



GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED ORCHARD SUPPLY HARDWARE STORE
720 WEST SAN CARLOS STREET
SAN JOSE, CALIFORNIA
Project Number: E82601.01

For:

Oppidan
6450 Via Del Oro
San Jose, California 95116

February 13, 2013



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E82601.01

Oppidan
6450 Via Del Oro
San Jose, California 95116

Attention: Mr. Ted Anderson

Subject: Geotechnical Engineering Investigation
Proposed Orchard Supply Hardware Store
720 West San Carlos Street
San Jose, California

Dear Mr. Anderson:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed Orchard Supply Hardware Store to be located at 720 West San Carlos in San Jose, California.

The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations. It is recommended that those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement, however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

In addition, it is recommended that Moore Twining be retained to provide inspection and testing services for the excavation, earthwork, pavement, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the construction complies with our recommendations. These services are not, however, part of this current contractual agreement. We would appreciate the opportunity to provide a proposal for these additional services after construction documents are completed.

**Proposed Orchard Supply Hardware
720 West San Carlos, San Jose, California
February 13, 2013**

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We appreciate the opportunity to be of service to Oppidan. 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

Dean B. Ledgerwood II
Project Geologist
Geotechnical Engineering Division

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for the proposed Orchard Supply Hardware Store, planned to be located at 720 West San Carlos Street in San Jose, California

It is our understanding that the proposed construction will include a new Orchard Supply Hardware (OSH) store with an attached garden center comprising approximately 47,000 square feet. It is our understanding that the store area will have a plan area of 35,000 square feet and the garden center will comprise 12,000 square feet. The proposed development will include asphaltic concrete parking, Portland cement concrete pavements, underground utilities, and isolated landscape areas.

At the time of this investigation, the site included three (3) structures. The structures consisted of the existing OSH store within the western portion of the site, the customer pick-up warehouse building in the southwest corner of the site, and an abandoned single family residence in the southeast corner of the site. The remainder of the site is generally covered by existing asphaltic concrete pavement. In addition, a railroad spur was noted adjacent to the west of the existing dock high building.

The site is relatively flat, sloping slightly from the northwest corner to the southwest corner of the site.

On November 9, 2012, six (6) test borings were drilled at the site to depths ranging from about 10 to 51½ feet BSG. After the November 9, 2012 field investigation was performed, the location of the proposed building was revised. Therefore, on January 30, 2013, three (3) additional test borings were drilled within the limits of the proposed building to depths ranging from 20 to 25 feet BSG. In addition, two (2) shallow test borings were drilled to depths ranging from 2½ to 3 feet BSG within planned landscape areas to obtain samples for laboratory permeability testing.

In general, the near surface soils encountered within the building pad consisted of undocumented fill soils extending to depths ranging from 3½ to 18½ feet BSG. The undocumented fill soils encountered comprised lean and fat clay soils with variable amounts of brick and wood debris. Below the undocumented fills, native lean clay soils and fat clay soils with relatively thin interbedded layers of clayey sands and well graded sands with silt, were encountered to the maximum depth explored of 51½ feet BSG. Within the limits of the proposed parking areas, below the existing pavements, undocumented fill soils were encountered to depths ranging from 1½ feet to 15 feet BSG. The undocumented fills comprised lean and fat clay soils with gravel, glass, brick, plastic, and metal debris. Below the undocumented fills, native lean and fat clay soils were encountered to the maximum depths explored in the parking area of 21½ feet BSG. It should be noted that due to the variable depth of undocumented fills encountered, areas of deeper fills may be present throughout the site.

During the November 2012 geotechnical investigation, groundwater was encountered at a depth of 29½ feet BSG. Groundwater was not encountered during the January 2013 investigation.

EXECUTIVE SUMMARY, Continued

In order to reduce the potential for excessive differential static settlement of shallow spread foundations, it is recommended foundations be supported on engineered fill established by over-excavation and compaction of the existing undocumented fills. Foundations supported on engineered fill as recommended in this report would reduce estimated static total and differential static settlements to typical tolerable limits of 1 inch total and ½ inch differential in 40 feet.

The depth and extent of the undocumented fills are not known. It is possible that the deeper fills encountered are associated with removal of 8,000 cubic yards of soil as part of a soil remediation work conducted around 1996. Information from the soil remediation, such as the limits and depths of the excavation conducted as part of this work, should be provided to our firm for review and consideration of the project. This information would also be helpful for use in developing guidelines for construction estimates and for contractors bidding the work. If information regarding this excavation work is not available, a supplemental investigation could be conducted just prior to construction to estimate the depths and extent of the undocumented fills.

The moisture contents of the native and undocumented fill fat clay soils are anticipated to be overly moist to achieve compaction. Therefore, aeration or chemical treatment should be anticipated for the site preparation in order to dry the soils and achieve a moisture content suitable for compaction of the onsite soils as engineered fill. In addition, soils with high moisture contents are also anticipated at the bottom of areas of over-excavation; thus, the excavations are anticipated to require stabilization such as by chemical soil treatment, placement of a bridge lift of geotextile and aggregate base, or a combination of these methods to achieve a stable surface for conducting earthwork operations.

Due to the depth of fills to be over-excavated in some areas, as an alternative to over-excavation of the existing undocumented fills, it may be possible to consider alternative methods of ground improvement such as stone columns or deep soil-cement mixing along with a stiffened foundation and floor slab design. However, the feasibility and cost effectiveness of alternate approaches would need to be further evaluated.

Based on the expansion potential of the near surface soils, recommendations for placement of an imported non-expansive engineered fill below interior, exterior slabs on grade and PCC pavements, are provided in the Site Preparation section of this report. As an alternative to importing granular fill, it may be possible to chemically treat the onsite soils for use as a non-expansive fill within the building pad. However, in order to evaluate the feasibility of lime treatment of the onsite soils for this use, laboratory lime suitability / mix design testing would be required.

The near surface soils tested exhibited a medium expansion potential, high plasticity and poor pavement support characteristics.

The nearest known active or potentially active fault is the Simi-Santa Rosa Fault, located about 4.2 miles (6.7 kilometers) south of the site. Therefore, the potential for fault rupture at the site is considered low.

EXECUTIVE SUMMARY, Continued

The results of the seismic settlement analysis indicate an estimated total seismic settlement of up to ½ inch and differential seismic settlements of about ¼ inch in 40 feet for the design earthquake.

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

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

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SAN JOSE, CALIFORNIA
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1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed Orchard Supply Hardware store planned to be located at 720 West San Carlos Street in San Jose, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Oppidan to perform this geotechnical engineering investigation.

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

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

2.0 PURPOSE AND SCOPE OF INVESTIGATION

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

- 2.1.2 Evaluation of the near surface soils within the zone of influence of the proposed foundations;
- 2.1.3 Assessment of the potential for liquefaction and recommendations for CBC seismic near source factors and coefficients;
- 2.1.4 Geotechnical parameters for use in design of foundations and slabs-on-grade, (e.g., soil bearing capacity, settlement and development of lateral resistance);
- 2.1.5 Recommendations for site preparation including placement, moisture conditioning, and compaction of engineered fill soils;

- 2.1.6 Recommendations for the design and construction of new asphaltic concrete (AC) and Portland cement concrete (PCC) pavements;
- 2.1.7 Recommendations for temporary excavations and trench backfill; and
- 2.1.8 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed Orchard Supply Hardware Store referenced in the Anticipated Construction section of this report. This investigation did not include a geologic/seismic hazards evaluation, flood plain investigation, in-place density tests, environmental investigation, or environmental audit.

2.2 Scope: Our proposal and amendment, dated September 6, 2012 and dated January 14, 2013, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

- 2.2.1 A plan, entitled Site Plan Alt 2, prepared by AMS Associates, dated November 28, 2012, was reviewed and is referred to in this report as the site plan.
- 2.2.2 A visual site reconnaissance and subsurface exploration were conducted.
- 2.2.3 A recent aerial photograph of the site from online sources, was reviewed.
- 2.2.4 The Phase I report prepared by PSI, identified by project number 0575442-2, dated July 20, 2012 was reviewed.
- 2.2.5 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.
- 2.2.6 Mr. Ted Anderson (Oppidan) and Mr. Al Shaghaghi (AMS Associates) were consulted during the investigation.
- 2.2.7 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and the engineering properties of the subsurface soils encountered.
- 2.2.8 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

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

3.1 Site History: At the time of the field exploration, the project site was occupied by an existing Orchard Supply Hardware (OSH) Store and associated site improvements. Existing site improvements included three (3) structures, including the existing OSH store within the western side of the site, the customer pick-up warehouse building in the southwest corner of the site, and an abandoned single family residence in the southeast corner of the site. Based on information from Orchard Supply Hardware's website, it is our understanding that the OSH store building on the site was constructed in 1946.

Based on our review of the referenced Phase I report, the subject site was used in 1915 as a baseball field. The existing residential structure located at the southeast corner of the site was constructed in 1886 and was reportedly occupied through 1970. Until 1945, the area of the existing Orchard Supply Hardware building was undeveloped. However, according to the Phase I report, single family homes similar to the one present at the southeast corner were previously located adjacent to Royal Avenue. The existing Orchard Supply Hardware store building was noted in historic documents since 1946. The Phase I report indicates based on review of aerial images, the single family homes adjacent to Royal Avenue, with the exception of the one existing residential structure, were not present after 1965. No significant changes to the site were noted from 1965 to present.

3.2 Previous Studies: Moore Twining reviewed the referenced Phase I report prepared by PSI dated July 20, 2012. A summary of the site history reviewed from the Phase I report is included above in section 3.1 of this report. According to the Phase I report, the site is listed on the LUST, SWEEPS UST, HIST UST, and Hazardous Substance Storage Container Database Cortese associated with former USTs that were present on the subject property. However, the report does not identify the location of the former underground storage tanks. The Phase I report indicates the site was formerly a LUST site due to release of hydrocarbons from three (3) former USTs. According to the Phase I report, monitoring wells and vapor extraction wells were installed and operated for 6 months, and 8,000 cubic yards of hydrocarbon impacted soil was reportedly excavated from the site. The Phase I report did not indicate the precise date of removal of the impacted soils, however, the report indicates the project obtained closure status from the RWQCB on August 16, 1996. The depth and limits of the impacted soil were not described. According to the Phase I report, the site received closure on August 16, 1996 from the RWQCB. If the limits and depths of the previous excavations are available in other documents, these should be provided to Moore Twining for review.

The Phase I report indicates groundwater depths are estimated to be around 25 feet BSG.

No other previous geotechnical engineering, geological, or environmental studies conducted for this site were provided for review during this investigation. In addition, Moore Twining requested historic geotechnical engineering and compaction reports from the City of San Jose Planning, Public Works, and Building Departments, however, no previous soils reports associated with the subject site could be located for review. If available, these reports should be provided for review and consideration for this project.

3.3 Site Description: The project site comprises approximately 5.6 acres located at 720 West San Carlos Street in San Jose, California. A site location map is presented on Drawing No. 1 in Appendix A. The site is bound to the north by West San Carlos Street; to the west by railroad tracks with Los Gatos Creek beyond; to the south by Auzerais Avenue and to the east by Royal Avenue.

At the time of this investigation, the site included three (3) structures. The structures consisted of the existing OSH store within the western portion of the site, the customer pick-up warehouse building in the southwest corner of the site, and an abandoned single family residence in the southeast corner of the site. The remainder of the site is generally covered by existing asphaltic concrete pavement. The existing pavements were noted to be in poor condition. Some chain link fencing was noted around the existing single family residence and at the truck access lane entrance.

Based in our review of the site plan provided, it is our understanding that each of the two (2) existing structures will be demolished to allow for construction of the new building. The existing customer pick up warehouse building is located within the limits of the proposed new building and the abandoned single family residence is located in the proposed parking lot area. It is not known whether the existing sales building will be demolished as part of the proposed new site development.

The existing customer pick-up/receiving building appears to have been constructed in multiple phases. The receiving building included areas of dock high type construction, with concrete slabs on grade, metal framing and metal siding. In addition, a railroad spur was noted adjacent to the west of the existing dock high building. The customer pick up portion of the building was noted to have concrete slabs on grade, metal framing, and metal siding. The abandoned residence in the southeast corner was noted to be of wood frame construction, and appeared to have a raised wood subfloor. It is not known whether basements are present. The existing sales building was noted to have concrete slabs on grade and CMU masonry walls. The type of foundation system supporting the existing structures are not known.

It is anticipated that numerous underground utilities are present within the site. The site topography was relatively flat. A topographic survey may was not provided for review at the time of this report. Based on review of available on-line data, the site elevations appear to ranged from approximately 101 feet AMSL in the northwest to 100 feet AMSL in the southeast portions of the site.

3.4 Anticipated Construction: It is our understanding that the proposed construction will include a new Orchard Supply Hardware store with an attached garden center comprising approximately 47,000 square feet. It is our understanding that the store area will have a plan area of 35,000 square feet and the garden center will comprise 12,000 square feet.

It is anticipated that the proposed construction will consist of a single story building including concrete masonry unit (CMU) wall construction with interior steel columns and concrete slab-on-grade floors. Appurtenant construction is anticipated to include asphaltic concrete and Portland cement concrete pavements, underground utilities, and isolated landscaping areas.

It is anticipated that maximum wall and column loads will be less than 4 kips per lineal foot and 75 kips, respectively. A total allowable static settlement or heave of 1 inch and a maximum allowable differential settlement for floor slabs and foundations of ½ inch in 40 linear feet were assumed for design.

Furthermore, it is our understanding that bio-swales are currently planned in landscaped areas in the proposed parking lot. Details of the proposed bio-swale construction have not been provided to Moore Twining to review at the time of this report.

Cuts and fills in the building pad areas are anticipated to be approximately 2 to 3 feet to achieve the proposed site elevations.

4.0 INVESTIGATIVE PROCEDURES

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

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, soil sampling, and standard penetration tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Amer Razaq of Moore Twining on November 9, 2012 and Mr. Zubair Anwar of Moore Twining on January 30, 2013. The features noted are described in the background information section of this report.

4.1.2 Drilling Test Borings: The depths and locations of the test borings were selected based on the size of the structure, type of construction, estimated depth of influence of the anticipated foundation loads, and the subsurface soil conditions encountered.

On November 9, 2012, six (6) test borings were drilled at the site to depths ranging from about 10 to 51½ feet BSG. After the November 9, 2012 field investigation was performed, the location of the proposed building was revised. Therefore, on January 30, 2013, three (3) additional test borings

were drilled within the limits of the proposed building to depths ranging from 20 to 25 feet BSG. In addition, two (2) shallow test borings were drilled to depths ranging from 2½ to 3 feet BSG within planned landscape areas to obtain samples for laboratory permeability testing. The borings were drilled with a conventional truck-mounted CME-75 drill rig equipped with 6-5/8 inch outside diameter (O.D.) hollow stem augers.

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

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

Test boring locations were determined by pacing with reference to existing site features. The locations, as described, should be considered accurate to within about 10 feet. The locations of the test borings are shown on Drawing No. 2 in Appendix A. The 51½ foot deep test boring was backfilled with neat cement grout in accordance with the requirements of the Santa Clara County Water District permit. The remainder of the test borings were backfilled with on-site cuttings and patched with asphaltic concrete cold patch, thus some settlement should be anticipated.

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

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

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

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

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

5.0 FINDINGS AND RESULTS

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

5.1 Surface Conditions: The site was covered with existing buildings and asphaltic concrete pavements at the time of this investigation. The existing pavements were noted to be in poor condition. It is anticipated that numerous existing underground utilities are present within the limits of the site and proposed building.

5.2 Soil Profile: In general, the near surface soils encountered within the building pad consisted of undocumented fill soils extending from the surface to depths ranging from 3½ to 18½ feet BSG. The undocumented fill soils encountered comprised lean and fat clay soils with variable amounts of brick and wood debris. Below the undocumented fills, native lean clay soils and fat clay soils with relatively thin interbedded layers of clayey sands and well graded sands with silt, were encountered to the maximum depth explored of 51½ feet BSG. Within the limits of the proposed parking areas, below the existing pavements, undocumented fill soils were encountered to depths ranging from 1½ feet to 15 feet BSG. The undocumented fills comprised lean and fat clay soils with gravel, glass, brick, plastic, and metal debris. Below the undocumented fills, native lean and fat clay soils were encountered to the maximum depths explored in the parking area of 21½ feet BSG. It should be noted that due to the variable depth of undocumented fills encountered, deeper fills may be present throughout the site.

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

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

Undocumented Lean and Fat Clay Fills: The undocumented fill soils encountered were described as medium stiff to very stiff, as determined by standard penetration resistance, N-values, ranging from 4 to 21 blows per foot. It should be noted that the standard penetration resistance, N-values, reported for the undocumented fills, appear to be inflated due to the presence of debris such as asphalt. The moisture content of the undocumented fill samples tested ranged from 15 to 28 percent. Two (2) relatively undisturbed samples revealed in-place dry densities of 94.2 and 98.2 pounds per cubic foot. Two (2) expansion indexes performed on fill samples resulted in expansion indexes of 76 and 88. An atterberg limits test resulted in a plasticity index of 31 and liquid limit value of 54.

Native Lean Clays: The native lean clay soils were described as medium stiff to very stiff, as determined by standard penetration resistance, N-values, ranging from 5 to 29 blows per foot. One (1) relatively undisturbed sample revealed an in-place dry density of 108.8 pounds per cubic foot. The moisture content of the samples tested ranged from 11 to 30 percent. Two (2) atterberg limits tests performed on these soils resulted in plasticity indexes of 22 and 27 with liquid limit values of 47 and 47, respectively.

Native Fat Clays: The native fat clay soils were described as medium stiff to stiff, as determined by standard penetration resistance, N-values, ranging from 7 to 15 blows per foot. One (1) relatively undisturbed sample revealed an in-place dry density of 103.4 pounds per cubic foot. The moisture content of the samples tested ranged from 19 to 37 percent. An atterberg limits test performed on one sample resulted in a plasticity index of 34 with a liquid limit value 55. A direct shear test performed on these soils resulted in an internal angle of friction of 20 degrees with a cohesion value of 200 pounds per square foot.

Clayey Sands: The clayey sands encountered were described as dense, as determined by a standard penetration resistance, N-value, of 30 blows per foot.

Well Graded Sands with silt: The well graded sands with silt encountered were described as medium dense, as determined by a standard penetration resistance, N-value, of 15 blows per foot. The moisture content of a sample was 11 percent.

R-Value Tests: Two (2) R-value tests performed on near surface soil samples resulted in R-values of 15 and 17.

Laboratory Permeability Tests: The results of two (2) laboratory permeability tests (ASTM D5084) performed on samples collected between 1 and 2½ feet BSG and 1½ and 3 feet BSG resulted in hydraulic conductivity rates of 1.67×10^{-8} centimeters per second and 1.99×10^{-8} centimeters per second.

Chemical Tests: Chemical tests performed on two (2) near surface soil samples indicated pH values of 7.3 and 7.9; minimum resistivity values of 5,800 and 2,400 ohm-centimeters; 0.019 and 0.027 percent by weight concentrations of sulfate; and a 0.0013 and 0.0014 percent by weight concentration of chloride, respectively.

5.4 Groundwater Conditions: During the November 2012 geotechnical investigation groundwater was encountered at a depth of 29½ feet BSG. After drilling, groundwater was measured at a depth of 45 feet BSG. Approximately 30 minutes after drilling groundwater was measured at 44 feet BSG. Groundwater was not encountered during the January 2013 investigation. Based on review of the Seismic Hazard Map, San Jose West 7.5 Minute Quadrangle, historic high groundwater depths in the vicinity of the site range between 15 and 20 feet BSG.

Based on our review of the State Water Resources Geotracker Website, monitoring wells located between 50 and 200 feet southeast of the site, reported static groundwater levels between about 26 and 29 feet BSG during the years of 2005 and 2010.

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

6.0 EVALUATION

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

6.1 Existing Site Improvements and Undocumented Fills: Undocumented fill soils were encountered throughout the site at depths ranging from 1½ to 18½ feet BSG. The existing undocumented fills are not considered suitable for support of proposed shallow foundations. Therefore, if the building is to be designed and supported on conventional spread foundations, all undocumented fills within influence of the new foundations will need to be over-excavated and the excavations backfilled with engineered fill. The depth of fill encountered in the borings varied significantly. The depth and extent of the undocumented fills are not known. It is possible that the deeper fills encountered are associated with removal of 8,000 cubic yards of soil as part of a soil remediation work conducted around 1996. Information from the soil remediation, such as the limits and depths of the excavation conducted as part of this work, should be provided to our firm for review and consideration of the project. This information would also be helpful for use in developing guidelines for construction estimates and for contractors bidding the work. If information regarding this excavation work is not available, a supplemental investigation could be conducted just prior to construction to estimate the depths and extent of the undocumented fills.

In addition, the existing undocumented fills within the parking area should be removed to reduce the potential for uncontrolled settlement in proposed pavement areas and new underground utilities. However, if the owner is willing to accept risk due to uncontrolled settlement to the pavements, the undocumented fills could remain in-place within the proposed pavement areas that are beyond the influence zone of the building.

Many of the fill samples collected as part of this investigation contained debris, such as metal, wood and brick. Due to the presence of debris, some of the undocumented fills may not be suitable for reuse as engineered fill, unless the oversized material, organics and debris are removed. Therefore, in order to reuse the existing fills, the Contractor should anticipate special procedures, such as hand picking, screening, etc. to remove debris and unsuitable material from these soils prior to use as engineered fill, or the undocumented fills would need to be exported from the site. However, it should be noted that the amount and extent of the debris was not evaluated as part of this investigation. Therefore, the amount of unsuitable debris material greater than 6 inches, organic material, trash, etc., is unknown. In addition, as noted in Section 6.2 of this report, the majority of the onsite soils encountered contained excessive moisture contents to allow compaction as engineered fill. Therefore, the Contractor should anticipate the need for drying/aeration or chemical treatment of the soils prior to use as engineered fill.

Existing structures and underground utilities are located within the limits of the proposed building and site improvements. Proper removal of existing underground utilities, existing foundations, unknown subsurface improvements, etc., and soils which are disturbed from demolition and removal of the existing improvements will be an important aspect of the site preparation. All existing subsurface improvements should be removed and backfilled with compacted engineered fill in accordance with the recommendations provided in the site preparation section of this report.

6.2 Overly Moist Near Surface Soils, Stabilization and Groundwater: The moisture contents of the native and undocumented fill fat clay soils are anticipated to be overly moist to achieve compaction. Therefore, aeration or chemical treatment should be anticipated for the site preparation in order to dry the soils and achieve a moisture content suitable for compaction of the onsite soils as engineered fill. Contractors should anticipate these conditions and include the costs to aerate and/or treat the soils to achieve moisture contents suitable for compaction in their bids. As an alternative, the onsite soils which are overly moist could be exported and replaced with a suitable, imported fill material.

Soils with high moisture contents are also anticipated at the bottom of areas of over-excavation; thus, the excavations are anticipated to require stabilization such as by chemical soil treatment, placement of a bridge lift of geotextile and aggregate base, or a combination of these methods to achieve a stable surface for conducting earthwork operations.

In addition to the wet soil conditions, groundwater was encountered at around 29½ feet BSG in one of the borings conducted for this investigation and historic groundwater levels are reported to be as shallow as 15 feet BSG. Therefore, dewatering may be required along with the stabilization measures for deeper excavations due to the groundwater conditions at the time of construction. Accordingly, contractors should address the requirements for stabilization of wet soils, dewatering, chemical treatment and drying of the onsite soils for use as engineered fill in their bids.

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

In evaluation of the potential for expansive soils at the site, expansion index testing was performed on representative samples of the near surface soils which are anticipated to be within the zone of influence of the planned improvements. The testing was performed in accordance with ASTM D4829. The results of two (2) expansion index tests indicated that the near surface soils have a medium expansion potential, with expansion indexes of 76 and 88. Therefore, recommendations for placement of an imported non-expansive engineered fill below interior and exterior slabs on grade are provided in the Site Preparation section of this report. As an alternative to importing granular fill, it may be possible to chemically treat the onsite soils for use as a non-expansive fill within the building pad. In order to evaluate the feasibility of lime treatment of the onsite soils for this use, laboratory lime suitability testing would be required.

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

It is estimated that foundations supported on the existing undocumented fills may exhibit unpredictable performance and would be anticipated to exceed the assumed tolerable differential static settlement of ½ inch. Accordingly, this report recommends over-excavation and compaction to remove the undocumented fills and support new foundations on engineered fill.

As an alternative to over-excavation of the existing undocumented fills, it may be possible to consider alternative methods of ground improvement such as stone columns or deep soil-cement mixing along with a stiffened foundation and floor slab design. However, the feasibility and cost effectiveness of alternate approaches would need to be further evaluated.

Based upon the minimum footing depths and widths recommended in this report, a net allowable soil bearing pressure of 2,000 pounds per square foot, for dead-plus-live loads, may be used for design. The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and the weight of the footing may be neglected. The net allowable soil bearing pressure presented was selected using the Terzaghi bearing capacity equations for foundations considering a minimum factor of safety of 3.0 and based on the anticipated static settlements noted in this report.

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

6.5 Seismic Ground Rupture and Design Parameters: The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the San Jose Fault located about 2 miles west of the site. In addition, the Silver Creek Fault Zone is located approximately 2.2 miles east of the site. Given that no mapped faults were identified trending through the site, the potential for fault rupture at the site is considered low.

It is our understanding that the 2010 CBC will be used for structural design, and that seismic site coefficients are needed for design.

Based on the 2010 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 1.00, the peak horizontal ground acceleration, as defined in the CBC for liquefaction analysis, was estimated to be 0.40g. Based on the results of hazard deaggregation analysis, a maximum considered earthquake magnitude of 6.7 was estimated for the site.

6.6 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.

One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

Liquefaction/seismic settlement analyses were conducted based on soil properties revealed by test borings and the results of laboratory testing. The evaluations were conducted for soils encountered using the computer program LiquefyPro, developed by CivilTech Software. The peak horizontal ground acceleration of 0.40g, a design earthquake magnitude of 6.7 and the N-values determined from the standard penetration testing were used in the analysis. Based on the historic groundwater data reviewed, an estimated groundwater depth of 15 feet was used in the analysis.

The results of the seismic settlement analysis indicate an estimated total seismic settlement of up to ½ inch and differential seismic settlements of about ¼ inch in 40 feet for the design earthquake. Due to the relatively flat nature of the site, the potential for lateral spreading is considered low.

6.7 Asphaltic Concrete (AC) Pavements: Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highways Design Manual. The analysis was based on traffic index values ranging from 5.0 to 8.0. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

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

The anticipated subgrade soils are silty sands. The subgrade support characteristics of the soils were evaluated by Resistance (R)-value testing. R-value tests conducted on two (2) near surface samples resulted in R-values of 15 and 17. Therefore, an R-value of 15 was used to determine the pavement section thickness recommendations.

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

In evaluation of the pavement design for this project, samples of the onsite soils anticipated to be representative of the soils which will support pavements were obtained and R-value testing performed in accordance with California Test Method 301. The R-value test results are summarized in Appendix C of this report.

The recommendations provided in this report for PCC pavements are based on EALs for traffic index values ranging from 5.0 to 7.0. These traffic index values were converted to the equivalent number of 5-axle trucks per day for the analysis. The PCC pavement sections were designed for a life of 20 years, a load safety factor of 1.1, a single axle weight of 12,000 pounds, and two tandem axle weights of 34,000 pounds, each.

The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements." A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 6-inch layer of Class 2 aggregate base material (minimum R-value of 78), of 150 psi/in was used for pavement design.

6.9 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. Corrosion can eventually damage or destroy a metallic object.

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

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

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

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

7.0 CONCLUSIONS

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

- 7.1 The site is considered suitable for the proposed construction with regard to support of the proposed structure, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and observation of earthwork and foundation activities by Moore Twining are integral to this conclusion.
- 7.2 The near surface soils encountered within the building pad consisted of undocumented fill soils extending from 3½ to 18½ feet BSG. The undocumented fill soils encountered were comprised of lean and fat clay soils with variable amounts of debris such as metal, brick, asphalt and wood. Below the undocumented fills, native lean clay soils and fat clay soils with relatively thin interbedded layers of clayey sands and well graded sands with silt, were encountered to the maximum depth explored of 51½ feet BSG. Undocumented fill soils were also encountered in the proposed pavement areas to depths ranging from 1½ feet to 15 feet BSG. The undocumented fills encountered comprised lean and fat clay soils with gravel, glass, brick, plastic, and metal debris. Below the undocumented fills, native lean and fat clay soils were encountered in the parking area.
- 7.3 The undocumented fills encountered are not considered suitable for support of proposed foundations. In order to reduce the potential for excessive, uncontrolled differential settlement of the proposed building, recommendations are included in this report for removal of the existing undocumented fill within the building pad area and support the proposed structure on engineered fill. As an alternative to over-excavation of the existing undocumented fills, it may be possible to consider alternative methods of ground improvement such as stone columns or deep soil-cement mixing along with a stiffened foundation and floor slab design. However, the feasibility and cost effectiveness of alternate approaches would need to be further evaluated.

- 7.4 In order to prevent the risk of excessive distress of improvements outside the building pad limits as a result of settlement of the undocumented fills, the undocumented fills would need to be over-excavated from areas of proposed improvements outside the building pad. However, considering the age of the fill and given that similar site work improvements (i.e., pavements) are currently supported on the undocumented fills, the existing undocumented fills could be left in place below improvements outside the building pad if the project stakeholders accepted the risk that settlement, distress and higher maintenance would be required for improvements supported on the undocumented fills outside the building pad.
- 7.5 The onsite soils encountered and tested are overly moist, and therefore the on-site soils are anticipated to require aeration or chemical treatment to dry the soils in order to allow compaction as engineered fill. In addition, soils exposed in the bottom of excavations will be overly moist and are anticipated to require stabilization to achieve a firm bottom. Stabilization could be conducted by chemical soil treatment or bridge lifts using a 12 or 18 inch thick section of rock encapsulated in geotextile fabric.
- 7.6 Provided the subgrade soils are prepared as recommended in this report, proposed new foundations may be designed for total and differential static settlements of 1 inch total and ½ inch differential in 40 feet. In addition, total and differential seismic settlements of ½ inch and ¼ inch in 40 feet, respectively should be considered for design.
- 7.7 The near surface soils tested exhibited a medium expansion potential and poor support characteristics for pavements when compacted as engineered fill.
- 7.8 During the November 2012 geotechnical investigation, groundwater was encountered at a depth of 29½ feet BSG. Based on review of the Seismic Hazard Map, San Jose West 7.5 Minute Quadrangle, historic high groundwater depths in the vicinity of the site are between 15 and 20 feet BSG.
- 7.9 The potential for fault rupture is low. Based on the 2010 CBC and considering a five percent damped design spectral response acceleration for short period (S_{DS}) of 1.000, the CBC peak horizontal ground acceleration was estimated to be 0.40g.
- 7.10 The onsite clay soils possess a very low permeability and are not appropriate for infiltration of storm water. Limited laboratory testing conducted on a near surface sample indicated a saturated hydraulic conductivity of 1.67×10^{-8} centimeters per second and 1.99×10^{-8} centimeters per second. Based on the results of the laboratory classification tests performed and laboratory permeability tests, the near surface clay soils tested are classified as being a NRCS Hydrologic Soil Group D soil type.

- 7.11 Chemical testing of soil samples indicated the soils exhibit a “corrosive” to “moderately corrosive” corrosion potential.
- 7.12 Chemical analyses indicated a negligible potential for sulfate attack on concrete placed in contact with the near surface soils.

8.0 RECOMMENDATIONS

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

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 Cuts and fills in the building pad areas are anticipated to be around 2 to 3 feet. If the finished grades are higher or lower than assumed, the recommendations presented may not be appropriate. Moore Twining should be provided the opportunity to review the final grading plans and foundation plans before the plans are released for bidding purposes so that any relevant recommendations can be presented.
- 8.1.2 This report was prepared based on assumed foundation loads noted in the Anticipated Construction section of the report. When the actual foundation loads are known, this information should be provided to Moore Twining for review. In the event the foundation loads are different than assumed, the minimum recommended over-excavation depth may need to be revised.
- 8.1.3 In order to reduce the potential for heave of concrete slabs on grade due to expansive soils, this report includes recommendations included in the Site Preparation section of this report to support proposed interior and exterior slabs on imported non-expansive engineered fill. As an alternative to importing granular fill, it may be possible to chemically treat the onsite soils for use as a non-expansive fill within the building pad. In order to evaluate

the feasibility of lime treatment of the onsite soils for this use, laboratory lime suitability / mix design testing would be required. Approximately 2 ½ weeks should be allowed for the testing.

- 8.1.4 A preconstruction meeting including, as a minimum, the owner, general contractor, earthwork contractor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling.
- 8.1.5 Numerous underground utilities were noted within the limits of the proposed exterior concrete slabs on grade and drive thru lane. Moore Twining is not aware of documentation of compaction testing of the trench backfill soils. In the event the existing trench backfill was not compacted as engineered fill, proposed improvements constructed over the existing utilities may be subject to future settlement which could result in distress to overlying improvements. The recommendations in this report include over-excavation and compaction of all existing utility trenches which are parallel to and are located within a 1.5H:1V plane from new foundations. The recommendations for subgrade preparation outside the 1.5H:1V influence line from foundations assume this risk is acceptable to the owner. If this risk is not acceptable to the owner, the existing backfill could be removed and replaced with on-site soils compacted as engineered fill and this requirement would need to be provided to contractors bidding the project. Alternatively, the trench backfill could be assessed by in-place density testing to evaluate the characteristics of the backfill and better quantify the potential for future settlement.
- 8.1.6 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.
- 8.1.7 The contractor should use appropriate equipment such as low-pressure equipment, steel tracks, etc. to achieve the required over-excavation, compaction and site preparation to minimize rutting and subgrade instability.
- 8.1.8 Contractors should be aware that the on-site soils contained high moisture contents and that drying of the soils to achieve compaction and stabilization measures to achieve the recommendations of this report will be required. Soil drying and/or chemical treatment for stabilization should be anticipated to achieve the required relative compaction and stable conditions.

- 8.1.9 Historic high groundwater levels are indicated to range from about 15 to 20 feet below grade in the site area. Dewatering should be anticipated for excavations which approach groundwater.

8.2 Site Grading and Drainage

- 8.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least five feet away from the structure, or as necessary to preclude ponding of water adjacent to foundations, whichever is more stringent. Adjacent exterior grades which are paved should be sloped at least 1 percent away from the foundations.
- 8.2.2 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from proposed structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.
- 8.2.3 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and should establish positive drainage of water away from the structure. Care should be taken to maintain a leak-free sprinkler system.
- 8.2.4 To prevent the migration of water, the curbs where asphaltic concrete or PCC pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.
- 8.2.5 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements are recommended.
- 8.2.6 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system. As an alternative, the roof drains should extend a minimum of 5 feet away from the structure and the resulting runoff directed away from the structure at a minimum of 2 percent.

- 8.2.7 It is our understanding that bioswales are planned within the proposed landscape areas, including adjacent to the proposed garden center area. Sufficient setbacks from the bioswales to existing improvements should be maintained, and measures such as deepened curbs, cutoffs, liners, etc. should be incorporated in the designs to reduce the potential for excessive settlement/heave of adjacent improvements due to moisture and freewater migration from bioswales or other storm water pre-treatment and disposal systems. Moore Twining should be provided the proposed details of these features for review and comment.

8.3 Site Preparation

- 8.3.1 Stripping shall be conducted to remove surface vegetation and root systems (if any). The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoils. The stripping should be reviewed by our firm at the time of construction. Stripping and clearing of debris should extend laterally a minimum of 10 feet outside the new footing and pavement perimeters. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner. It should be anticipated that topsoil will settle about 1 inch per foot of thickness as a result of decay of organic material.
- 8.3.2 Existing utilities within a 1.5 Horizontal to 1 Vertical plane from the top of new building slabs and/or foundations should be removed in their entirety and all backfill associated with these utilities should be over-excavated and backfilled as engineered fill. Utilities should be completely removed and should not be crushed and buried in-place. All excavations should be backfilled as engineered fill. Portions of utilities lines abandoned in place beyond the removal limits should be plugged with concrete.
- 8.3.3 Existing utilities not scheduled to remain should be removed in their entirety and all backfill and bedding material associated with these utilities should be over-excavated and backfilled as engineered fill. Utilities should be completely removed and disposed of off-site and should not be crushed and buried in-place. If utilities are encountered and are scheduled to be removed, loosened soils resulting from the removal of the utilities should be over-excavated to at least 12 inches below the bottom of the utilities, moisture conditioned, and compacted as engineered fill.
- 8.3.4 After site stripping, removal of surface improvements, removal of trees (if any) and associated root systems, subsurface improvements (if any), over-excavation should be conducted throughout the building pad and over-build

zone to a depth of at least 24 inches below bottom of footings, to a minimum depth of 4 feet below preconstruction site grades, to the depth required to remove existing undocumented fills (encountered to depths of up to 18½ feet BSG), or to at least 12 inches below the bottom of improvements to be removed (if any), whichever is greater. Differential fill thickness below each building pad should be less than one (1) foot vertical over five (5) feet horizontal, whichever is greater. The over-excavation shall include the entire building footprint, foundations, all concrete slabs directly adjacent to the building, and a minimum of 5 feet horizontally beyond these improvements or by the horizontal distance equal to the depth of over-excavation required below the foundations, whichever is greater. Due to the depth of over-excavation anticipated to remove the undocumented fills (encountered as deep as 18½ feet BSG), the horizontal limits of over-excavation will need to extend up to 18½ feet beyond the foundations in some areas. Slot cutting only below foundations will not be allowed. Upon review of the survey data showing the depth and over-excavation limits provided by the Contractor and confirmation of removal of the undocumented fills based on observation of the over-excavation by a Moore Twining engineer or geologist, the bottom of the excavation should be processed by scarification to a minimum depth of 8 inches, moisture conditioned to 1 to 4 percent above optimum moisture content, and compacted as engineered fill. Where wet, unstable soil conditions are encountered, the bottom of the over-excavation should be stabilized by chemical soil treatment, geotextile fabric and aggregate base, or other methods which are submitted and approved by Moore Twining.

After over-excavation and compaction of engineered fill in the building pad, the concrete slabs on grade located within the building pad should be supported on a minimum of 4 inches of non-recycled Class 2 aggregate base over a minimum of 26 inches of imported non-expansive engineered fill.

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

- 8.3.6 After stripping and removal of existing improvements, miscellaneous lightly loaded foundations (i.e. retaining walls, screen walls, etc.) should be over-excavated to a minimum of 12 inches below foundations, a minimum of 24 inches below preconstruction site grade, to the depth required to remove all existing on-site fills (encountered to depths ranging from about 1 ½ to 18½ feet BSG), or to a minimum of 12 inches below subsurface structures to be removed, whichever provides the deeper fill. The over-excavation limits should extend a minimum of 5 feet beyond the limits of the foundations on all sides. After over-excavation, the bottom should be scarified, moisture conditioned to between 1 and 4 percent above optimum and compacted to a depth of 8 inches prior to fill placement. Upon review of the survey data showing the depth and over-excavation limits provided by the Contractor and confirmation of removal of the undocumented fills based on observation of the over-excavation by a Moore Twining engineer or geologist, the bottom of the excavation should be processed by scarification to a minimum depth of 8 inches, moisture conditioned to 1 to 4 percent above optimum moisture content, and compacted as engineered fill. Where wet, unstable soil conditions are encountered, the bottom of the over-excavation should be stabilized by chemical soil treatment, geotextile fabric and aggregate base, or other methods which are submitted and approved by Moore Twining.
- 8.3.7 If the increased risk of distress to paving and flatwork areas outside the building pad due to the unpredictable performance of the undocumented fills is acceptable, areas to receive exterior slabs and pavements outside the building pad preparation zone should be over-excavated to a minimum depth of 12 inches below preconstruction site grades, 12 inches below improvements to be removed, to the bottom of the proposed aggregate base section for pavements and to the bottom of the recommended non-expansive section for exterior slabs and PCC slabs, whichever is greater. If the risk of distress due to the undocumented fills is not acceptable to the project stakeholders, exterior slab and pavement areas outside the building pad should be over-excavated to a minimum of 12 inches below preconstruction site grades or to the depth required to remove undocumented fills (encountered outside the building pad at depths ranging from 1½ to 15 feet BSG), whichever is greater. Prior to placement of engineered fills, the bottom of the excavation should be processed by scarification to a minimum depth of 12 inches, moisture conditioned to 1 to 4 percent above optimum moisture content, and compacted as engineered fill. Where wet, unstable soil conditions are encountered, the bottom of the over-excavation should be stabilized by chemical soil treatment, geotextile fabric and aggregate base, or other methods which are submitted and approved by Moore Twining.

- 8.3.8 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.
- 8.3.9 The contractor should locate all on-site water wells (if any). All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Moore Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters) should have their casings removed to a depth of at least 5 feet below preconstruction site grades or finished pad grades, whichever is deeper. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill.
- 8.3.10 The moisture content and density of the compacted soils should be maintained between 1 and 4 percent above optimum until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.
- 8.3.11 Final grading shall produce a building pad ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad (before aggregate base is placed) shall not depress more than one-half ($\frac{1}{2}$) inch under the wheels of a fully loaded water truck, or equivalent loading. If depressions more than one-half ($\frac{1}{2}$) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the owner.
- 8.3.12 The Contractor is responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor is responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

8.4 Engineered Fill

- 8.4.1 The on-site near surface soils encountered are predominantly lean and fat clays that are not recommended for use as engineered fill within the upper 30 inches below concrete slabs on grade within the building pad limits, nor within the upper 18 inches below the bottom of concrete slabs and PCC

pavements outside the building pad preparation limits. The on-site clay fill soils that are free of organics (less than 3 percent by weight), irreducible particles larger than 6 inches in largest dimension and free of debris will be suitable for use as fill at depths greater than 30 inches below the bottom of concrete slabs on grade within the building pad limits and below a depth of 18 inches below exterior slabs on grade and PCC pavements, provided they are properly moisture conditioned and compacted. The native soils should be moisture conditioned (wetted or aerated) to between 1 and 4 percent above optimum moisture content and compacted to at least 92 percent relative compaction. If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

The Contractor should note that debris such as concrete, asphaltic concrete, plastic, metal, etc. was encountered within the undocumented fills and there is a potential that debris and unsuitable material may be present in the fills which will need to be removed prior to reuse of these soils as engineered fill. Accordingly, prior to reuse as engineered fill, oversize debris or organics may need to be removed from the undocumented fills prior to use of the material as engineered fill, such as by hand picking, screening or other methods.

- 8.4.2 The compactability of the onsite soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.
- 8.4.3 Import fill soil should be non-expansive, non-recycled and granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	75 - 100
Percent Passing No. 200 Sieve	10 - 40
Plasticity Index	Less than 15
Expansion Index	Less than 15
Organics	Less than 3 percent by weight
R-Value	Minimum 30
Sulfates	< 0.05 percent by weight
Min. Resistivity	> 10,000 ohms-cm

Prior to importing fill, the Contractor shall submit test data that demonstrates that the proposed import complies with the recommended criteria for both

geotechnical and environmental criteria. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner and Moore Twining) that the soils do not contain any environmental contaminants regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, recent analytical data specific to the source of the import material including proper chain-of-custody documentation. The Contractor shall provide the owner and Moore Twining with the list of constituents to be tested for review and approval. Upon review and approval of satisfactory environmental and geotechnical test results, the Contractor may transport the material to the site at their own risk. Moore Twining will sample and test the material after the submittal is approved to verify that the proposed material complies with the requirements of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested. The Contractor shall pay for any environmental and geotechnical testing required to determine compliance with the requirements of this report.

- 8.4.4 Native soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within 1 and 4 percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density for the upper 10 feet BSG and 95 percent of the maximum dry density for fills placed greater than 10 feet BSG, as determined by ASTM Test Method D1557. Also, the upper 12 inches of subgrade soils below pavements should be compacted to a minimum of 95 percent of the maximum dry density based on ASTM D 1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.
- 8.4.5 Imported, non-expansive engineered fill soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to three (3) percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density for the upper 10 feet BSG and 95 percent of the maximum dry density for fills placed greater than 10 feet BSG, as determined by ASTM Test Method D1557. Also, the upper 12 inches of subgrade soils below pavements should be compacted to a minimum of 95 percent of the maximum dry density based on ASTM D 1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.
- 8.4.6 Open graded gravel and rock material such as $\frac{3}{4}$ -inch crushed rock or $\frac{1}{2}$ -inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill

(Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining.

- 8.4.7 Aggregate base shall comply with the State of California Department of Transportation requirements for Class 2 aggregate base and should not include recycled materials where used below the building. Aggregate base shall be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all requirements for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.
- 8.4.8 Recycled materials cannot be used in the building pad and overbuild zone if they contain asphaltic concrete materials.

8.5 Foundations

- 8.5.1 Perimeter foundations shall have a minimum depth of 30 inches below the bottom of the slab on grade or 30 inches below lowest adjacent grade, whichever is greater. Interior footings should have a minimum depth of 18 inches below the bottom of the slab-on-grade, or 18 inches below the lowest adjacent grade, whichever is greater. All footings should have a minimum width of 15 inches, regardless of load.
- 8.5.2 Foundations may be supported on spread or continuous footings placed entirely on engineered fill prepared in accordance with the recommendations section of this report entitled, "Site Preparation." Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.
- 8.5.3 Static settlements of 1 inch total and ½ inch differential in 40 feet should be anticipated for design of foundations supported as recommended herein. In addition, anticipated seismic settlements of ½ inch total and ¼ inch differential in 40 feet should be considered for design.

- 8.5.4 The foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.
- 8.5.5 Foundation excavations or exposed soils should not be left uncovered and allowed to dry such that the moisture content of the soils is less than optimum moisture content or drying produces cracks in the soils. The exposed soils, such as sidewalls, excavation bottoms, etc. should be continuously moistened to maintain the moisture content at least one percent above optimum until concrete is placed.
- 8.5.6 Structural loads for miscellaneous foundations (such as retaining walls, sound walls, screen walls, monument and pylon signs, etc.) should be evaluated on a case by case basis to present supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, miscellaneous foundations may be supported on spread or continuous footings extending a minimum depth of 18 inches which are supported entirely on engineered fill prepared in accordance with the recommendations section of this report entitled, "Site Preparation." Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.
- 8.5.7 The following factors were developed based on the tables in Chapter 16 of the 2010 CBC and the digitized active fault locations published by USGS.

Seismic Factor	2010 CBC Value
Site Class	D
Spectral Response At Short Period (0.2 Second), S_s	1.500
Spectral Response At 1-Second Period, S_1	0.600
Site Coefficient (based on Spectral Response At Short Period), F_a	1.000

Seismic Factor	2010 CBC Value
Site Coefficient (based on spectral response at 1-second period) F_v	1.500
Maximum considered earthquake spectral response acceleration for short period, S_{Ms}	1.500
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	0.900
Five percent damped design spectral response accelerations for short period, SD_s	1.000
Five percent damped design spectral response accelerations at 1-second period, SD_1	0.600

8.5.8 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement. The contractor shall provide a minimum of 48 hours notice for these observations.

8.5.9 Cast-in-drilled-hole reinforced concrete foundations (pier) for support of signage and lighting may be based on an allowable skin friction of 150 pounds per square foot per foot of embedment to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained procedure (Section 1806.8.2.1). An allowable passive value of 200 pounds per square foot per foot of depth may be used.

8.6 Frictional Coefficient and Earth Pressures

8.6.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.34 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

- 8.6.2 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 200 pounds per cubic foot. A minimum factor of safety of 1.5 should be used when combining the frictional and passive resistance of the soil to determine the total lateral resistance. The upper 6 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance.
- 8.6.3 The active and at-rest pressures of the engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 61 and 81 pounds per cubic foot, respectively. These pressures assume the use of an imported granular backfill as specified in section 8.7.3 of this report. These pressures also assume level ground surface and do not include the surcharge effects of sloping backfill, construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.
- 8.6.4 The active and at-rest pressures were calculated based on a maximum soil unit weight of 130 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 130 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 130 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of over 130 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind retaining walls.
- 8.6.5 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.
- 8.6.6 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a backdrain as recommended in this report.
- 8.6.7 Since the pressures recommended in this section do not include vehicle surcharges, it is recommended to use lighter hand operated or walk behind compaction equipment to avoid wall damage during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure.

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

8.7 Retaining Walls/Screen Walls

- 8.7.1 Retaining wall plans, when available, should be reviewed by Moore Twining to evaluate the actual backfill materials, proposed construction, drainage conditions, and other design geotechnical parameters.
- 8.7.2 Structural loads for retaining walls/screen walls may be designed based on the recommendations included in Section 8.5.6 of this report. Retaining wall foundations should be supported on engineered fill placed in accordance with the recommendations in the Site Preparation section of this report.
- 8.7.3 Retaining walls should be constructed with imported, non-expansive 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 backfill will reduce the effects of swell pressures on the wall. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	10 - 30
Plasticity Index	Less than 10
Internal Angle of Friction	30 degrees (minimum)

- 8.7.4 The import fill material should be tested and approved as indicated under the Engineered Fill section of this report.
- 8.7.5 Segmented wall design (mechanically stabilized walls) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. None of the data included in this report should be used for segmented wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. The wall designer should perform sufficient observations of the wall construction to certify that the wall was constructed in accordance with the design plans and specifications.

- 8.7.6 Retaining walls should be constructed with a drainage system including perforated pipe surrounded by at least 1 cubic foot of crushed rock or class 2 permeable material. If open graded materials such as crushed rock are used as drain material surrounding drain pipes, these materials should be fully encased in filter fabric such as Mirafi 140 N and vibrated in place to a non-yielding condition under the observation of the geotechnical engineer. The final selection of filter fabric should be as recommended by the fabric manufacturer for the specific site conditions. A Caltrans Class 2 permeable material, installed without the use of filter fabric, is preferable to open graded material as it presents a lower potential for clogging than the filter fabric. Class 2 permeable material should be compacted to 95 percent relative compaction in accordance with ASTM D1557. Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain or subdrain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system did not function properly. 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.
- 8.7.7 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.
- 8.7.8 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.

8.8 Interior Slabs-on-Grade

The slabs on the project that should be prepared and supported as recommended for interior slabs include interior floor slabs and all concrete slabs on grade adjacent to the building.

- 8.8.1 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade based on a total and differential settlement and heave of 1 inch and ½ inch, respectively.
- 8.8.2 It is recommended interior concrete slabs-on-grade be supported on a minimum of 4 inches of non-recycled aggregate base over a minimum of 26 inches of imported non-expansive engineered fill over the depth of engineered fill recommended below the foundations. The minimum 4 inches of aggregate base sand is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction purposes. The aggregate base should be moisture conditioned to slightly above optimum moisture content, and compacted to a minimum of 95 percent relative compaction. Aggregate base used below the slabs-on-grade within the building pad and overbuild zone should not contain any asphalt materials.
- 8.8.3 The recommendations provided herein are intended only for the design of interior concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, cement mixers, and rock trucks, etc.). The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.
- 8.8.4 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.
- 8.8.5 The moisture content of the imported non-expansive engineered fill below the aggregate base section should be verified to be slightly above optimum moisture content prior to placing the aggregate base section, and also within 48 hours of placement of the vapor retarding membrane or the concrete for the slab-on-grade if a vapor retarding membrane is not used. The moisture content of the upper 12 inches of the subgrade soils should be tested and confirmed to be between 1 and 4 percent above optimum prior to placement of imported non-expansive engineered fill.

- 8.8.6 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarding membrane when the potential exists that the underlying subgrade or granular layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 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 should overlay the compacted base or sand layer. Alternative vapor retarding membranes may be considered. If vapor retarding membranes other than recommended in this report are proposed for use, they should be submitted to Moore Twining for review and consideration. It should be noted that placing the PCC slab directly on the vapor retarding membrane 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 aggregate base layer and the concrete can be placed directly on the Stegowrap. If sand is placed above the membrane, the potential for increased moisture vapor emissions may be realized. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. We recommend 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 retarding membrane 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.

- 8.8.7 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.
- 8.8.8 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.
- 8.8.9 The moisture retarding membrane is not required beneath exposed concrete floors, such as warehouses and garages, provided that moisture intrusion into the structure are permissible for the design life of the structure.
- 8.8.10 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.
- 8.8.11 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity, etc., at a frequency and method as specified by the flooring manufacturer or as required by the plans and specifications, whichever is most stringent. The results of vapor transmission tests, pH tests, internal relative humidity tests, ambient building conditions, etc. should be within floor manufacturer's and adhesive manufacturer's specifications at the time the floor is placed. It is recommended that the floor manufacturer and subcontractor review and approve the test data prior to floor covering installation.
- 8.8.12 To reduce the potential for damaging slabs during construction, the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to interior columns; 2) provide an aggregate base layer below the slabs. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be considered in the design of the slabs and evaluated by the contractor prior to loading the slab. If cranes are to be used, the contractor should assess the capacity of the floor slab to support these loads.

- 8.8.13 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.
- 8.8.14 If the subgrade or base materials below the slab section will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The subgrade can experience instability under construction loading.
- 8.8.15 Aggregate base shall comply with State of California Department of Transportation requirements for Class 2 aggregate base or The Crushed Aggregate Base (CAB) from the Standard Specifications for Public Works Construction. Aggregate base shall be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all requirements for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.9 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc. outside the overbuild zone. The slabs on the project to be prepared as exterior flatwork include: all sidewalks not including the store front, sidewalks adjacent to the building and other slabs adjacent to the building.

- 8.9.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.
- 8.9.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the exterior slabs should be supported on a minimum of 4 inches of Class 2 aggregate base over a minimum of 14 inches of imported non-expansive engineered fill over subgrade soils prepared in accordance with the recommendations provided in the "Site Preparation" section of this report.

- 8.9.3 The moisture content of the subgrade should be verified to be between 1 and 4 percent above the optimum moisture content prior to placement of the imported non-expansive fill. The imported non-expansive engineered fill should be verified to be slightly above optimum placing the aggregate base, and also within 48 hours of placement of the slab-on-grade. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.
- 8.9.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the non-expansive fill section.
- 8.9.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing non-expansive materials and/or concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing non-expansive fill and concrete flatwork. Written test results indicating passing density and moisture tests should be in the general contractor's possession prior to placing concrete for exterior flatwork.

8.10 Asphaltic Concrete (AC) Pavements

- 8.10.1 Areas to receive pavements should be prepared in accordance with the recommendations in the site preparation section of this report (8.3).
- 8.10.2 The following pavement sections are based on an R-value of 15 and traffic index values ranging from 5.0 to 8.0. It should be noted that if pavements are constructed prior to the building construction, the traffic index value should account for construction traffic. The actual traffic index values applicable to the site should be determined by the project civil engineer.

Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Minimum Compacted Subgrade, inches
5.0	2.5	9.0	12
5.5	3.0	10.0	12
6.0	3.0	11.5	12

Traffic Index	AC thickness, inches	AB thickness, inches	Minimum Compacted Subgrade, inches
6.5	3.5	12.5	12
7.0	4.0	14.0	12
8.0	4.5	15.5	12

AC - Asphaltic Concrete compacted per 8.10.9 of this report
 AB - Class II Aggregate Base with minimum R-value of 78 and compacted to at least 95 percent relative compaction (ASTM D1557)
 Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

- 8.10.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.
- 8.10.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.
- 8.10.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.
- 8.10.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.
- 8.10.7 Pavement materials and construction method should conform to Sections 26 and 39 of the State of California Standard Specification Requirements.
- 8.10.8 It is recommended that the base 2 inch thick course of asphaltic concrete consist of a $\frac{3}{4}$ inch maximum medium gradation. The top course or wear course should consist of a $\frac{1}{2}$ inch maximum medium gradation.
- 8.10.9 The asphaltic concrete, including joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent, and no single test value being above a relative compaction of 97 percent, of the referenced laboratory density according to AASHTO T209 or ASTM D2041.

8.10.10 The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. The Contractor shall provide an asphalt concrete mix design prepared and signed by a California registered civil engineer and approved by Moore Twining and the developer prior to construction.

8.11 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. The PCC pavement design assumes a minimum modulus of rupture of 550 psi. The design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

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

8.11.2 The subgrade soils should be not be allowed to dry prior to placement of the aggregate base. The moisture content of the subgrade soils below the aggregate base should be verified to be between 1 and 4 percent above optimum moisture content to a minimum depth of 12 inches prior to placing the recommended aggregate base, and also within 48 hours of placement of the PCC pavements. If necessary to achieve the recommended moisture content, the subgrade should be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.11.3 The following pavement section designs are based on a design modulus of subgrade reaction K-value of 150 psi/in, considering a recommended 6-inch layer of Class 2 aggregate base material (minimum R-Value of 78) over the native compacted soil. The design thicknesses were prepared based on traffic indexes ranging from 5.0 to 8.0, and the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," assuming the following: 1) minimum of 3,500 psi concrete, 2) load transfer by aggregate interlock or dowels, 3) no concrete shoulder, 4) a load safety factor of 1.1, and 5) truck loading consisting of 1 single axle load of 12 kips and tandem axle loads of 18 kips, each.

Portland Cement Concrete Pavement Section Thicknesses

Traffic Index	PCC Layer Thickness (inches)	AB Layer Thickness (inches)	Imported Non-Expansive Engineered Fill (inches)	Compacted Subgrade (inches)
5.0	6.0	6.0	12.0	12.0
6.0	6.5	6.0	12.0	12.0
7.0	6.5	6.0	12.0	12.0
8.0	7.0	6.0	12.0	12.0

ADTT - Average Daily Truck Traffic based on a loaded semi-tractor trailer
PCC - Portland Cement Concrete (minimum Modulus of Rupture=550 psi)
AB - Aggregate Base compacted to at least 95 percent relative compaction (ASTM D1557)
Import - Compacted to a minimum of 95 percent relative compaction
Subgrade - Minimum depth of subgrade soils prepared and compacted in accordance with the recommendations in the Site Preparation section of this report.

- 8.11.4 The PCC pavements adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the aggregate base section.
- 8.11.5 The PCC pavement should be constructed in accordance with the American Concrete Institute requirements, the requirements of the project plans and specifications, whichever is the most stringent. The pavement design engineer should include appropriate construction details and specifications for construction joints, contraction joints, joint filler, concrete specifications, curing methods, etc.
- 8.11.6 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 550 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, etc. should be provided by the designer of the PCC slabs.
- 8.11.7 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or 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.

- 8.11.8 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.
- 8.11.9 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.
- 8.11.10 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.
- 8.11.11 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.
- 8.11.12 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.
- 8.11.13 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.
- 8.11.14 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.
- 8.11.15 Pavement construction should conform to Sections 40 and 90 of the State of California Standard Specifications.

8.12 Slopes and Temporary Excavations

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

- 8.12.2 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.
- 8.12.3 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.
- 8.12.4 Shoring should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Moore Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.
- 8.12.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.13 Utility Trenches

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

excavate, stockpile and backfill the non-expansive fill such that the non-expansive fill material is replaced in the upper section of the trench backfill to match the thickness non-expansive fill section recommended in this report. The onsite clayey soils should not be used as fill within the recommended non-expansive fill section present below the slabs on grade. In addition, if the non-expansive fill materials are blended with onsite clay soils during trench excavation, the mixed soils will not be acceptable for final trench backfill within the recommended non-expansive fill section and an import, non-expansive fill meeting the recommendations of this report (see Engineered Fill section) will be required.

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

- 8.13.4 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of select sand with a minimum sand equivalent of 30, 100 percent passing the 1/4 inch sieve, a minimum of 90 percent passing the No. 4 sieve and not more than 10 percent passing the No. 200 sieve. The sand shall be placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) sand meeting the above sand equivalent and gradation requirements for select sand bedding. The width of the trench should meet the requirements of ASTM D2321-00 listed in Table No. 1 (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. 1
Minimum Trench Widths for HDPE Pipe with
Sand Bedding Initial Backfill**

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

- 8.13.5 Open graded gravel and rock material such as 3/4-inch crushed rock or 1/2-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used

without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining.

- 8.13.6 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to within optimum to 3 percent above the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.
- 8.13.7 Trench backfill should be placed in 8 inch lifts, moisture conditioned as recommended for engineered fill soils (See Section 8.4) and compacted to achieve the minimum relative compaction. Lift thickness can be increased if the contractor can demonstrate the minimum compaction requirements can be achieved.
- 8.13.8 On-site soils that are free of organics (less than 3 percent by weight) and free of debris may be used as final backfill (12 inches above the pipe to the ground surface) in trenches below the specified imported, non-expansive fill sections.
- 8.13.9 Jetting of trench backfill is not allowed to compact the backfill soils.
- 8.13.10 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 create a cutoff and prevent the trench from acting as a conduit to exterior surface water.
- 8.13.11 Storm drains and/or utility lines should be designed to be “watertight.” If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil movement causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. The Contractor is required to video inspect or pressure test the wet utilities prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are “watertight.” The Contractor shall provide the Owner a copy of the results of the testing. The Contractor is required to repair all noted deficiencies at no cost to the owner.

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

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

8.14 Corrosion Protection

8.14.1 Based on the ASTM Special Technical Publication 741 and the analytical results of sample analyses indicate the sample tested had a resistivity values of 2,400 and 5,800 ohm-centimeters, with pH values of 7.3 and 7.9. Based on the resistivity value, the soils exhibit a “corrosive” to “moderately corrosive” corrosion potential. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a “corrosive” corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.14.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (0.019 and 0.027 percent by dry weight concentrations of sulfate). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs are needed due to the sulfate content.

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

recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

9.0 DESIGN CONSULTATION

- 9.1 Moore Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.
- 9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.
- 9.3 If Moore Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

- 10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design.
- 10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.
- 10.3 In-place density tests should be conducted in accordance with ASTM D6938 (nuclear methods) at a frequency of at least:

Area	Minimum Test Frequency
Mass Fills or Subgrade	1 test per 2,500 square feet per compacted lift
Pavement Subgrade	1 test per 5,000 square feet per compacted lift
Utility Lines	1 test per 150 feet per lift

- 10.4 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.
- 10.5 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.
- 10.6 If Moore Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structure or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance of at least \$3,000,000 and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.
- 10.7 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to provide this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

- 11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations.

- 11.2 The nature and extent of subsurface variations between borings may not become evident until construction.
- 11.3 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.
- 11.4 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.
- 11.5 Changed site conditions, or relocation of proposed structure, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.
- 11.6 The conclusions and recommendations contained in this report are valid only for the project discussed in Section 3.4, Anticipated Construction. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in Section 3.3, Site Description is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.
- 11.7 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.
- 11.8 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

- 11.9 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.
- 11.10 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

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

Sincerely,
MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

DRAFT

Dean B. Ledgerwood II, PG
Project Geologist

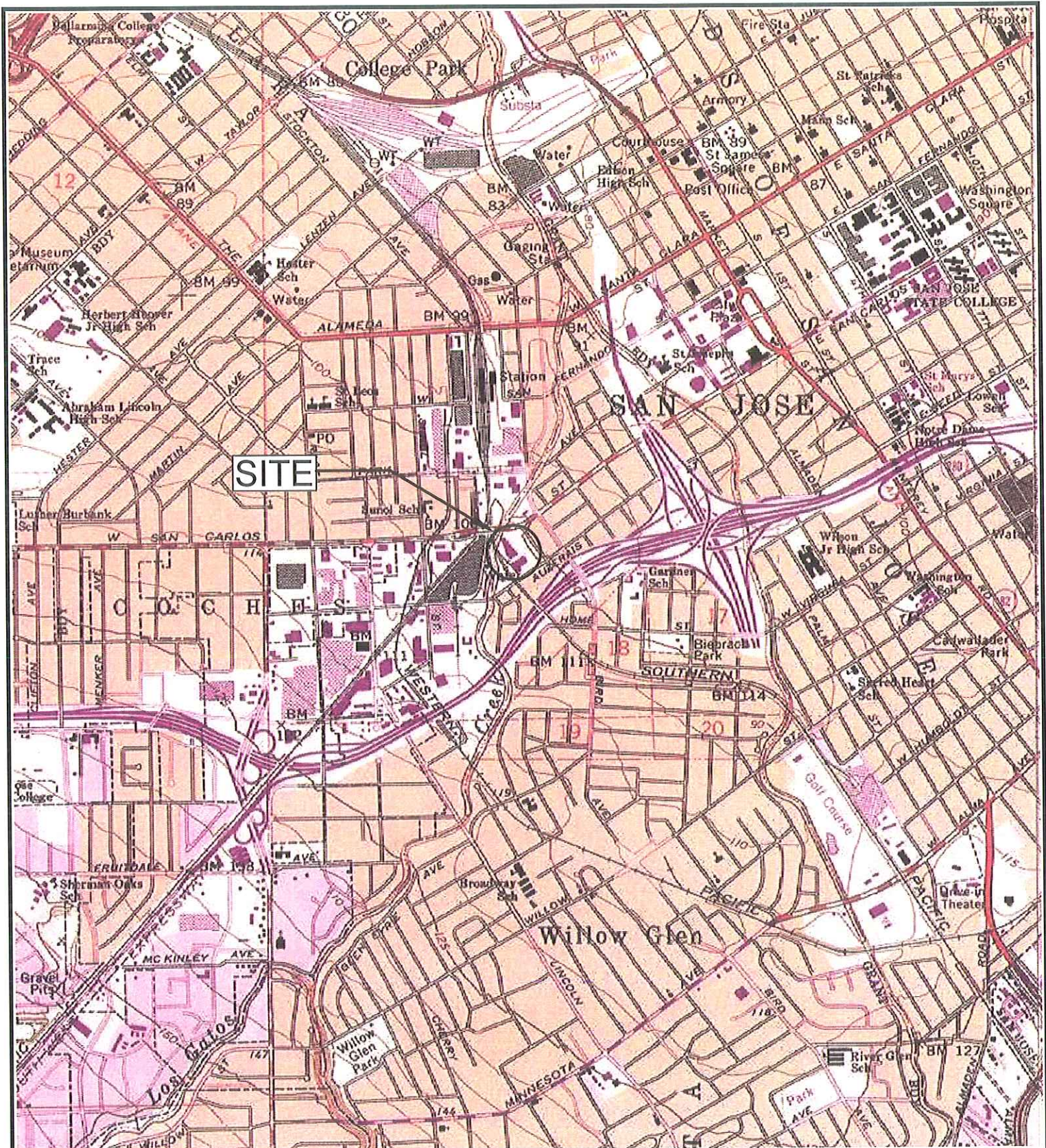
DRAFT

Read L. Andersen, RGE
Manager

APPENDIX A

DRAWINGS

Drawing No. 1 -	Site Location Map
Drawing No. 2 -	Test Boring Location Map



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 1/2 MINUTE SERIES
 SAN JOSE WEST, CALIFORNIA QUADRANGLE 1979, PHOTOREVISED 1980

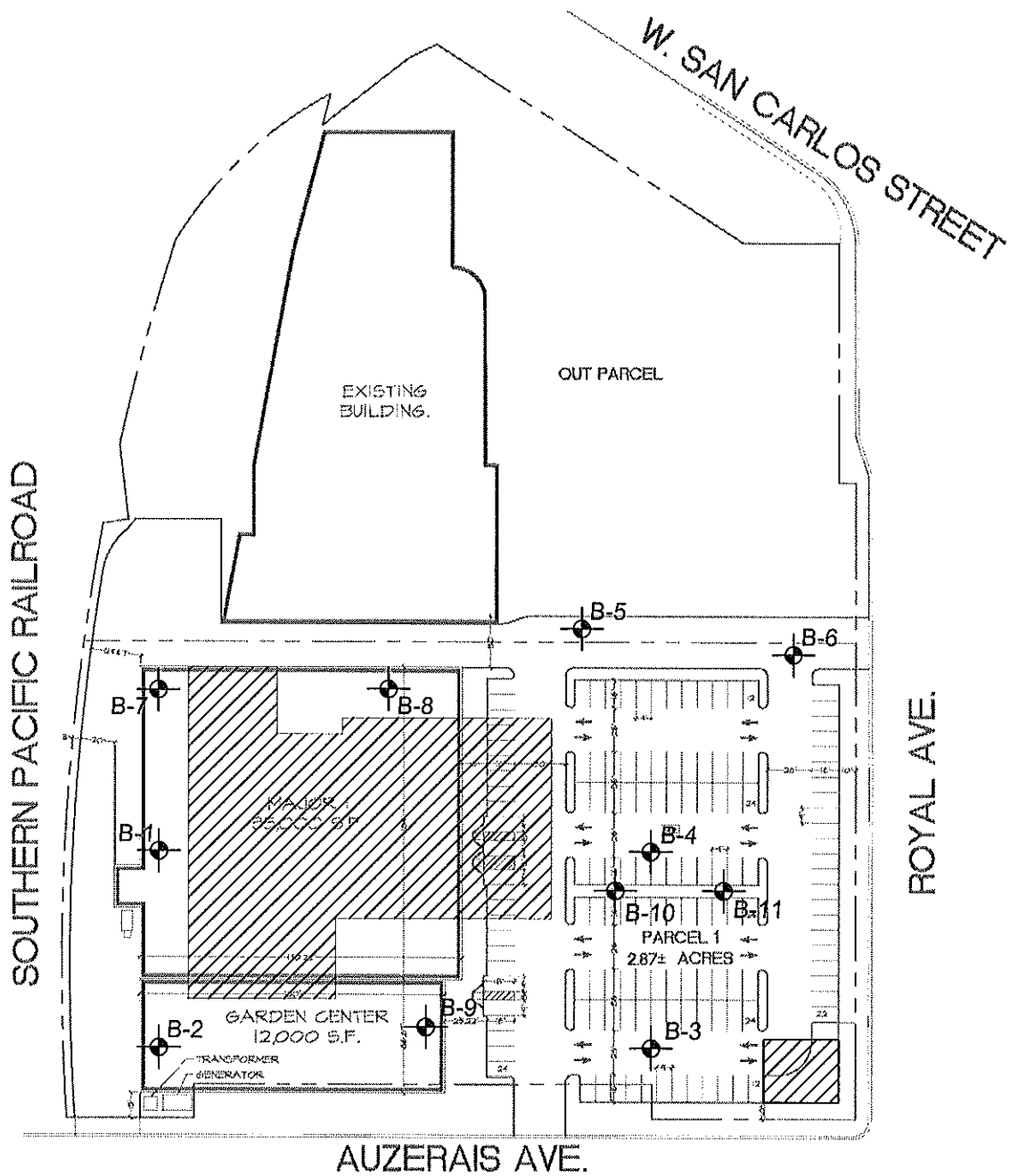
0 2000
 APPROXIMATE SCALE
 IN FEET

SITE LOCATION MAP
 ORCHARD SUPPLY HARDWARE
 W SAN CARLOS STREET AND ROYAL AVENUE
 SAN JOSE, CALIFORNIA

FILE NO: 82601-01-01	DATE: 02/13/13
DRAWN BY: RM	APPROVED BY:
PROJECT NO. E82601.01	DRAWING NO. 1



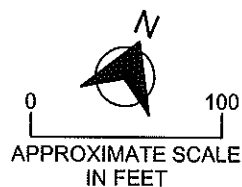
**MOORE TWINING
 ASSOCIATES, INC.**



APPROXIMATE TEST BORING LOCATION



APPROXIMATE LIMITS OF EXISTING STRUCTURES



TEST BORING LOCATION MAP
ORCHARD SUPPLY HARDWARE
W SAN CARLOS STREET AND ROYAL AVENUE
SAN JOSE, CALIFORNIA

FILE NO.
82601-01-02A

DRAWN BY:
RM

PROJECT NO.
E82601.01

DATE DRAWN:
02/13/13

APPROVED BY:

DRAWING NO.
2



MOORE TWINING
ASSOCIATES, INC.

APPENDIX B

LOGS OF BORINGS

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

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

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



Test Boring: B-1

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: 29.5 Feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	2/6 10/6 7/6	AC FILL	Asphaltic Concrete= 3.5 Inches FILL: LEAN CLAY, Sandy; Very stiff, moist, low to medium plasticity, brown		17	
5	2/6 4/6 5/6	CH CL	Asphalt debris and fine to coarse gravel noted FAT CLAY, Sandy; Stiff, moist, high plasticity, trace gravel, dark brown to black LEAN CLAY, Sandy; Stiff, moist, low to medium plasticity, brown, iron oxide staining Medium stiff	DD=103.4 PCF Gravel=1% SAND=32% -200=67% PI=34 LL=55 ø=20° C=200 PSF	PUSH 9	19
10	2/6 3/6 4/6		Stiff, trace fine gravel	DD=108.8 PCF	PUSH 9	30
15	2/6 4/6 5/6		Medium stiff, very moist	SAND=36% -200=64% PI=27 LL=47	6	23
20	2/6 4/6 4/6				8	
25	2/6 4/6 4/6					

Notes: Groundwater encountered at 29.5 Feet BSG, after drilling water measured at 45 Feet BSG, 30 minutes after drilling water measured at 44 Feet BSG

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-1

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

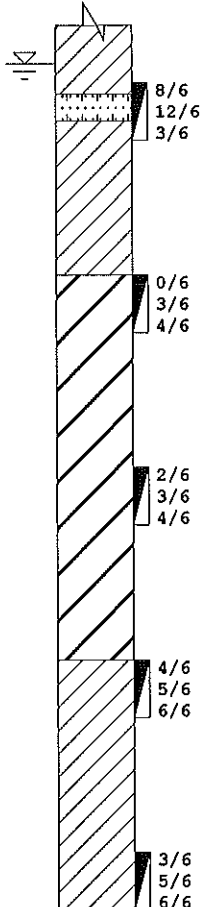
Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: 29.5 Feet BSG

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
30		SW-SM CL	Wet SAND, Well Graded with Silt and Gravel; Medium dense, moist to wet, fine to coarse, gray brown LEAN CLAY, Sandy; Stiff, very moist to wet, low to medium plasticity, brown	GRAVEL=35% SAND=55% -200=10% SAND=45% -200=55% PI=22 LL=42	15	11
35		CH	FAT CLAY; Medium stiff, very moist to wet, high plasticity, olive		7	26
40			with sand		7	20
45		CL	LEAN CLAY, SANDY; Stiff, moist to wet, low to medium plasticity, olive gray	SAND=50% -200=50% PI=23 LL=43	11	21
50					11	24
55			Bottom of Boring at 51.5 Feet BSG			

Notes: Groundwater encountered at 29.5 Feet BSG, after drilling water measured at 45 Feet BSG, 30 minutes after drilling water measured at 44 Feet BSG

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-2

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	10/6	AC	Asphaltic Concrete=3.75 inches	EI=76		
	6/6	FILL	FILL: FAT CLAY, Sandy; Stiff,	DD=112.6 PCF	13	15
	7/6		moist, medium plasticity, trace fine	GRAVEL=1%	4	
	2/6		to coarse gravel, brown	SAND=34%		
	1/6			-200=65%		
	3/6	CH	FAT CLAY; Soft, moist, high	PI=31		37
			plasticity, dark brown to black	LL=54		
5	2/6	CL	LEAN CLAY, Sandy; Medium stiff,		8	
	3/6		moist, low to medium plasticity,			
	5/6		iron oxide staining, gray brown			
10	2/6				7	21
	2/6					
	5/6					
15	2/6				5	
	2/6					
	3/6					
20	3/6				8	
	3/6					
	5/6					
25			Bottom of Boring at 21.5 Feet BSG			

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-3

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	6/6 9/6 5/6	AC FILL	Asphaltic Concrete=3 inches		14	23
5	3/6 6/6 8/6		FILL: LEAN CLAY, Sandy; Stiff, moist, low to medium plasticity, trace fine to coarse gravel, with asphalt debris, dark brown		14	
10	5/6 7/6 9/6		FILL: FAT CLAY; Stiff, moist, high plasticity, trace gravel, dark brown glass debris			
15	3/6 5/6 7/6	CH	Very stiff, plastic debris and brick debris	DD=98.2 PCF	PUSH	25
20	3/6 7/6 8/6		Stiff, metal and plastic debris		12	21
25			FAT CLAY; Stiff, moist, high plasticity, dark brown to black		15	
			Bottom of Boring at 21.5 Feet BSG			

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-4

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0	4/6	AC FILL	Asphaltic Concrete=3 inches	El=88 DD=94.4 PCF	16	28
	7/6		FILL: LEAN CLAY, Sandy; Stiff,		7	
	9/6		moist, low to medium plasticity,			
	3/6		trace fine gravel, dark brown			
	3/6		FILL: FAT CLAY; Medium stiff,			
	4/6		moist, high plasticity, dark brown			
-5	--	CL	to black	PUSH		16
	--		FILL: LEAN CLAY, with Sand;			
	2/6		Medium stiff, moist, low to medium		5	
	2/6		plasticity, wood debris, brown			
	3/6		trace fine gravel			
-10	2/6	CL			5	16
	3/6					
	2/6					
-15	4/6	CL	LEAN CLAY, with Sand; Stiff,		15	
	6/6		moist, low to medium plasticity,			
	9/6		trace coarse gravel, dark brown			
-20	4/6	CL	brown, iron oxide staining		9	
	4/6					
	5/6					
-25			Bottom of Boring at 21.5 Feet BSG			

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-5

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphaltic Concrete=3 inches			
		FILL	Aggregate Base= 3 inches		8	19
		CL	FILL: LEAN CLAY, Sandy; Stiff, moist, low to medium plasticity, trace fine gravel, dark brown		8	32
5		CH	LEAN CLAY, Sandy; Medium stiff, moist, low to medium plasticity, brown to black			
			FAT CLAY; Medium stiff, moist, high plasticity, dark brown to brown			
10		CL	LEAN CLAY, with Sand; Stiff, moist, low to medium plasticity, brown		13	
			Bottom of Boring at 10 Feet BSG			
15						
20						
25						

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-6

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: AR

Drill Type: CME 75

Date: 11-8-12

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0						
		AC	Asphaltic Concrete=3 inches			
		FILL	Aggregate Base= 3 inches		11	37
		CH	FILL: LEAN CLAY, Sandy; Stiff, moist, low to medium plasticity, trace fine gravel, brick debris, dark brown		7	
5			FAT CLAY; Stiff, moist, high plasticity, dark brown to black Medium stiff, iron oxide staining			
		CL	LEAN CLAY, with Sand; Very stiff, moist, low to medium plasticity, trace gravel, brown		29	11
10			Bottom of Boring at 10 Feet BSG			
15						
20						
25						

Notes:

Figure Number



Test Boring: B-7

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: ZA

Drill Type: CME 75

Date: 1-30-13

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0						
	2/6 2/6 2/6	AC FILL	Asphaltic Concrete=5 inches Aggregate Base= 3 inches FILL: LEAN CLAY; Medium stiff, moist, medium plasticity, brick debris, dark brown, blended Native at 3.5 feet?	DD= 103.6 PCF	4	22
5	3/6 4/6 6/6	CL	LEAN CLAY, Sandy; Stiff, moist, low to medium plasticity, some gravel, light brown to brown		10	
10	1/6 2/6 3/6		brown to red brown, decrease in percent sand		5	
15	3/6 3/6 3/6		brown, some gravel, increase in percent sand		6	
20	9/6 12/6 18/6	SC	SAND, Clayey; Dense, moist, fine to medium, brown, with gravel		30	
			Bottom of Boring at 20 Feet BSG			
25						

Notes:

Figure Number



Test Boring: B-8

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: ZA

Drill Type: CME 75

Date: 1-30-13

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphaltic Concrete= 10 inches			
		FILL	FILL: LEAN CLAY, with sand; Medium stiff, moist, medium plasticity, brown to dark brown brown, trace gravel, FAT CLAY, Sandy; Medium stiff, moist, medium plasticity, dark brown, Native at 4 feet? trace gravel	DD=79.8 PCF	4 PUSH	28 31
5		CL	LEAN CLAY, Sandy; Medium stiff, moist, medium plasticity, brown to light brown, trace gravel		7 8	
10		CL	LEAN CLAY; with Sand; Medium stiff, moist, low to medium plasticity, iron oxide staining, light brown		PUSH 5	
15					6	
20			very moist		7	
25			Bottom of Boring at 21.5 Feet BSG			

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-9

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: ZA

Drill Type: CME 75

Date: 1-30-13

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC	Asphaltic Concrete= 11 inches		8	
		FILL	FILL: LEAN CLAY; Medium stiff, moist, medium plasticity, brown Stiff, brown to dark brown, trace gravel blended, trace gravel	DD= 106.7 PCF	PUSH	17
5					14	
					14	
			blended		9	
10			red brick fragments		9	
			wood debris		9	
15			increase in sand, decrease in gravel Very stiff, cobble		14	
					21	
					10	
20		CL	LEAN CLAY, Sandy, Stiff, moist, low to medium plasticity, brown to light brown some gravel, olive brown		12	
					10	
25			Bottom of Boring at 25 Feet BSG			

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-10

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: ZA

Drill Type: CME 75

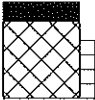
Date: 1-30-13

Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater
First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC FILL	Asphaltic Concrete= 6 inches FILL: FAT CLAY; Moist, medium plasticity, dark brown Bottom of Boring at 2.5 Feet BSG		PUSH	
5						
10						
15						
20						
25						

Notes:

Figure Number



MOORE TWINING ASSOCIATES, INC.

Test Boring: B-11

Project: Proposed Orchard Supply Hardware, 720 West San Carlos, San Jose, Ca

Project Number: E82601.01

Drilled By: JS

Logged By: ZA

Drill Type: CME 75

Date: 1-30-13

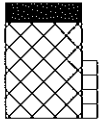
Auger Type: 6 5/8" H.S.A.

Elevation:

Hammer Type: 140 lb TRIP

Depth to Groundwater

First Encountered During Drilling: NE

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-Values blows/ft.	Moisture Content %
0		AC FILL	Asphaltic Concrete= 7 inches FILL: FAT CLAY; Moist, medium plasticity, dark brown		PUSH	
5			Bottom of Boring at 3.0 Feet BSG			
10						
15						
20						
25						

Notes:

Figure Number


KEY TO SYMBOLS


Symbol Description

Symbol Description


Strata symbols


Misc. Symbols

 ASPHALTIC CONCRETE


 Boring continues


 Fill


 Water table during drilling


 FAT CLAY


Soil Samplers

 LEAN CLAY

 Standard penetration test

 Well graded sand with silt

 California Modified split barrel ring sampler

 Clayey sand

Notes:

1. Test Borings were drilled on November 8, 2012 and January 30, 2013 using a CME 75 equipped with 6 5/8" Hollow Stem Augers .
2. Groundwater was encountered during drilling. See Logs.
3. Test boring locations were located by pace with reference to the existing site features.
4. These logs are subject to the limitations, conclusions, and recommendations in this report.
5. The "N-value" reported for the California Modified Split Barrel Sampler is the uncorrected field blow count. This value should not be interpreted as an SPT equivalent N-value.
6. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

DD =	Natural dry density (pcf)
LL =	Liquid limit (%)
PI =	Plasticity index (%)
-200 =	Percent passing #200 sieve (%)
c =	Cohesion (psf)
ø =	Angle of internal friction (degrees)
N/A =	Not applicable
N/E =	None encountered
pcf =	pounds per cubic foot
psf =	pounds per square foot
BSG =	Below Site Grade

APPENDIX C

RESULTS OF LABORATORY TESTS

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

These Included:

Moisture Content
(ASTM D2216)

Dry Density
(ASTM D2216)

Moisture-Density
Relationship
(ASTM D1557)

Expansion Index
(ASTM D4829)

Atterberg Limits
(ASTM D4318)

R-Value
(CTM 301)

Sulfate Content
(ASTM D4327)

Chloride Content
(ASTM D4327)

Resistivity
(ASTM D1125)

pH (ASTM D4972)

To Determine:

Moisture contents representative of field conditions at the time the sample was taken.

Dry unit weight of sample representative of in-situ or in-place undisturbed condition.

The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.

Swell potential of soil with increases in moisture content.

Determines the moisture content where the soil behaves as a viscous material (liquid limit) and the moisture content at which the soil reaches a plastic state

The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.

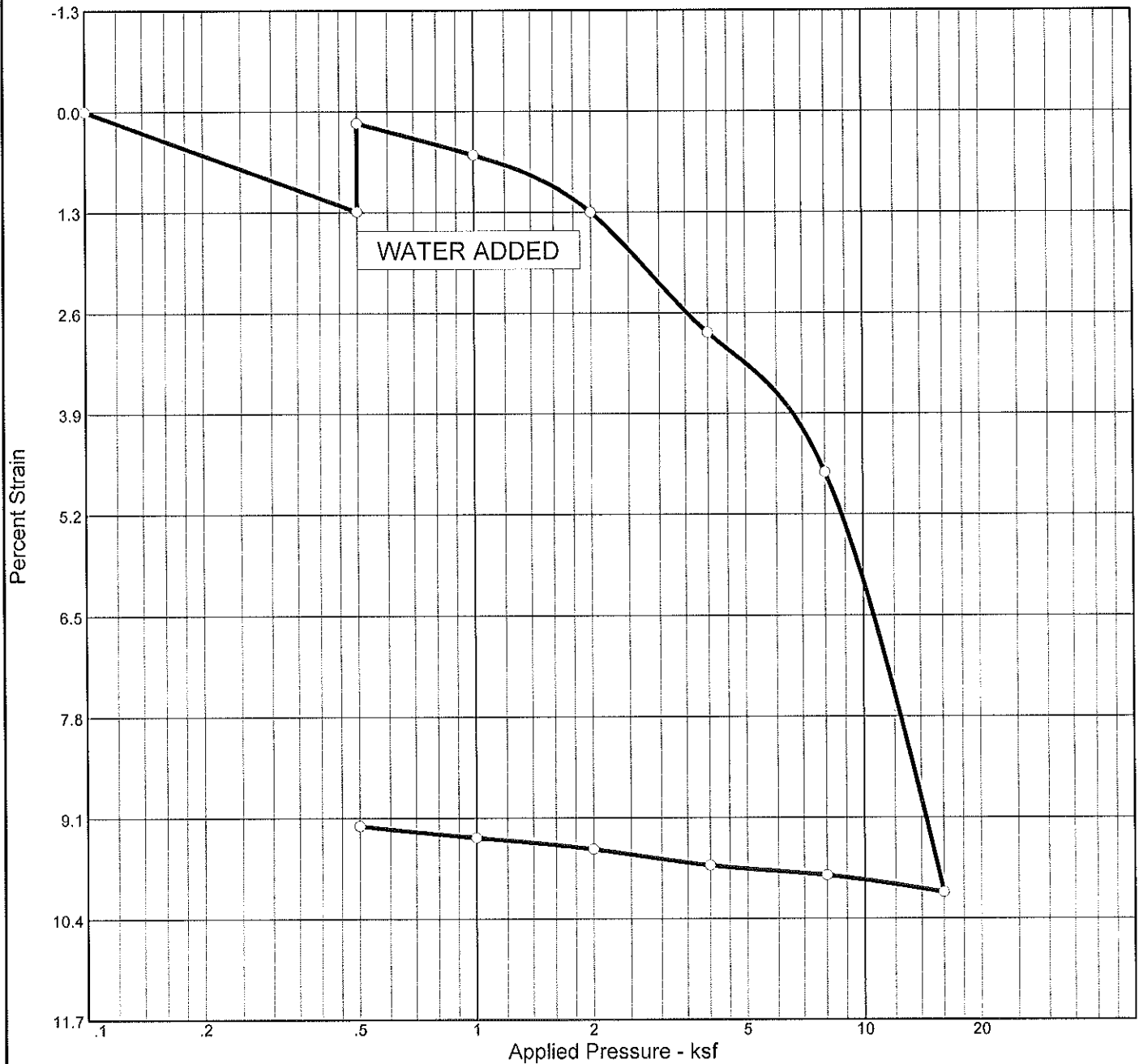
Percentage of water-soluble sulfate as (SO₄) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.

Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.

The potential of the soil to corrode metal.

The acidity or alkalinity of subgrade material.

CONSOLIDATION TEST REPORT



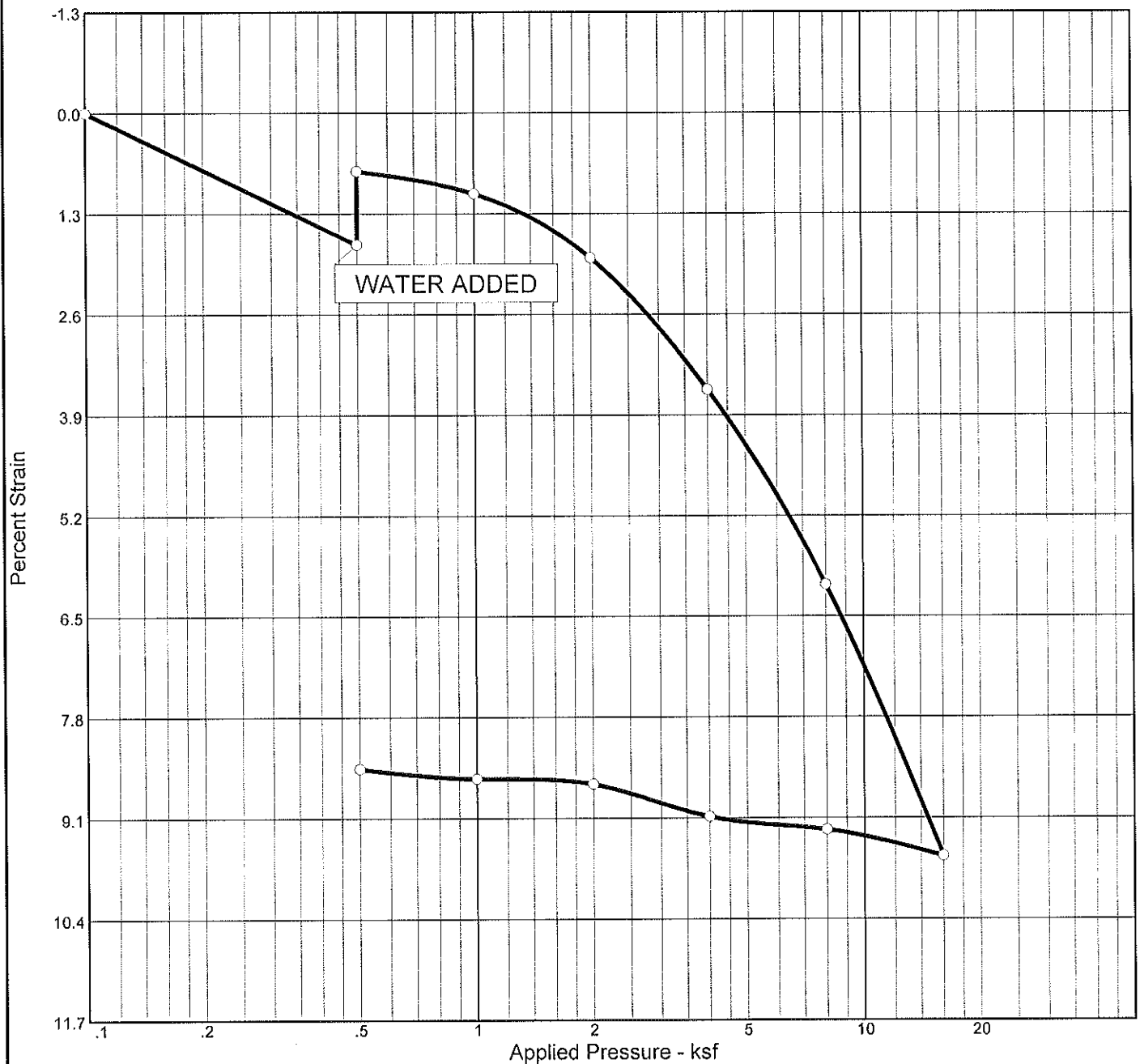
Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
92.3 %	21.8 %	101.7	55	34	2.65		8.04	0.30	0.01	1.99	1.2	0.627

MATERIAL DESCRIPTION										USCS	AASHTO
Sandy fat clay										CH	

Project No. E82601.01			Client:			Remarks:
Project: Proposed Orchard Supply Hardware						
Source:		Sample No.: B-1		Elev./Depth: 3.5-5'		
Moore Twining Associates, Inc. Fresno, CA						
						Figure

Figure

CONSOLIDATION TEST REPORT

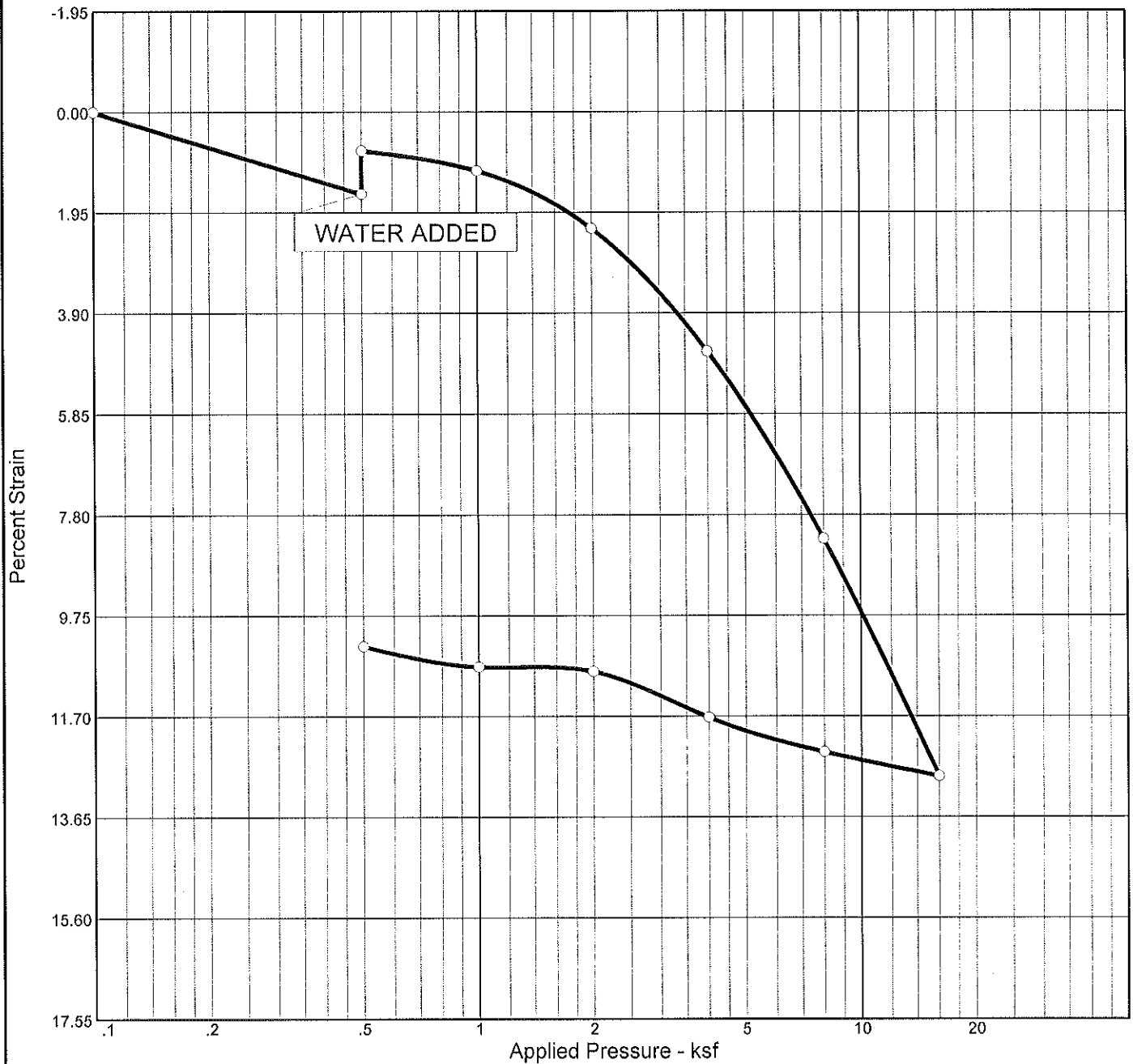


Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
94.6 %	29.4 %	90.7			2.65		4.07	0.22	0.01	1.81	0.9	0.824

MATERIAL DESCRIPTION										USCS	AASHTO
Undocumented Fill; Lean Clay											

Project No. E82601.01			Client:			Remarks:
Project: Proposed Orchard Supply Hardware						
Source:		Sample No.: B-4		Elev./Depth: 0.5-2'		
Moore Twining Associates, Inc. Fresno, CA						
						Figure

CONSOLIDATION TEST REPORT

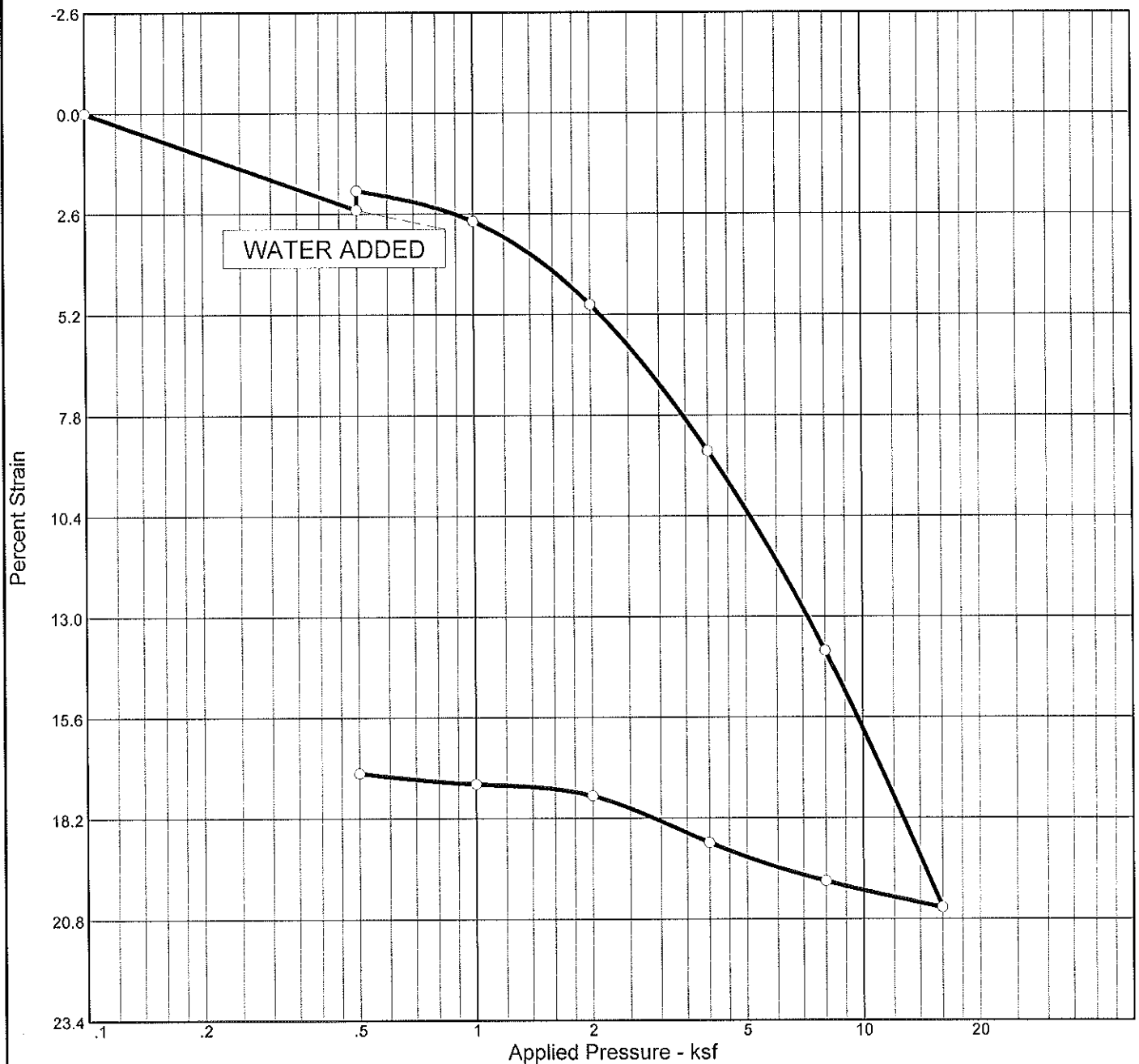


Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
88.1 %	22.1 %	99.4	66	42	2.65		3.72	0.26	0.03	1.44	0.8	0.665

MATERIAL DESCRIPTION										USCS	AASHTO
Undocumented Fill; Sandy fat clay										CH	

Project No. E82601.01			Client:			Remarks:
Project: Proposed Orchard Supply Hardware						
Source:		Sample No.: B-7		Elev./Depth: 3.5-5'		
Moore Twining Associates, Inc.						
Fresno, CA						Figure

CONSOLIDATION TEST REPORT

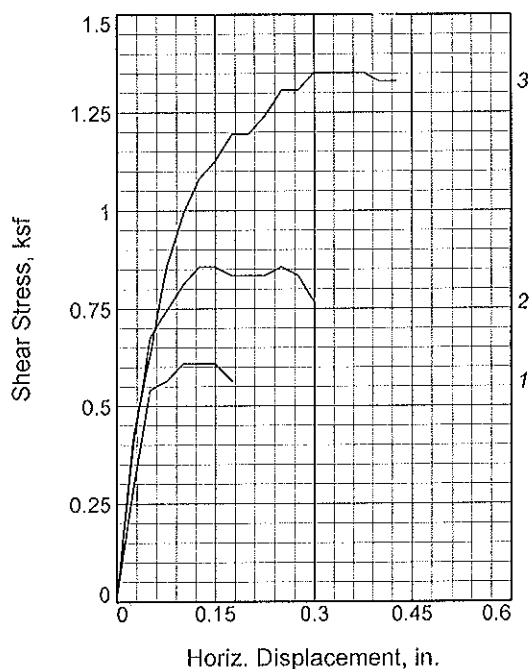
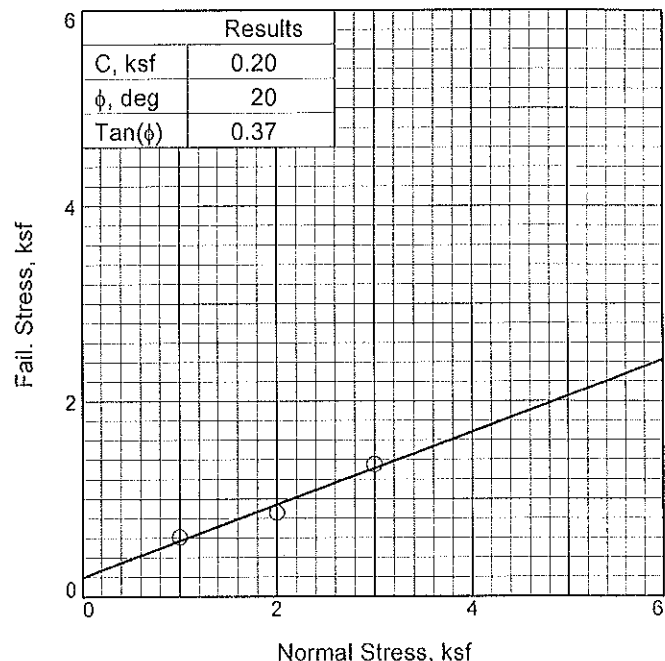
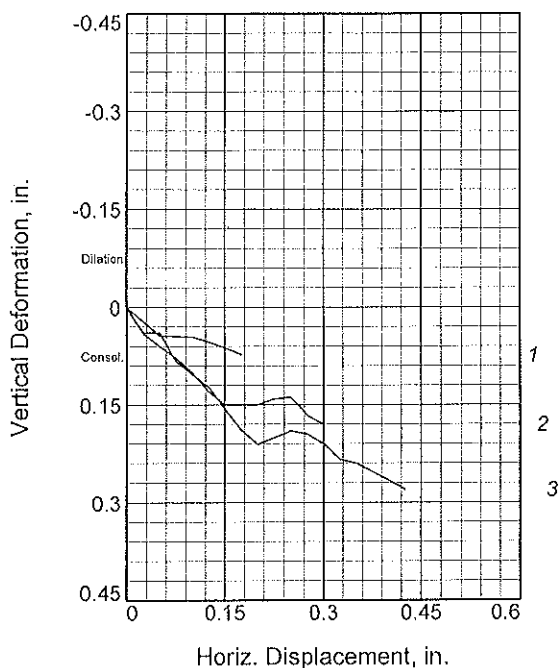


Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
88.5 %	52.8 %	64.1			2.65		2.86	0.57	0.06	0.84	0.5	1.579

MATERIAL DESCRIPTION										USCS	AASHTO
Undocumented Fill; Fat Clay											

Project No. E82601.01			Client:			Remarks:
Project: Proposed Orchard Supply Hardware						
Source:		Sample No.: B-8		Elev./Depth: 2.5-4'		
Moore Twining Associates, Inc. Fresno, CA						
						Figure

Figure



Sample No.		1	2	3
Initial	Water Content, %	21.5	18.7	20.4
	Dry Density, pcf	95.4	101.6	98.8
	Saturation, %	77.7	78.8	80.1
	Void Ratio	0.7339	0.6279	0.6743
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.4	21.5	23.6
	Dry Density, pcf	96.4	103.2	100.9
	Saturation, %	90.2	94.5	98.0
	Void Ratio	0.7164	0.6036	0.6393
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.99	0.98
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.61	0.86	1.35
Displacement, in.		0.10	0.13	0.30
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		4.1	5.2	12.4

Sample Type:

Description: Sandy fat clay

LL= 55

PL= 21

PI= 34

Assumed Specific Gravity= 2.65

Remarks:

Client:

Project: Proposed Orchard Supply Hardware

Sample Number: B-1

Depth: 3.5-5'

Proj. No.: E82601.01

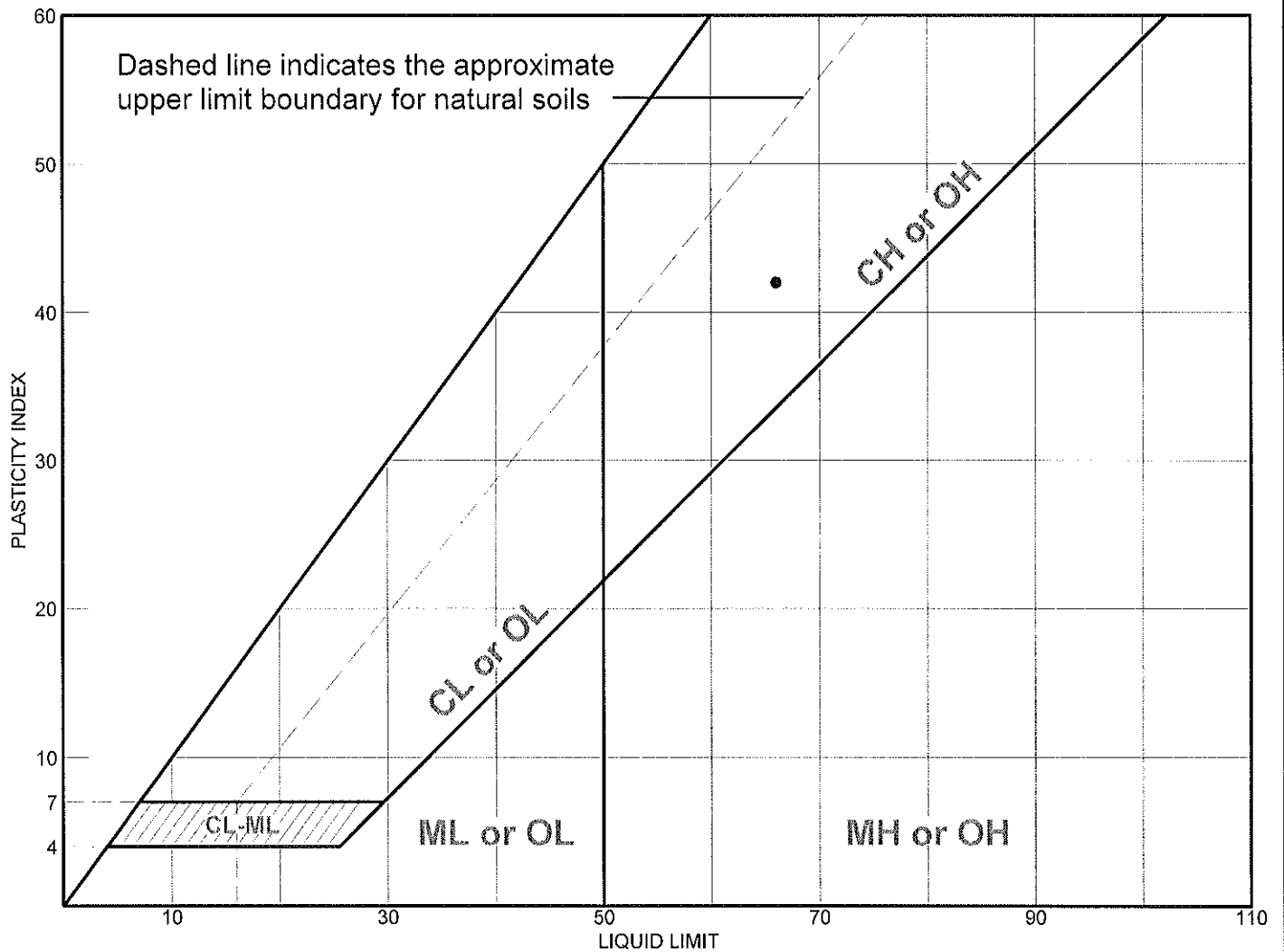
Date Sampled: 11/08/12

DIRECT SHEAR TEST REPORT

Moore Twining Associates, Inc.

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
•	Undocumented Fill; Sandy fat clay	66	24	42	81.7	66.0	CH

Project No. E82601.01 Client:

Project: Proposed Orchard Supply Hardware

• Source: Sample No.: B-7 Elev./Depth: 3.5-5'

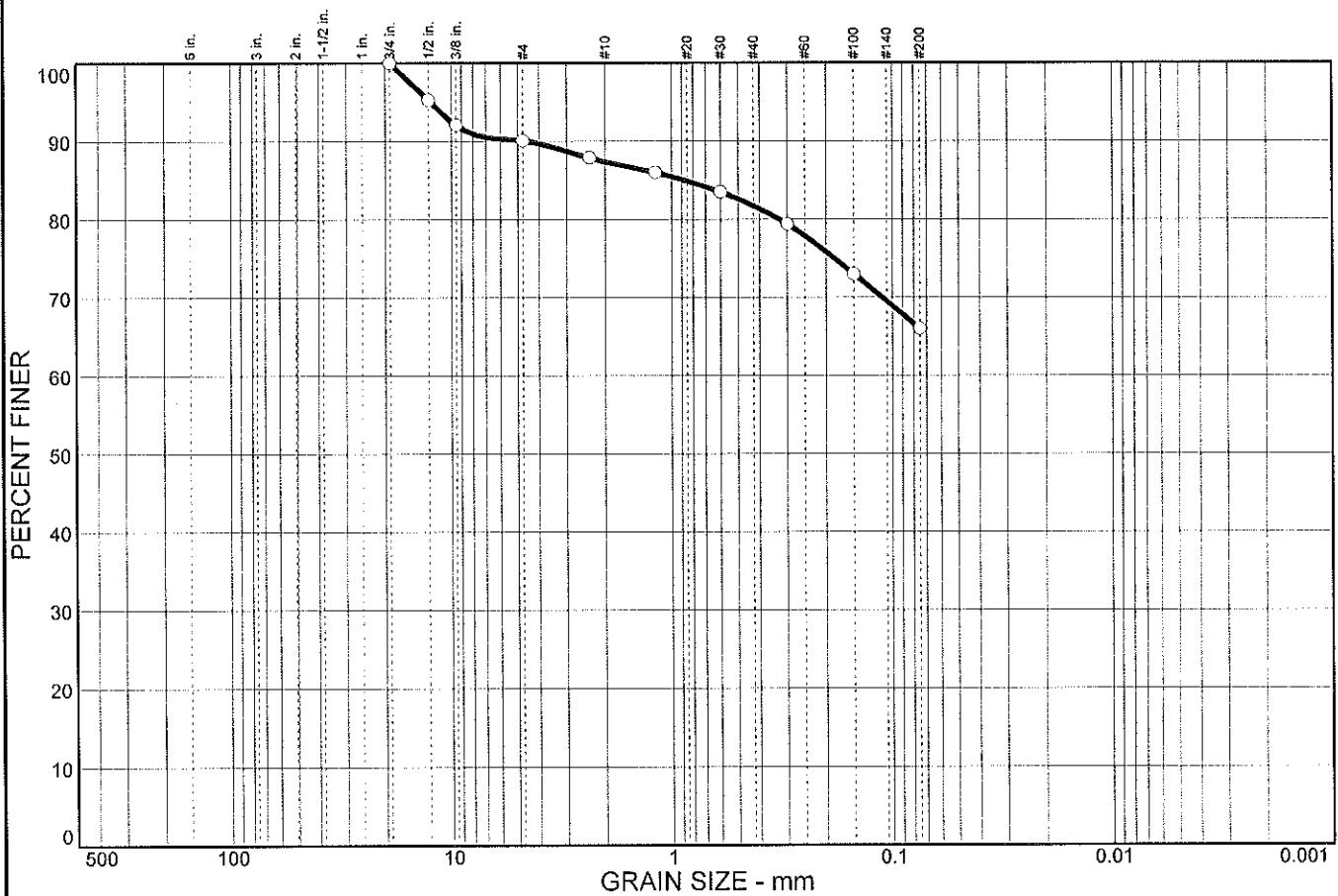
Moore Twining Associates, Inc.
Fresno, CA

Remarks:

•

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	9.9	2.7	5.7	15.7	66.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100.0		
1/2 in.	95.3		
3/8 in.	92.1		
#4	90.1		
#8	87.9		
#16	86.0		
#30	83.5		
#50	79.4		
#100	73.0		
#200	66.0		

* (no specification provided)

Material Description		
Undocumented Fill; Sandy fat clay		
Atterberg Limits		
PL= 24	LL= 66	PI= 42
Coefficients		
D ₈₅ = 0.866	D ₆₀ =	D ₅₀ =
D ₃₀ =	D ₁₅ =	D ₁₀ =
C _u =	C _c =	
Classification		
USCS= CH	AASHTO=	
Remarks		
F.M.=1.08		

Sample No.: B-7
Location:

Source of Sample:

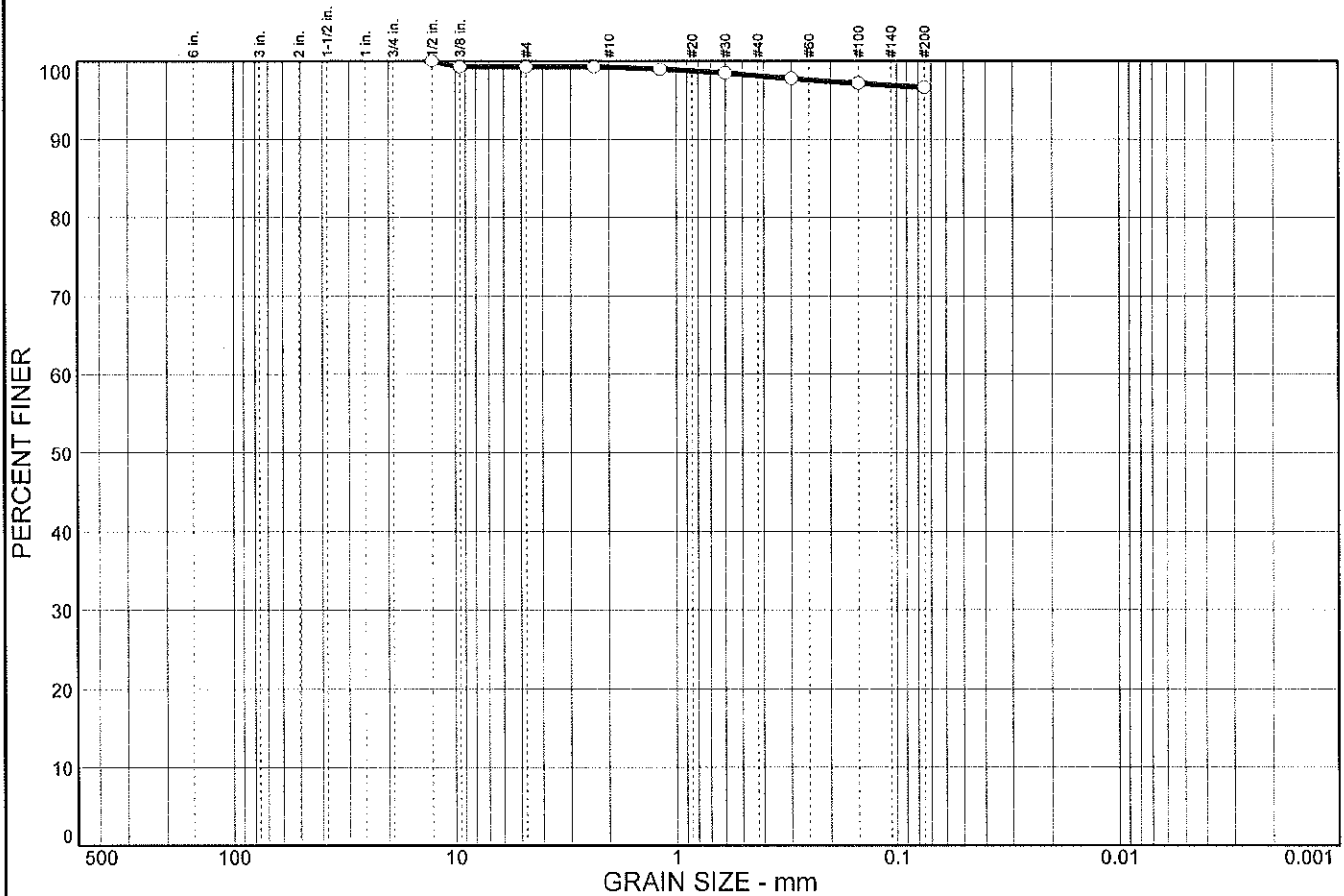
Date: 01/30/13
Elev./Depth: 3.5-5'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Proposed Orchard Supply Hardware
Project No: E82601.01

Figure

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.8	0.1	1.1	1.4	96.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100.0		
3/8 in.	99.2		
#4	99.2		
#8	99.2		
#16	98.9		
#30	98.4		
#50	97.7		
#100	97.1		
#200	96.6		

* (no specification provided)

Material Description		
Undocumented Fill; Fat Clay		
<div> <div>Atterberg Limits</div> <div> <div>PL=</div> <div>LL=</div> <div>PI=</div> </div> </div>		
<div> <div>Coefficients</div> <div> <div>D₈₅=</div> <div>D₆₀=</div> <div>D₃₀=</div> <div>D₁₅=</div> <div>C_u=</div> <div>C_c=</div> </div> <div> <div>D₅₀=</div> <div>D₁₀=</div> </div> </div>		
<div> <div>Classification</div> <div> <div>USCS=</div> <div>AASHTO=</div> </div> </div>		
<div> <div>Remarks</div> <div>F.M.=0.10</div> </div>		

Sample No.: B-8
Location:

Source of Sample:

Date: 01/30/13
Elev./Depth: 2.5-4'

Moore Twining Associates, Inc.
Fresno, CA

Client:
Project: Proposed Orchard Supply Hardware
Project No: E82601.01

Figure



MOORE TWINING

ASSOCIATES, INC.

EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME: Proposed Orchard Supply REPORT DATE: 11/19/2012
Hardware TEST DATE: 11/16/2012
MTA PROJECT NO.: E82601.01
SAMPLE I.D.: B-2 @ 1-3.5'
SAMPLED BY: AR
SAMPLE DATE: 11/8/2012 TESTED BY: TD

MATERIALS DESCRIPTION: Lean Clay

% PASSING # 4 SIEVE 100

Initial Moisture Determination:

Pan + Wet Soil Wt., gm 250.0
Pan + Dry Soil Wt., gm 219.0
Pan Wt., gm 0.0
Initial % Moisture Content 14.2

Final Moisture Determination:

Wet Soil Wt., lbs 0.8616
Dry Soil Wt., lbs 0.6984
Final % Moisture Content 23.4

Initial Expansion Data:

Ring + Sample Wt., lbs 0.7973
Ring Wt., lbs 0.0000
Remolded Wt., lbs 0.7973
Remolded Wet Density, pcf 109.6
Remolded Dry Density, pcf 96.0

Final Expansion Data:

Ring + Sample Wt., lbs 0.8616
Ring Wt., lbs 0.0000
Remolded Wt., lbs 0.8616
Remolded Wet Density, pcf 110.1
Remolded Dry Density, pcf 89.3

Expansion Data:

Initial Gage Reading, in: 0.0500
Final Gage Reading, in: 0.1258
Expansion, in: 0.0758
Expansion Index 76

Initial Volume 0.00727222 Final Volume 0.007823

Comments: (Medium Expansion Potential)

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

Expansion Index

0-20
21-50
51-90
91-130
>130

Potential Expansion

Very Low
Low
Medium
High
Very High



EXPANSION INDEX TEST, ASTM D4829

MTA PROJECT NAME: Proposed Orchard Supply REPORT DATE: 11/19/2012
Hardware TEST DATE: 11/16/2012
 MTA PROJECT NO.: E82601.01
 SAMPLE I.D.: B-4 @ 1-5'
 SAMPLED BY: AR
 SAMPLE DATE: 11/8/2012 TESTED BY: TD

MATERIALS DESCRIPTION: Lean Clay

% PASSING # 4 SIEVE 100

Initial Moisture Determination:

Pan + Wet Soil Wt., gm 250.0
 Pan + Dry Soil Wt., gm 221.2
 Pan Wt., gm 0.0
 Initial % Moisture Content 13.0

Final Moisture Determination:

Wet Soil Wt., lbs 0.9011
 Dry Soil Wt., lbs 0.7177
 Final % Moisture Content 25.5

Initial Expansion Data:

Ring + Sample Wt., lbs 0.8112
 Ring Wt., lbs 0.0000
 Remolded Wt., lbs 0.8112
 Remolded Wet Density, pcf 111.5
 Remolded Dry Density, pcf 98.7

Final Expansion Data:

Ring + Sample Wt., lbs 0.9011
 Ring Wt., lbs 0.0000
 Remolded Wt., lbs 0.9011
 Remolded Wet Density, pcf 113.9
 Remolded Dry Density, pcf 90.7

Expansion Data:

Initial Gage Reading, in: 0.0500
 Final Gage Reading, in: 0.1380
 Expansion, in: 0.088
 Expansion Index 88

Initial Volume 0.00727222 Final Volume 0.007912

Comments: (Medium Expansion Potential)

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

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



Project Name: Proposed Orchard Supply Hardware

Report Date: 2/13/2013

Sample Date: 1/30/2013

Project Number: E82601.01

Sample Lab I.D.: 13-0167

CONSTANT VOLUME FLEXIBLE-WALL PERMEABILITY TEST (ASTM D-5084)
FALLING HEAD RISING TAIL

Sample Location: B-11 @ 1.5-3'

Sample Description: Fat Clay

Sampled By: ZA

Tested By: M. Shwiyhat

Setup Date: 2/7/2013

Completion Date: 2/13/2013

Stage	Initial	Final
Wet Weight, gm	351.4	361.6
Dry Weight, gm	265.7	265.7
Moisture content, %	32.3	36.1
Average Length, in	2.473	2.651
Average Diameter, in	2.407	2.413
Vt, ft ³	6.51E-03	7.02E-03
Vw, ft ³	3.02E-03	3.38E-03
Vs, ft ³	3.47E-03	3.47E-03
Va, ft ³	1.46E-05	1.61E-04
Vv, ft ³	3.04E-03	3.55E-03
S, Degree of Saturation, %	99.5	95.5
Dry, Density, pcf	89.9	83.4
e, Void Ratio	0.88	1.02
n, Porosity	46.7	50.5
"B" Value	0.95	
Back Pressure, psi	50	
Effective Pressure, psi	2	
Average Permeability, cm/sec	1.67E-08	

Notes:

- 1- De-aired tap water was used as the permeant liquid
- 2- Porosity was based on an assumed specific gravity of 2.70

Moore Twining Associates, Inc.

Michael Shwiyhat, EIT
Manager
Materials Testing Division



MOORE TWINING

ASSOCIATES, INC.

Project Name: Proposed Orchard Supply Hardware

Report Date: 2/13/2013

Sample Date: 1/30/2013

Project Number: E82601.01

Sample Lab I.D.: 13-0167

CONSTANT VOLUME FLEXIBLE-WALL PERMEABILITY TEST (ASTM D-5084)
FALLING HEAD RISING TAIL

Sample Location: B-10 @ 1-2.5'

Sample Description: Fat Clay

Sampled By: ZA

Tested By: M. Shwiyhat

Setup Date: 2/7/2013

Completion Date: 2/13/2013

Stage	Initial	Final
Wet Weight, gm	288.1	288.6
Dry Weight, gm	208.8	208.8
Moisture content, %	38.0	38.2
Average Length, in	2.127	2.130
Average Diameter, in	2.407	2.413
Vt, ft ³	5.60E-03	5.64E-03
Vw, ft ³	2.80E-03	2.82E-03
Vs, ft ³	2.73E-03	2.73E-03
Va, ft ³	7.14E-05	9.45E-05
Vv, ft ³	2.87E-03	2.91E-03
S, Degree of Saturation, %	97.5	96.8
Dry, Density, pcf	82.1	81.5
e, Void Ratio	1.05	1.07
n, Porosity	51.3	51.6
"B" Value	0.96	
Back Pressure, psi	50	
Effective Pressure, psi	2	
Average Permeability, cm/sec	1.99E-08	

Notes:

- 1- De-aired tap water was used as the permeant liquid
- 2- Porosity was based on an assumed specific gravity of 2.70

Moore Twining Associates, Inc.

Michael Shwiyhat, EIT

Manager

Materials Testing Division



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

California ELAP Certificate# 1371

November 28, 2012

Work Order #: 2K14053

Dean Ledgerwood
MTA Geotechnical Division
2527 Fresno Street
Fresno, CA 93721

RE: Proposed Orchard Supply Hardware

Enclosed are the analytical results for samples received by our laboratory on 11/14/12. For your reference, these analyses have been assigned laboratory work order number 2K14053.

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

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

Sincerely,

Moore Twining Associates, Inc.

A handwritten signature in cursive script that reads "Lisa Montijo".

Lisa Montijo
Client Services Assistant



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MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Orchard Supply Hardware
Project Number: E82601.01
Project Manager: Dean Ledgerwood

Reported:
11/28/2012

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B1 3.5-5	2K14053-01	Soil	11/08/12 00:00	11/14/12 15:35
B5 @ 1-3.5	2K14053-02	Soil	11/09/12 00:00	11/14/12 15:35



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MTA Geotechnical Division
2527 Fresno Street
Fresno CA, 93721

Project: Proposed Orchard Supply Hardware
Project Number: E82601.01
Project Manager: Dean Ledgerwood

Reported:
11/28/2012

B1 3.5-5
2K14053-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
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Inorganics

Chloride	13	12	mg/kg	T2K1901	11/19/12	11/20/12	ASTM D-4327-84	
Chloride	0.0013	0.0012	% by Weight	{CALC}	11/19/12	11/20/12	ASTM D4327-84	
Sulfate as SO ₄	0.027	0.0012	% by Weight	{CALC}	11/19/12	11/20/12	ASTM D4327-84	
pH	7.3	0.60	pH Units	T2K1901	11/19/12	11/20/12	ASTM D4972-89 Mod	
Resistivity	5800		ohms-cm	T2K1901	11/19/12	11/20/12	ASTM D1125-82	
Sulfate as SO ₄	270	12	mg/kg	T2K1901	11/19/12	11/20/12	ASTM D4327-84	



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Project: Proposed Orchard Supply Hardware
Project Number: E82601.01
Project Manager: Dean Ledgerwood

Reported:
11/28/2012

B5 @ 1-3.5
2K14053-02 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	14	6.0	mg/kg	T2K1901	11/19/12	11/20/12	ASTM D-4327-84	
Chloride	0.0014	0.00060	% by Weight	{CALC}	11/19/12	11/20/12	ASTM D4327-84	
Sulfate as SO ₄	0.019	0.00060	% by Weight	{CALC}	11/19/12	11/20/12	ASTM D4327-84	
pH	7.9	0.30	pH Units	T2K1901	11/19/12	11/20/12	ASTM D4972-89 Mod	
Resistivity	2400		ohms-cm	T2K1901	11/19/12	11/20/12	ASTM D1125-82	
Sulfate as SO ₄	190	6.0	mg/kg	T2K1901	11/19/12	11/20/12	ASTM D4327-84	

Notes and Definitions

ND Analyte NOT DETECTED at or above the reporting limit
NR Not Reported
RPD Relative Percent Difference