum relative compaction of 95 percent in accordance with ASTM D1557 standards. Documentation, including laboratory test data, should be provided to the Walmart 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.

8.6 Foundations

8.6.1 Structural loads for the buildings may be supported on spread or continuous footings placed entirely on engineered fill prepared as recommended in the "Site Preparation" (Section 8.4) of this report. Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.6.2 Perimeter foundations should have a minimum depth of 24 inches below the top of the slab or 24 inches below the lowest adjacent finished exterior ground surface, whichever is greater. Interior footings should have a minimum depth of at least 18 inches below the top of the slab-on-grade. All footings should have a minimum width of 15 inches, regardless of load.

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

8.6.4 The foundations should be designed and reinforced for the anticipated settlements and heave. 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: 1) a total

static settlement of 1 inch, 2) a differential static settlement of 0.53 inches in 40 linear feet, 3) a total seismic settlement of ¼ inch, 4) a differential seismic settlement of ⅛ inch, and 5) a potential total and differential heave of 1 inch and ½ inch, respectively.

8.6.5 Miscellaneous lightly loaded (less than 1.5 kips per lineal foot) foundations may be supported on spread or continuous footings supported on engineered fill as recommended in the Site Preparation section of this report. 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 3,000 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	1.500
Spectral Response At 1-Second Period, S_1	0.600
Site Coefficient (based on Spectral Response At Short Period), Fa	1.0
Site Coefficient (based on spectral response at 1-second period) Fv	1.5
Maximum considered earthquake spectral response acceleration for short period, SM_S	1.500
Maximum considered earthquake spectral response acceleration at 1 second, SM_1	0.900
Five percent damped design spectral response accelerations for short period, SDs	1.000
Five percent damped design spectral response accelerations at 1-second period, SD_1	0.3600

8.6.7 The moisture content of the footing subgrade soils should be maintained by periodic wetting until the placement of concrete. If necessary to maintain the specified moisture content of between optimum and three (3) percent above

optimum moisture content, the subgrade should be over-excavated, moisture conditioned as necessary and compacted as engineered fill. The Contractor is responsible for proper notification to the CTL and receipt of written confirmation of this observation prior to placement of concrete.

8.6.8 Site lighting and pylon signs (if any) may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier) extending a minimum depth of 8 feet below finished grade. An allowable skin friction of 150 pounds per square foot per foot of embedment may be used to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained procedure (Section 1806.8.2.1). The allowable passive resistance of the native soils may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per square foot per foot of depth to a maximum of 3,000 pounds per square foot. The passive pressure may be assumed to act over twice the pier diameter. The upper 12 inches of subgrade soils in landscaped areas should be neglected in determining the total passive resistance.

8.7 Frictional Coefficient and Earth Pressures

8.7.1 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 0.40, can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10, can be used for design.

8.7.2 The allowable passive resistance of the engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. The upper 6 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.7.3 The passive pressure was calculated based on an allowable passive earth pressure coefficient of 3.0, and a minimum soil unit weight of 100 pounds per cubic foot.

8.7.4 The active and at-rest pressures of the native soils and engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 45 and 67 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.

8.7.5 The active and at-rest pressures were calculated based on an active earth pressure coefficient of 0.32, an at rest coefficient of 0.48, and a maximum soil unit weight of 140 pounds per cubic foot.

8.7.6 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.

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

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

8.8 Retaining Walls/Screen Walls

8.8.1 Structural loads for lightly loaded retaining walls or screen walls should be supported on engineered fill prepared as recommended in the Site Preparation section of this report. Spread and continuous footings extending a minimum of 12 inches in depth may be designed for a maximum net allowable soil bearing pressure of 3,000 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

8.8.2 The on-site soils are not suitable for use as backfill behind retaining walls such as the loading dock walls. Retaining walls should be backfilled with imported, granular backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular wall backfill should meet the following requirements:

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

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

8.8.4 Granular wall backfill should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.8.5 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 retaining walls.

8.8.6 Retaining walls should be constructed with a drain system including, as a minimum, drain pipes surrounded by at least 1.5 cubic feet of ¾-inch open graded rock fully encapsulated by geotextile filter fabric (140N or equivalent) as shown on Drawing No. 6 in Appendix A. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system do not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The drainage system should be designed by the wall designer and detailed on the plans.

8.8.7 For loading dock area retaining walls only, as an alternative to using drain pipes behind the wall to adequately reduce the potential for hydrostatic pressures behind the wall, weep holes may be used, provided that a continuous crushed rock (minimum 1 cubic feet per lineal foot fully encapsulated by a geotextile filter fabric) section is provided directly behind the wall as shown on Drawing No. 6 in Appendix A. The weep holes cannot have the potential for clogging. The weep holes should discharge directly to an approved drainage.

8.8.8 None of the data included in this report should be used for segmental wall design. A design level geotechnical investigation report should be conducted to provide design parameters for segmental retaining walls (if any). If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data.

8.8.9 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The

Contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

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

8.9 Interior Slabs-on-Grade

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

8.9.1 From a geotechnical perspective, slab reinforcement is not required to address concerns with expansive soil pressures. A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade as well as any reinforcement for temperature and shrinkage stresses based on the settlements noted in this report. According to the Geotechnical Investigation Specification and Report Requirements for Walmart, it is our understanding that a minimum 4-inch thick Portland cement concrete slab is required.

8.9.2 Interior concrete slabs-on-grade should be supported on a minimum of 4 inches of non-recycled Class 2 AB over the depth of engineered fill recommended in this report. The 4 inches of non-recycled Class 2 AB may include a ¾-inch choker course (if necessary).

8.9.3 The subgrade to receive floor slabs should be tested to verify adequate moisture content and compaction prior to placement of the aggregate base. If the moisture content just prior to placement of the floor slab is not at least optimum moisture content, the soils should be moisture conditioned to at least optimum prior to placing a vapor barrier or concrete. If adequate compaction 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. This condition should be verified within 48 hours prior to placement of the aggregate base.

8.9.4 The recommended minimum 4 inches of aggregate base material (R-value of 78) and preparation of the subgrade soils as recommended in this report will provide a design modulus of subgrade reaction of 150 pounds per square inch per inch (psi/in). The AB material is recommended for structural purposes, to provide a capillary break, and to provide a working surface during construction. If concrete trucks 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.

8.9.5 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards and applicable Portland Cement Concrete Association standards.

8.9.6 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade could be wet or saturated prior to placement of the slab-on-grade. 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 vapor retarder is not required beneath exposed concrete floors, provided that moisture intrusion into the structure is permissible for the design life of the structure. The layer of Stegowrap 15 should overlay a minimum of 4 inches of compacted aggregate base (95 percent relative compaction). 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.

8.9.7 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.

8.9.8 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with 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 sealed per manufacturer's recommendations.

8.9.9 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 manufacturer's recommendations.

8.9.10 The manufacturer's requirements vary regarding the surface and cover material around the placed membrane. Vapor retarding membranes should be installed in accordance with the manufacturers' specifications.

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

8.9.12 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity of the floor slab, etc., at a frequency and method as specified by the flooring manufacturer, adhesive manufacturer,

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

8.9.13 To reduce the potential for damaging slabs during construction, the following recommendations are presented: 1) delay construction of the floor slab next to CMU walls until the majority of the wall settlement has occurred; 2) design for a differential slab movement of ½ inch relative to interior columns; and 3) provide at least 4 inches of aggregate base below the slabs. It is expected that erection of roof steel may require cranes. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.9.14 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.

8.10 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs on grade within the building pad limits, nor for slabs subjected to traffic loading. These recommendations are intended for rather lightly loaded sidewalks outside the building pad limits. Recommendations for asphaltic concrete pavements and Portland cement concrete slabs subjected to traffic loading are included in a later section of this report.

8.10.1 Concrete slabs-on-grade outside the building pad preparation limits (such as sidewalks, etc.) should be supported on a minimum of 4 inches of aggregate base over engineered fill prepared as recommended in the Site Preparation section of this report.

8.10.2 The exterior slabs-on-grade should be designed with thickened edges which extend at least to the bottom of the aggregate base below the bottom of the slabs. 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.

8.10.4 The exposed subgrade to receive aggregate base below exterior slabs should be tested to verify adequate compaction and moisture conditions. If adequate compaction 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. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

8.11 Asphaltic Concrete (AC) Pavements

8.11.1 Site preparation for asphaltic concrete pavements should be conducted in accordance with recommendations section entitled, "Site Preparation." The upper 12 inches of subgrade soils below the aggregate base section should be compacted to at least 95 percent relative compaction (ASTM D-1557) as part of final subgrade preparation.

8.11.2 The following pavement sections are based on an R-value of 14, a traffic index of 7.0 for the Walmart "Standard Duty Pavements," and a traffic index of 8.0 for the Walmart "Heavy Duty Pavements." In addition, pavement sections are provided below for the pavements in the outlot areas based on an R-value of 14 and traffic index values ranging from 5.0 to 7.0. The pavement section should be specified by the design engineer based on the anticipated traffic loading. If the pavements are placed prior to construction, or if more frequent traffic is anticipated, Moore Twining should be contacted to re-evaluate the pavement designs based on higher traffic index values. An asphaltic concrete pavement drawing for the Walmart pavement sections is provided on Drawing No. 4 in Appendix A.

Traffic Index = 7.0 Walmart "Standard Duty Pavements"

AC Thickness, inches	AB Thickness, inches	Compacted Subgrade (Min. Thickness), inches
4.0	13.0	12

Traffic Index = 8.0 Walmart "Heavy Duty Pavements"

AC Thickness, inches	AB Thickness, inches	Compacted Subgrade (Min. Thickness), inches
4.5	16.0	12

AC - Asphaltic Concrete compacted as recommended in section 8.11.8 of this report
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557) to provide at least 12 inches of compacted material below the AB.

*Note: According to the Guide for Designing Geotextiles, dated April 28, 2009 (California Department of Transportation), the two-layer pavement thickness may be reduced by use of an effective R-value of 20 provided a geotextile stabilization layer, Mirafi HP570, is placed between the aggregate base and the subgrade soils. In the event the pavement design relying on geotextile enhancement is used, the aggregate base thickness for the two-layer standard duty and heavy duty pavements may be reduced to 12.0 inches and 14.0 inches, respectively.

Two-Layer Asphaltic Concrete Pavements for Outlot Areas

Traffic Index	AC Thickness, inches	AB Thickness, inches	Compacted Subgrade (Minimum Thickness), inches
5.0	3.0	8.0	12
5.5	3.0	10.0	12
6.0	3.0	12.0	12
6.5	3.5	12.5	12
7.0	4.0	13.0	12

AC - Asphaltic Concrete compacted as recommended in section 8.11.8 of this report
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557) to provide at least 12 inches of compacted material below the AB.

*Note: According to the Guide for Designing Geotextiles, dated April 28, 2009 (California Department of Transportation), the two-layer pavement thickness may be reduced by use of an effective R-value of 20 provided a geotextile stabilization layer, Mirafi HP570, is placed between the aggregate base and the subgrade soils. In the event the pavement design relying on geotextile enhancement is used, the aggregate base thickness for the two-layer pavements for traffic indices of 5.0, 5.5, 6.0, 6.5 and 7.0 may be reduced to 7.0 inches, 9.0 inches, 10.5 inches, 11.0 inches, and 12.0 inches, respectively.

8.11.3 Even if the Mirafi HP 570 geotextile layer is not incorporated into design, a geotextile fabric would help prolong the life of the pavements by preventing fine grained subgrade soils from migrating into the AB section. A geotextile stabilization fabric such as Mirafi 600X, or equivalent, placed below the AB section can extend the life of the pavements by reducing the migration of fines into the aggregate base section. This is a suggestion for Walmart to consider and is not intended to become a project requirement unless elected by Walmart.

8.11.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement section should be re-evaluated for the changed subgrade conditions.

8.11.5 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. In addition, planters adjacent to paving should be lined. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

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

8.11.7 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.

8.11.8 The asphaltic concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to AASHTO T209 or ASTM D2041.

8.11.9 At a minimum, it is recommended the asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the 2010 State of California Department of Transportation (Caltrans) Standard Specification Requirements. The contractor shall provide mix designs, prepared and signed by a registered civil engineer in California, to the Walmart CEC for review and approval prior to placement of asphaltic concrete.

8.12 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland cement concrete pavement structural sections are presented in the following subsections. The PCC pavement details should be prepared by a civil engineer experienced in PCC pavement design for similar uses. The pavement thicknesses included herein assume a minimum modulus of rupture of 550 pounds per square inch for the concrete pavement. A k-value of 150 psi/in at the top of the aggregate base was used to determine pavement section thicknesses. The pavement designer should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic. The Portland cement concrete pavement sections are shown on Drawing No. 5 in Appendix A.

8.12.1 Site preparation for Portland cement concrete (PCC) pavements should be in accordance with the recommendations section entitled, “Site Preparation.” The aggregate base should be compacted to a minimum of 95 percent relative compaction. In addition, the upper 12 inches of subgrade soils below the aggregate base should be compacted to at least 95 percent relative

compaction (ASTM D-1557) as part of final subgrade preparation. A Portland Cement Concrete section is provided on Drawing No. 5 in Appendix A.

8.12.2 The following "standard duty" PCC pavement section thicknesses is based on a design k-value of 150 psi/in. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum modulus of rupture of 550 psi for the concrete, 2) load transfer by aggregate interlock or dowels, 3) concrete shoulder, 4) a load safety factor of 1.1, and 5) an ESAL (18 kips) of 15 axles per day (this equates to 109,500 ESALs for the design life of the pavement).

Pavement Component	Thickness, Inches
Portland Cement Concrete	6.5
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Minimum Compacted Subgrade (95% Minimum Relative Compaction)	12.0

8.12.3 The following "heavy duty" PCC pavement section thicknesses is based on a design k-value of 150 psi/in. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum modulus of rupture of 550 psi for the concrete, 2) load transfer by aggregate interlock or dowels, 3) concrete shoulder, 4) a load safety factor of 1.1, and 5) an ESAL (18 kips) of 46 axles per day (this equates to 335,800 ESALs for the design life of the pavement).

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

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

8.12.5 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 4,000 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC pavement.

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

8.12.7 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-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

8.12.9 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.

8.12.12 Isolation (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.

8.13 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.

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 Engineered shoring systems shall be designed by the Contractor's consultant to limit potential deflection of existing adjacent improvements. This analysis and assessment shall be conducted by a registered professional engineer and shall be submitted to the applicable governing agency for review and approval. Moore Twining should also be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.13.4 Prior to the start of excavation and/or shoring installation activities, the owners of adjacent properties, public street improvements and utilities shall be notified within the time period required by the applicable jurisdiction.

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

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

8.14 Utility Trenches

8.14.1 The utility trench subgrade should be prepared by excavation of a neat trench without disturbance to the bottom of the trench. If sidewalls are unstable the Contractor shall either slope the excavation to create a stable sidewall or shore the excavation. All trench subgrade soils disturbed during excavation, such as by accidental over-excavation of the trench bottom, or by excavation equipment with cutting teeth, should be compacted to a minimum of 95 percent relative compaction prior to placement of bedding material. The Contractor is responsible for notifying the CTL when these conditions occur and arrange for the CTL to observe and test these areas prior to placement of pipe bedding. The Contractor shall use such equipment as necessary to achieve a smooth undisturbed native soil surface at the bottom of the trench with no loose material at the bottom of the trench. The Contractor shall either remove all loose soils or compact the loose soils as engineered fill prior to placement of pipe and backfill of the trench.

8.14.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. The Contractor is responsible for contacting the governing agency and pipe manufacturer to determine the requirements for pipe bedding, pipe zone and final backfill. The Contractor is responsible for notifying the Owner and the CTL if the requirements of the agency, manufacturer 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 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The haunches and initial backfill (12 inches above the top of pipe) should consist of a select

sand meeting these sand equivalent and gradation requirements that is placed in maximum 6-inch thick lifts and compacted to a minimum relative compaction of 95 percent using hand equipment. The final fill (12 inches above the pipe to the surface) may be on-site soils or approved imported granular fill compacted to a minimum of 95 percent relative compaction. All materials should be placed within optimum to three (3) percent above optimum moisture content.

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

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

8.14.4 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to reduce the potential for migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Walmart.

8.14.5 Trench backfill should be placed in maximum 8 inch loose lifts, moisture conditioned to within optimum to three (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 during placement and compaction of the backfill materials.

8.14.6 On-site soils or approved imported granular fill may be used as final backfill in trenches.

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

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

8.14.9 Storm drains and/or utility lines should be designed to be watertight. If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines, stormwater, sewer, water, retaining wall drains, etc. be inspected by video inspection 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 to Walmart and the Walmart CEC a copy of video tape and a written description of the pipe condition prepared by the video inspection firm prior to placement of improvements above the utilities. In addition, the Contractor is required to inspect and test the utility lines as required by the pipe manufacturer and governing agencies.

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

8.14.11 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 parallel building foundations.

8.15 Corrosion Protection

8.15.1 Based on the ASTM Special Technical Publication 741 and the analytical results of a single soil sample analysis, the soils are "mildly corrosive" corrosion potential to ferrous alloy pipes, as indicated by resistivity values of 21,000; 19,000; 13,000 ohm-cm and a pH values of 7.6, 7.6 and 7.4, respectively. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on the "mildly corrosive" corrosion potential of the soil. 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.

8.15.2 Corrosion of concrete due to sulfate attack is not anticipated based on the none-detected, 0.00073, and 0.00068 concentrations of sulfates determined for the near-surface soils. According to the 2010 California Building Code, the concentration of sulfates falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, restrictions are not required regarding the type, water-to-cement ratio, or strength of the concrete used for foundations due to the sulfate content. A type I or type II cement may be used for concrete structures.

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, slabs-on-grade, pavements, 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, Moore Twining assumes 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 to receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations section 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 The soil samples will be retained for six months from the completion of the field exploration unless desired to be retained for a longer period of time.

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

11.7 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 3.1, 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.

11.8 This report has been prepared for the exclusive use of Kimley-Horn & Associates, Inc., Walmart and their successors. 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.

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

11.10 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.

11.11 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 is 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 Kimley-Horn & Associates, Inc. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
MOORE TWINING ASSOCIATES, INC.

DRAFT

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