Appendix E

Geotechnical Reports

THE TWINING LABORATORIES, INC.
EST. 1898
ANALYTICAL CHEMISTRY • ENVIRONMENTAL SERVICES
GEOTECHNICAL ENGINEERING • SAMPLING SERVICES
CONSTRUCTION INSPECTION & MATERIALS TESTING

DRAFT

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED HOME DEPOT

325 HAMPSHIRE ROAD

THOUSAND OAKS, CALIFORNIA

Project Number: D050A3.01-02

For:

Home Depot U.S.A., Inc.
3800 West Chapman Avenue
Orange, CA 92868

September 13, 2005

CORPORATE
2527 Fresno Street
Fresno, CA 93721-1804
(559) 268-7021
Fax 268-7126

MODESTO
5253 Jerusalem Court, Suite E
Modesto, CA 95356-9322
(209) 342-2061
Fax 579-1480

CORONA
840 East Parkridge Avenue, Suite 104
Corona, CA 92879
(951) 898-8932
Fax 898-8974

BAKERSFIELD
3651 Pegasus Drive, #117
Bakersfield, CA 93308-6843
(661) 393-5088
Fax 393-4643

MONTEREY
501 Ortiz Avenue
Sand City, CA 93955
(831) 392-1056
Fax 392-1059

SACRAMENTO
5675 Power Inn Road, Suite C
Sacramento, CA 95824
(916) 381-9477
Fax 381-9478

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September 13, 2005 D050A3.01-02

Home Depot U.S.A., Inc.
3800 West Chapman Avenue
Orange, CA 92868

Attention: Mr. John Teeter

Subject: **Draft: Geotechnical Engineering Investigation**
Proposed Home Depot
325 Hampshire Road
Thousand Oaks, California

Dear Mr. Teeter:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed Home Depot store to be located at 325 Hampshire Road in Thousand Oaks, 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 The Twining Laboratories, Inc. (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 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. A representative of our firm will contact you in the near future regarding these services.

CORPORATE	MODESTO	CORONA	BAKERSFIELD	MONTEREY	SACRAMENTO
2527 Fresno Street	5253 Jerusalem Court, Suite E	840 East Parkridge Avenue, Suite 104	3651 Pegasus Drive, #117	501 Ortiz Avenue	5675 Power Inn Road, Suite C
Fresno, CA 93721-1804	Modesto, CA 95356-9322	Corona, CA 92879	Bakersfield, CA 93308-6843	Sand City, CA 93955	Sacramento, CA 95824
(559) 268-7021	(209) 342-2061	(951) 898-8932	(661) 393-5088	(831) 392-1056	(916) 381-9477
Fax 268-7126	Fax 579-1480	Fax 898-8974	Fax 393-4643	Fax 392-1059	Fax 381-9478

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

Sincerely,

THE TWINING LABORATORIES, INC.

DRAFT

Harry D. Moore, RCE, RGE
President

HDM/id

EXECUTIVE SUMMARY

The Twining Laboratories, Inc. (Twining) was authorized by Mr. John Teeter to prepare the following geotechnical engineering investigation report for the proposed Home Depot U.S.A., Inc. to be located at 325 Hampshire Road, Thousand Oaks, California. The site is currently developed as a Kmart store.

As background, Twining initially prepared a draft geotechnical engineering investigation report for the proposed remodel of the existing Kmart store. Since that time, the project was modified to comprise the demolition of the existing Kmart store and the construction of a new Home Depot store. As a result, the previously prepared draft geotechnical engineering report is not considered appropriate for the new store construction.

The purpose of this report was use the data included in the previous report to provide geotechnical engineering parameters for use in design of the new Home Depot store and preparation of construction documents.

The proposed development includes demolition and removal of the existing structures, underground utilities, hardscapes and parking and drive areas within the overbuild zone. The planned development will include a new Home Depot store building and a garden center. The total building area of the Home Depot will comprise approximately 110,098 square feet in plan dimensions.

At the time of the initial field exploration conducted in June 2004, the project site was occupied by an existing Kmart store building, garden shop, associated asphaltic concrete (AC) parking and drive areas, and isolated landscape areas. For the purpose of discussion, this portion of the Home Depot site is referred to, hereinafter, as the Kmart property. Underground utilities, including electric, telephone, water, irrigation, sewer, and storm drain lines, were noted throughout the Kmart property.

An existing Burger King restaurant and a gasoline service station are located southeast of the Kmart store, at the northwest corner of the intersection of Foothill Drive and Hampshire Road.

The proposed development will include demolition and removal of the existing structures, underground utilities, hardscapes and parking and drive areas. The planned development will include a new Home Depot store building and a garden center. The total building area of the proposed Home Depot will comprise approximately 110,098 square feet in plan dimension. The approximate locations of the proposed Home Depot store and garden center are shown on the site plan, Drawing No. 2 in Appendix A.

The Home Depot store is anticipated to be a single-story building consisting of CMU or concrete tilt-up walls and a combination wood and steel frame roof with a concrete slab-on-grade floor. The development will include underground utilities, screen walls, retaining walls and paved parking and drive areas. A new screen wall is being proposed adjacent to Foothill Drive at the western property

EXECUTIVE SUMMARY (continued)

line. Based on the Grading Plan, the screen wall will be constructed of cast-in-place concrete or CMU wall construction. Screen walls will be supported on a drilled concrete pier foundation system. New retaining walls are planned to replace the existing CMU retaining walls west of the proposed Home Depot store. Based on the Grading Plan, the retaining walls are planned with heights ranging from 8 feet to about 32 feet and will be constructed of cast-in-place concrete. The higher sections of wall will be supported on a drilled concrete pier foundation and the shorter sections of wall will be supported on a shallow foundation system.

Construction of the new retaining walls adjacent to Foothill Drive will require temporary shoring of the excavations. The shoring will need to be designed by a qualified engineer and installed without damage to the existing public street and adjacent improvements. It is recommended a preconstruction survey of the conditions of the existing Foothill Drive and public improvements be completed by the contractor including photographs and a written description of existing distress.

The existing AC parking and drive area pavement sections were observed to be generally in a poor condition. The existing pavement sections do not comply with the Home Depot standard or heavy duty designs and appear to have served their projected design life. For the majority of the existing pavement areas, it is recommended that the existing AC section and underlying AB be removed and replaced with a pavement section that complies with the Home Depot criteria.

Structural loads may be supported on spread or continuous footings placed entirely on at least 2 feet of engineered fill, or engineered fill extending to at least 5 feet below preconstruction site grades, whichever provides the deeper fill. Exterior foundations should be supported at a minimum depth of 36 inches below the lowest adjacent finished grade, but not less than 42 inches below the finished slab surface. Interior footings should have a minimum depth of 30 inches below the finished slab surface. Footings should have a minimum width of 15 inches, regardless of load. Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

Based on the recommended site preparation, total and differential combined static and seismic settlements of 1 inch and ½ inch, respectively, are estimated for this project.

Interior concrete slabs-on-grade should be supported on a minimum of 6 inches of Class 2 aggregate base over at least 24 inches of imported non-expansive soil over the depth of engineered fill recommended below foundations. The minimum 6 inches of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction purposes. Aggregate base and all non-expansive fill should be compacted to a minimum relative compaction of 95 percent.

Based on the resistivity values, the soils exhibit a "moderately corrosive" corrosion potential. In addition, the results of soil sample analyses indicated the soils exhibit negligible potential for

EXECUTIVE SUMMARY (continued)

sulfate exposure to concrete. If pipes 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.

The nearest known active or potentially active fault is the Malibu Coast fault, with the surface trace of the fault located about 10.2 kilometers from the site. The potential for fault rupture at the site is therefore considered low.

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED HOME DEPOT

325 HAMPSHIRE ROAD

THOUSAND OAKS, CALIFORNIA

Project Number: D050A3.01-02

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed Home Depot store to be located at 325 Hampshire Road in Thousand Oaks, California. The Twining Laboratories, Inc. (Twining) was authorized by Mr. Scott Mommer (Scott A. Mommer Consulting) to perform the initial geotechnical engineering investigation and by Mr. John Teeter to prepare this revised report.

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

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

2.0 PURPOSE AND SCOPE OF INVESTIGATION

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

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations, exterior slabs-on-grade, and pavements with regard to Home Depot design criteria;

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

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

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

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

2.1.6 Evaluation of pavement overlay alternatives;

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 Home Depot store referenced in the Anticipated Construction section of this report.

This investigation did not include a geologic investigation, floodplain investigation, compaction tests, environmental investigation, or environmental audit.

2.2 Scope: Our proposed scopes of work, dated June 6, 2004, and outlined the scope of our services. It was not the intent of this investigation to fully comply with the Home Depot Design Manual requirements for the number of borings on the site (21 borings in the building area and on a 100-foot grid across the entire site). The actions undertaken during the investigation are summarized as follows.

2.2.1 The Home Depot Store Geotechnical Investigation Requirements, National Edition (Part Two, dated January 9, 2004), was reviewed.

2.2.2 A Site Plan for the proposed project, prepared by Scott A. Mommer Consulting, dated September 8, 2005 was reviewed. The plan is referred to, hereinafter as the Site Plan.

2.2.3 A Conceptual Grading and Drainage Plan for the proposed project, prepared by Scott A. Mommer Consulting, dated September 9, 2005 was reviewed. The plan is referred to, hereinafter as the Grading Plan.

2.2.4 The following plans, prepared for the Kmart Store #4342 by Herbert W. Angel, A.I.A., dated December 1968, were reviewed:
- Grading
- Structural
-Architectural

2.2.5 The following plans, prepared by Tait & Associates, dated April 15, 1997, were reviewed:
-Grading Plan for ADA Parking

2.2.6 The following refurbishment plans, prepared for the Kmart Store #4342 by Deusch Associated were reviewed:
- Structural
-Architectural

2.2.7 A map showing seismic hazard zones (Thousand Oaks Quadrangle), prepared by the California Department of Conservation, Division of Mines and Geology, dated November 17, 2000, was reviewed.

2.2.8 A site reconnaissance and subsurface exploration were conducted.

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

2.2.10 Mr. John Teeter (Home Depot) and Mr. Scott Mommer (Scott A. Mommer Consulting) were consulted during the investigation.

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

2.2.12 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 Description: The project site comprises approximately 9.03 acres located at 325 Hampshire Road in Thousand Oaks, California. A site location map is presented on Drawing No. 1 in Appendix A. The site was bound to the north by light industrial buildings; to the east by Hampshire Road; to the south by an adjacent vacant retail building with Foothill Drive beyond; and to the west by an approximately 20 to 25 foot tall retaining wall with Foothill Drive beyond.

At the time of the initial field exploration conducted in June 2004, the project site was occupied by an existing Kmart store building, garden shop, associated asphaltic concrete (AC) parking and drive areas, and isolated landscape areas. For the purpose of discussion, this portion of the Home

Depot site is referred to, hereinafter, as the Kmart property. Underground utilities, including electric, telephone, water, irrigation, sewer, and storm drain lines, were noted throughout the Kmart property.

An existing Burger King restaurant and a gasoline service station are located southeast of the Kmart store, at the northwest corner of the intersection of Foothill Drive and Hampshire Road.

According to Sheet SD-1 of the Grading Plans for the Kmart, the site elevations within the portion of the project site occupied by the Kmart development ranged from 927 feet above mean sea level (MSL) at or around the building grade to 908 feet MSL near the eastern section of the Kmart property (at the parking entrance off Hampshire Road). The finished floor elevation of the Kmart interior floor slab is reported to be 926.84 feet AMSL.

3.2 **Site History:** At the time of the initial field investigation in June of 2004, the subject site was occupied by an exiting Kmart store building, including existing asphaltic concrete parking and drive areas associated with the store. We understand the store is no longer in operation.

The existing store was originally built in the late 1960's to early 1970's. At some point in the mid 1990's, the Kmart store was remodeled, which included the expansion of the store to include the original K-Foods as part of the sales area. According to the plans, the existing Kmart project was reconstructed in the late 1990's to include handicap parking in the existing parking lot.

3.3 **Previous Studies:** Twining was not provided any previous geotechnical reports to review during this investigation. If these reports are available, they should be provided to Twining for review.

3.4 Existing and Anticipated Construction:

3.4.1 **Existing Kmart**: The remodel Kmart Shopping Center plans dated October 1993 indicate that the Kmart building (not including the garden center) comprises approximately 104,443 square feet in plan dimensions. Plans provided by Scott Mommer with Scott A. Mommer Consulting indicated that the original garden shop comprises approximately 5,023 square feet in plan dimensions. In the early 1990's the existing Kmart was remodeled, incorporating the pre-existing K-Foods store into the Kmart store. The remodel also included establishment of a new Little Caesars restaurant comprising approximately 1,862 feet in plan dimensions.

We understand the previously existing Auto Center had been remodeled into the existing Garden Center. It is presently unknown if the hydraulic lifts included as a part of the original Kmart autocenter were removed as a part of this remodel.

The existing building is a single-story structure with concrete masonry unit (CMU) walls, a steel frame roof, and concrete slab-on-grade floors. Review of the Kmart structural plans indicate that the structure is supported by a shallow spread foundation system. The general foundation notes on page S-2 of the 1969 Structural Specifications indicate minimum embedment of footings shall be 2 feet below adjacent finish grade or bottom of slab, and a design soil bearing pressure of 1,500 psf. However it should be noted that plans indicated on S-6 of the structural plans that the footings shall be 18 inches below the bottom of the slab.

In addition to the existing structure, existing 20 to 25 feet high CMU retaining walls are present west of the Kmart store building. The retaining walls support an existing hillslope and Foothill Drive beyond to the west. Based on the Grading Plan, we understand the existing retaining walls will be removed and replaced with new retaining walls to support the grade changes. The remaining portion of the site consists of asphalt concrete parking and drive areas.

The following table comprises thicknesses and reinforcement of the slabs indicated on the foundation plan of the Kmart Structural Plans.

Table No. 1
Thicknesses and Reinforcements of the Kmart Store Building

Location of the Slab	Thickness of the Slab, inches	Reinforcement
Original Automotive Center	6	#3 Rebar reinforcement spaced 18 inches
Kmart Sales area, Storage Rooms, and Original K-Foods	6	#3 Rebar reinforcement spaced 24 inches

The plans available for our review indicated standard asphaltic concrete (AC) to be 2 inches over 4 inches of aggregate base (AB) and heavy duty pavements to be 2 inches of AC over 6 inches of AB.

3.4.2 **<u>Proposed Home Depot</u>**: The proposed development will include demolition and removal of the existing structures, underground utilities, hardscapes and parking and drive areas. The planned development will include a new Home Depot store building and a garden center. The total building area of the proposed Home Depot will comprise approximately 110,098 square feet in plan dimension. The approximate locations of the proposed Home Depot store and garden center are shown on the site plan, Drawing No. 2 in Appendix A.

The Home Depot store is anticipated to be a single-story building consisting of CMU or concrete tilt-up walls and a combination wood and steel frame roof with a concrete slab-on-grade floor. The development will include underground utilities, screen walls, retaining walls and paved parking and drive areas. A new screen wall is being proposed adjacent to Foothill Drive at the western property line. Based on the Grading Plan, the screen wall will be constructed of cast-in-place concrete or CMU wall construction. Screen walls will be supported on a driller concrete pier foundation system. New retaining walls are planned to replace the existing CMU retaining walls west of the proposed Home Depot store. Based on the Grading Plan, the retaining walls are planned with heights ranging from 8 feet to about 32 feet and will be constructed of cast-in-place concrete. The higher sections of wall will be supported on a drilled concrete pier foundation and the shorter sections of wall will be supported on a shallow foundation system.

In addition, the project will include removal and replacement of the existing asphalt concrete pavements. Maximum loads for the Home Depot store of 5.0 kips per lineal foot for bearing walls, 120 kips for columns, and a slab-on-grade live load of 650 pounds per square foot were specified by the Home Depot Design Criteria Manual. Tolerable total and differential settlements due to anticipated dead plus live loads of 1-inch and ½-inch in 50 feet, respectively, were stipulated by the Design Manual for the purpose of design.

The proposed development will include driveways and parking for automobile and truck traffic. Equivalent 18 kip axle loads (EAL) of 50,000 and 220,000 for a design life of 10 years were indicated in the Design Manual for the Home Depot "standard duty" and "heavy duty" pavement sections, respectively.

The finished floor elevation for the proposed Home Depot Store is anticipated to be approximately 3.64 feet below the finished floor elevation of the existing Kmart building. Therefore, earthwork cuts of up to about 3.6 feet are anticipated in the building pad area to achieve the new building pad elevation and to provide positive site drainage. These estimates do not include additional over-excavation required to provide engineered fill below the proposed foundations as recommended in this report, or trenching and backfill of utility excavations.

4.0 <u>INVESTIGATIVE PROCEDURES</u>

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

4.1 <u>Field Exploration</u>: The field exploration consisted of a site reconnaissance, drilling test borings, coring the existing concrete slabs-on-grade, soil sampling, standard penetration tests, and cone penetrometer test (CPT) soundings.

4.1.1 <u>Site Reconnaissance</u>: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Jerry Kazynski on June 11 and Mr. Dean Ledgerwood on June 21, 2004. The features noted are described in the background information section of this report.

4.1.2 <u>Drilling Test Borings</u>: The depths and locations of test borings were selected based on the size of the structure, type of construction, estimated depths of influence of proposed surface loads, and the subsurface soil conditions.

On June 21, 2004, six (6) test borings were drilled in the general areas proposed for the new building addition, new Garden Center, new pick-up canopy, overbuild zone of the existing Kmart store building, new proposed parking and drive areas, and existing parking and drive areas associated with the existing Kmart store to depths of between 11.5 and 50 feet BSG. Four (4) borings were drilled inside the existing Kmart building to depths of between 5 and 15 feet BSG. Auger refusal was encountered, assumed to be from moderately cemented, very dense soil, in each of the test borings drilled in the existing building (B-1, B-2, B-3, B-4) at depths of between 5 and 15 feet BSG. Seven

(7) bulk samples of soil were obtained for Resistance (R)-value, sieve analysis, Atterberg limits, chemical analysis, and moisture-density relationship tests. The test boring and bulk sample locations are shown on Drawing No. 2 in Appendix A. The exterior test borings (outside the existing building) were drilled using a CME-75 drill rig equipped with 6⅝-inch outside diameter (O.D.) hollow-stem augers. The interior test borings were drilled using a limited access Beaver tri-Pod drill rig equipped with 6-inch O. D. solid-flight augers. The test borings were drilled under the direction of a Twining staff geologist. The soils encountered in the test borings were logged during drilling by a staff geologist. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

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

Test boring locations were determined by pacing with reference to the southwest corner of the existing Kmart store building. The locations, as described, should be considered accurate to within about 10 feet. The locations of the test borings are described on the boring logs in Appendix B of this report. Elevations of the test borings were not surveyed as a part of the investigation; however, they were interpreted from the Kmart As-Built Topographic Plans and Grading Plans. The test borings were loosely backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated at the boring locations.

4.1.3 <u>Cone Penetration Test (CPT) Soundings</u>: On June 21 and June 29, 2004, six (6) CPT soundings were advanced to depths ranging from 16 to 50 feet BSG in the overbuild zone of the existing building, the area proposed for the new addition, the proposed Garden Center area, and the proposed pick-up canopy area. It should be noted that refusal was encountered in two (2) of the CPT soundings (CPT-2 and CPT-3) at depths of 16 and 17 feet BSG, this refusal appears to be due to very dense, moderately cemented soil. The soundings were conducted under the direction of a Twining project geologist. The CPT locations are shown on Drawing No. 2 in Appendix A. CPT methods were used to obtain nearly continuous soil behavior type and penetration resistance information.

The CPT soundings were performed by Kehoe Testing and Engineering using an electronic piezocone with a 60-degree apex angle and a diameter of 35.7 millimeters (about 1½ inches) hydraulically advanced using a 30-ton CPT rig in accordance with ASTM Test Method D3441. CPT measurements of cone bearing resistance, sleeve friction, and dynamic pore water pressure were recorded at 0.25 foot intervals during penetration to provide continuous logs of the soil behavior types. The CPT logs are presented in Appendix B of this report.

CPT sounding locations were determined by pacing with reference to the southwest corner of the existing Kmart store building. The locations, as shown on Drawing No. 2 (Appendix A), should be considered accurate to within 5 feet. The sounding holes were filled with bentonite grout.

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

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

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

4.1.5 Concrete Slabs-On-Grade Coring: On June 21, 2004, the existing Kmart store interior slab was cored at four (4) locations. The concrete cores were returned to our laboratory in order to measure the core thickness and determine the approximate size and location of reinforcement.

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

The results of laboratory tests are summarized on Figure Numbers 1 through 14 in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B of this report.

5.0 FINDINGS AND RESULTS

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

5.1 Condition of Interior Slabs-on-Grade: Visual observations of the interior slabs-on-grade revealed the presence of hairline to ¼ inch wide longitudinal cracks throughout the slab in the Auto Center. Cracking was not noted in the interior sales area slab-on-grade; however, the majority of the interior slab-on-grade was covered with floor covering or merchandise.

The existing Kmart store interior slabs-on-grade were cored to determine the slab thicknesses at several locations. The test locations are shown on Drawing No. 2 in Appendix A. The findings at the coring locations are summarized in Table No. 2.

Table No. 2
Interior and Exterior Slabs-On-Grade - Summary of Core Tests

Test Boring Designation	Location	Average Core Thickness, inches *	Location of reinforcement, inches below top of the slab *
B-1	Southeast section of store sales area	7.3	None Detected
B-2	Northeast portion of storage area	7.3	Bottom of Core
B-3	Northwest portion of Auto Center	6.7	Center of Core
B-4	Southwest portion of storage room area	5.7	None Detected

* Note: Measurements made to the nearest 1/10- inch using a micrometer.

It should be noted that a vapor barrier was noted beneath the slab-on-grade.

5.2 **Asphaltic Concrete Pavements:** The asphaltic concrete pavements located around the existing store were noted to be in poor to very poor condition. Longitudinal and alligator cracking were noted to be worse in the front parking area north of the store. The pavements located in the rear of the store were noted to be in poor to fair condition. The pavements were drilled to determine the thicknesses of the asphaltic concrete and aggregate base. In addition, the types of subgrade materials were noted during the coring operations. These materials were also sampled for laboratory testing. The results of the measurements are summarized in Table No. 3.

Table No. 3
Thicknesses of Asphaltic Concrete Structural Section and Subgrade Conditions

Test Location	Pavement Location	AC Thickness, Inches *	AB Thickness, Inches **	Subgrade
B-5	Southeast corner adjacent to the existing Kmart building	3.00	6.00	Lean Clay, Sandy
B-6	Northeast corner adjacent to former Auto Center	4.00	7.50	Fill Silty Sand
B-7	North section of Parking	3.00	3.00	Lean Clay, Sandy
B-8	North section of Parking	4.50	4.50	Lean Clay, Sandy
B-9	Parking Area	2.00	2.50	Lean Clay, Sandy
B-10	Parking Area	4.50	Not encountered	Lean Clay, Sandy

* - AC = asphaltic concrete, Measurements made to the nearest ¼ inch.

5.3 **Soil Profile:** Fill soils were encountered in boring B-6, drilled in the existing parking area, to a depth of approximately ten (10) feet below the existing site grade. The fill soils encountered in boring B-6 generally consisted of silty sand. The existing fill soils are anticipated to be localized, since they were encountered in the area(s) reported to contain underground storage tanks. In addition, queried fill soils were encountered within the southeast portion of the Home Depot building pad area. The native soils encountered in each of the boring locations consisted of clays to at least the maximum depth explored of 50 feet. As a part of site preparation, all existing fill soils will require removal and compaction as engineered fill.

These subsurface descriptions constitute a general summary of the soil conditions encountered in the test borings drilled and the CPT soundings conducted for this investigation. Detailed descriptions of the soils encountered at each test boring are presented on the logs of borings in Appendix B. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

5.4 Soil Engineering Properties: The silty sand fill soils encountered in test borings B-6, drilled within the existing parking lot, were dense to very dense as indicated by standard penetration resistance, N-values, of up to 44 blows per foot. The moisture content of the fill soil was on the order of 24 percent.

The native lean clay soils encountered at the boring locations were medium stiff to hard as indicated by standard penetration resistance, N-values, ranging from 9 to greater than 100 blows per foot. The moisture content of the soils ranged from approximately 17 to 30 percent. Two (2) in-place density tests revealed dry densities of 104 and 106 pounds per cubic foot. The soils exhibited moderate compressibility characteristics with the addition of moisture. Upon inundation, the soils exhibited low collapse potential.

R-value tests were conducted on two (2) near surface native lean clay soil samples collected from below the proposed parking and driveway areas, between existing surface grade and a depth of about 3½ feet BSG. The results of the tests indicate R-values of 24 and less than 5.

Chemical tests performed on a near surface lean clay soil sample indicated a pH value of 7.7, a minimum resistivity value of 6,200 ohms-centimeters, and 0.0014 and 0.0026 percent by weight concentrations of sulfate and chloride, respectively.

5.5 Groundwater Conditions: Groundwater was not encountered in any of the test borings drilled for the investigation on June 21, 2004. The in-situ moisture contents of the encountered soils were, however, above the optimum moisture content.

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

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 conditions determined from the investigation and our understanding of the proposed construction.

6.1 Subsurface Conditions: Fill soils encountered in the proposed Home Depot building pad area extending to a depth of up to 10 feet below the existing ground surface elevation. The fills were encountered in an area where underground storage tanks may have been located. The existing fill soils appeared to range from loose to dense and foundations supported on the existing fill soils would be subject to static settlements in excess of the Home Depot criteria. As part of the site preparation, over-excavation and compaction of all existing fill soils is recommended.

The primary geotechnical concerns for design and construction of the proposed project are: 1) potentially compressible clay soils near the anticipated depths of the foundations for the proposed building; 2) loose soils that will be generated as a result of the removal of the existing improvements; 3) the moderate expansion potential of the near surface soils; 4) the moderately corrosive nature of the near surface soils; 5) support of temporary excavations adjacent Foothill Drive; and, 6) the high moisture content of near surface soils (above the groundwater table) that may require stabilization during site preparation. As a result, remedial grading (over-excavation and compaction of engineered fill) is considered necessary in order to provide foundation support for the proposed structures.

6.2 Expansive Soils: One of the geotechnical concerns 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 sections underlying foundations and slabs-on-grade.

In evaluation of the expansive soils at the site, expansion testing was performed on representative samples of the near surface soils which are anticipated to be within the zone of influence of planned improvements. The expansion testing was performed in accordance with UBC Standard 18-2. The soils tested were classified by expansion potential in accordance with UBC Table 18-1-B and are summarized in Appendix C of this report. Based on the results of testing performed to date, the expansion potential of the onsite soils is considered moderate. Therefore, an imported, non-expansive fill is recommended below slabs-on-grade for this project.

6.3 Static Settlement and Bearing Capacity of Shallow Foundations: The potential for excessive total and differential static settlements of foundations and slabs-on-grade is a geotechnical concern which should be evaluated for this building site. 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 should be evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Based on the subsurface data and laboratory testing performed as part of this report, static settlement calculations were performed. Calculations indicate that static settlement of foundations placed directly on native soils at the proposed foundation depth would exceed the Home Depot design criteria for differential settlements of ½ inch in 50 feet. To reduce the estimated static total and differential settlements to comply with the Home Depot criteria, the foundations would need to be supported on a uniform thickness of engineered fill.

Total and differential static settlements due to combined anticipated foundation loads were estimated considering: 1) the compressibility of the native soils following the recommended site preparation; 2) the structural loads anticipated, and 3) the use of a maximum allowable net bearing pressure of 1,500 pounds per square foot for dead-plus-live loads.

Net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and the weight of the concrete footing may be neglected in design. The net allowable soil bearing pressure presented was selected to satisfy both the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A factor of safety of 3 was used to determine the allowable bearing capacity based on the Terzaghi equations.

6.4 Interior Slab on Grade Construction: Several issues need to be considered to limit the potential for damage to slabs during construction. These issues include: 1) using perimeter pour-strips at tilt-up and CMU wall locations to avoid damage to slab-wall connections; 2) differential slab movement at interior columns; 3) aggregate base sections below the slabs, and 4) crane and construction equipment loads on the slabs.

Depending on the sequence of slab loading and the location of wall construction, damage to slabs from differential loading conditions could occur. It has been our experience that a concentrated amount of differential movement and damage at the slab-to-perimeter footing location can occur as heavy load bearing walls are placed and the footing is loaded. This typical procedure can result in cracking of slabs and foundations due to differential movement. This potential damage can be reduced by leaving a perimeter pour strip between the wall footing and the adjacent slabs. After the walls are erected, and a majority of the static movement has occurred, the pour strip can be placed.

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

It has been our experience that placing concrete for the concrete slabs by the tailgating method can cause subgrade instability due to the high frequency of concrete trucks which travel across the prepared subgrade. Compacted subgrade can experience instability under high traffic loads resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. A layer of aggregate base (AB) can reduce the potential for instability under the high frequency loading of concrete trucks. Also, the improved support characteristics of the AB can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design with at least 6 inches of AB for constructability and design purposes.

Finally, it is expected that erection of concrete tilt-up wall panels and roof steel will require cranes and heavy construction equipment. It should be noted that cranes and heavy construction equipment can impart intense loads on slabs and pavements. The loaded track pressure of cranes and/or heavy construction equipment that will operate on slabs or pavements should be assessed by the contractor prior to placing equipment on the slab.

6.5 **Cut Slopes**: Preliminary recommendations are provided in this report for permanent and temporary cut slopes based on soil conditions encountered in our test borings. It should be noted that the maximum slope grades provided are estimated to be appropriate for most of the slopes. However, considering the variable soil types, the steepest grades recommended will not be appropriate for all slopes. Contingencies to flatten slopes should be developed by the contractor prior to cutting so that unsafe temporary and permanent slopes can be safely flattened, or shored, without impacting adjacent improvements, habitats, or vegetation. Cut slopes should be observed by a qualified geotechnical engineer/engineering geologist during excavation to assess stability.

6.5.1 **Permanent Cut Slope Grades:** Cuts of as much as about 32 feet below original site grades are anticipated near the west end of the proposed site, behind the proposed Home Depot store to allow construction of the proposed retaining wall. On a preliminary basis, for permanent slopes, in general, the upper soils and should be graded at 3 horizontal (H) to 1 vertical (V) or flatter. These soils are anticipated to occur above depths of about 5 feet BSG on the slopes at and behind the existing retaining walls along the west side of the site. Existing slopes exposing these types of materials in the vicinity of the site were observed to be steeper, however, given the potential for erosion and surface sloughing, flatter slopes are recommended. Therefore, on a preliminary basis, it is expected that soil to a depth of about 5 feet BSG will have to be graded at 3 H to 1V or flatter. Remaining portions of the slope may be graded at a 2 H to 1 V (i.e., soils below a depth of 5 feet).

Slopes present stability hazards. Considering the heights of the cut slope proposed west of the Home Depot store, and the proximity of existing Foothill Drive upslope to the west, it is recommended that the retaining walls be designed for appropriate surcharge loading. In addition, a more detailed evaluation will be necessary to prepare recommendations for design of these retaining walls.

6.5.2 Temporary Cut Slope Grades: The soils can support temporary slopes steeper than those recommended for permanent slopes. However, additional evaluations are required to prepare recommendations for temporary excavations. Based on the proximity of existing Foothill Drive, temporary shoring of the excavations for the new retaining walls west of the Home Depot store will be required. Shoring should be designed by the contractor based on at-rest earth pressures and using applicable surcharges. Any damage to the existing improvements to remain should be repaired by the contractor at no cost to the Owner.

The following discussion is provided for planning purposes only. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. Based on OSHA excavation guidelines, the upper soils (fill and native) should be temporarily sloped based on a Type C condition, at 1½H to 1V or flatter.

6.6 Shoring, Construction Monitoring and Conditions Survey: Construction of the new retaining walls adjacent to Foothill Drive will require temporary shoring of the excavations. The shoring will need to be designed by a qualified engineer and installed without damage to the existing public street and adjacent improvements. It is recommended a preconstruction survey of the conditions of the existing Foothill Drive and public improvements be completed by the contractor including photographs and a written description of existing distress. This information should be provided to Home Depot and Twining prior to start of construction.

The contractor should determine the requirements for shoring, underpinning, etc. based on the recommendations of this report. In no case should the existing public improvement be undermined. The contractor should include in their bid the cost to design and construct shoring of the temporary excavations that must extend deeper than the bottom of existing public street improvements bordering the west side of the property or if excavations extend below a zone defined by a 2 horizontal to 1 vertical line that extends downward from the bottom of foundations. These areas should be shored or underpinned to prevent damage to the existing foundations. The contractor should monitor the movement of existing adjacent streets on a daily basis during construction by use of a level survey. If the existing public improvements move more than ⅛ of an inch, the Contractor should undertake remedial actions to prevent movement of the existing public improvements such as shoring, etc.

6.7 Ground Rupture and Seismic Ground Motion: The project site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the Malibu Coast fault, with the surface trace of the fault located about 10.2 kilometers from the site. The potential for fault rupture at the site is therefore considered low.

Seismic ground motion estimates were developed to conduct the liquefaction hazard analyses. The "Design Basis Ground Motion," Section 1627 of the California Building Code (CBC), is defined as the seismic ground motion having a 10 percent probability of being exceeded in a 50-year period. The probabilistic analyses described in this section was used to determine the design basis ground motion.

Probabilistic ground motion evaluation requires use of a seismicity model and ground motion attenuation functions to approximate the modification of seismic waves between the earthquake hypocenter (source) and the site. The seismicity model, including the location and fault parameters (such as slip rate, fault length, magnitude and rupture area) of faults capable of impacting the site, were based on published geologic papers and correspond with those listed in the California Geological Survey (CGS) database entitled "California Fault Parameters." Multiple probabilistic evaluations were conducted using the FRISKSP computer program and the faults indicated as those active and potentially active faults listed in the "California Fault Parameters" database.

Our evaluation considered the average of the predicted design basis ground motions for two separate analyses incorporating the ground motion attenuation relationships of Idriss (1994) and Abrahamson and Silva (1997), the active and potentially active faults within 100 kilometers of the site, including the San Andreas Fault located about 67 kilometers from the site. The average of design basis site accelerations based on the above attenuation relationships was determined to be 0.56g. Accordingly, a ground motion of 0.56g was selected for use in the liquefaction analyses. This represents a value not weighted for magnitude. Magnitude weighting is conducted in the liquefaction analysis.

Hazard deaggregation was conducted using the FRISKSP computer program. The results indicate that an earthquake magnitude of 7.8 represents the predominant earthquake magnitude for the site (Hollywood fault). This earthquake magnitude was used with the above ground motion estimate for the liquefaction analyses.

It is expected that the 2001 CBC will be used for structural design, and that seismic site coefficients are needed for design. Based on the CBC, the site classification is estimated to be a stiff soil S_D site with standard penetration resistance N-values averaging between 15 and 50 blows per foot in the upper 100 feet BSG.

The site coefficients for acceleration and velocity are based on the distance and activity of the local faults. Digitized seismic models published by the CGS indicate that the Malibu Coast Fault is located about 10.2 kilometers from the site. The maximum magnitude of the Malibu Coast fault is indicated to be 6.7, with a slip rate 0.3 mm/year. The site does not require near-source corrections (CBC Tables 16-S and 16-T) based on a seismic source Type B. Therefore, the values of the near-source acceleration factor, Na, may be taken as 1.0 and the near-source velocity factor, Nv, may be taken as 1.0. Based on these values, the seismic acceleration coefficient, Ca (Table 16-Q), would be 0.44, and the seismic velocity coefficient, Cv (Table 16-R), would be 0.64.

6.8 **Liquefaction and Seismic Settlement:** Based on review of the Seismic Hazard map of the Thousand Oaks Quadrangle, (2000), the subject site is not located in a Seismic Hazard Special Studies Zone for liquefaction which delineate areas of historical occurrence of liquefaction or local geological, geotechnical and groundwater conditions indicating a potential for permanent ground displacement such that a mitigation as defined in Public Resources Code Section 2693© would be required.

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.

Liquefaction and seismic settlement analyses were conducted based on soil properties revealed by test borings and CPT soundings. A design basis earthquake acceleration of 0.56g and a design earthquake magnitude of 7.8 were used. The N-values generated based on the CPT results were used to determine the cyclic stress ratio needed to initiate liquefaction. The N-values from the CPT data were relied upon in the evaluations. The CPT data is considered more reliable than the N-values from the hollow stem auger borings, due to the controlled-rate push of the CPT. Soil parameters, such as wet unit weight, N-value, fines content, and depth of N-value tests, were input for the soils layers encountered throughout the depths explored (see test boring logs, Appendix B).

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). The results of liquefaction analyses indicate liquefaction would not occur. Based on our evaluations, estimates of total seismic settlement are estimated to be on the order of less than 0.1 inches. Therefore, the potential for seismic settlement at this site is considered very low.

6.9 **<u>AC Pavements</u>:** 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 was performed in accordance with Caltrans Test Method 301.

The plans indicate that the K-mart "standard duty" pavements consist of 2.5 inches of asphaltic concrete over 4 inches of aggregate base over 18 inches of compacted subgrade. The "heavy duty" pavements are indicated to consist of 3.5 inches of asphaltic concrete over 5 inches of aggregate base over 18 inches of compacted subgrade. It should be noted that the difference between the "standard duty" and the "heavy duty" pavements is 1 inch of asphaltic concrete and 1 inch of aggregate base. The actual thicknesses of the AC and AB in the paved areas vary across the site. The findings did not reveal a clear trend regarding different structural sections for the driveways and the standard duty pavement areas. It does not appear that the in-situ pavements comply with the original design requirements on a consistent basis. The AC and AB thicknesses measured in test borings ranged from 2.0 to 4.5 inches and 0 to 7.5 inches, respectively.

The existing pavement sections were observed to be in a poor condition and do not comply with the minimum Home Depot criteria based on traffic index values of 6.5 and 7.5, and a design R-value of 5. As a comparison, the design section for standard duty pavements based on the minimum Home Depot criteria would be 3.5 inches of AC over 14.5 inches of AB. The design section for heavy duty pavements based on the minimum Home Depot criteria would be 4.0 inches of AC over 17.5 inches of AB.

In our opinion, the pavements would have to be removed and replaced to comply with the Home Depot Design Criteria.

Therefore, recommendations for new 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 Chapter 600 of the California Department of Transportation Highways Design Manual (fifth edition). The traffic loading data were obtained from the geotechnical investigation report specifications provided by Home Depot U.S.A., Inc. The "standard duty" pavement should be designed for a life of 10 years and an EAL (18 kips) of 50,000 axles. An EAL of 50,000 equates to a traffic index of 6.5. The "heavy duty" pavement was designed for a life of 10 years and an EAL (18 kips) of 220,000 axles. This equates to a traffic index of 7.5. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

6.10 **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 Resistance or R-value of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL) from the loading of heavy trucks called a traffic index (T.I).

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 Caltrans Test Method 301.

The EALs for each of the pavement sections were converted to the number of 5-axle trucks per day, one direction, anticipated for the proposed store. The EAL for the "standard duty" pavement section of 50,000 was converted to 14 axles or 6 five-axle trucks per day. The EAL for the "heavy duty" pavement section is 220,000 or 26 five-axle trucks per day. The recommended structural sections were based primarily on the Portland Cement Association "Thickness Design of Highway and Street Pavements."

The PCC pavement sections were designed for a life of 10 years, a load safety factor of 1.1, a single axle weight of 12,000 pounds, a tandem axle weight of 36,000 pounds. A modulus of subgrade reaction, K-value, for the pavement section, considering a minimum 6-inch layer of aggregate base material (minimum R-value of 78), of 150 psi/in at the top of the aggregate base was used for pavement design.

6.11 **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 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. Twining does not provide corrosion engineering services.

6.12 <u>Sulfate Attack of Concrete</u>: Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The test results are then compared with the categories of the 2001 California Building Code, Table 19-A-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.

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

It should also be noted that the near surface soils encountered during the field exploration had moisture contents above the optimum moisture content. Contractors should anticipate that exposed subgrade soils will have high moisture contents and soils drying, chemical treatment, or other means of stabilization will be necessary. The costs to stabilize the soils should be included in the contractors bid.

Based on the recommended engineered fill soils below the existing floor slab, and a 6-inch layer of aggregate base, a modulus of subgrade reaction of 150 pounds per square inch per inch may be used for design. The 6-inch layer of Class 2 aggregate base material is also recommended below new interior slabs for construction considerations and to provide a capillary break.

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, we present the following general conclusions.

7.1 The site is considered suitable for the proposed construction with regard to support of foundations and concrete slabs-on-grade, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and construction monitoring by Twining are integral to this conclusion.

7.2 The existing AC parking and drive area pavement sections were observed to be generally in a poor condition. The existing pavement sections do not comply with the Home Depot standard or heavy duty designs and appear to have served their projected design life. For the majority of the existing pavement areas, it is recommended that the existing AC section and underlying AB be removed and replaced with a pavement section that complies with the Home Depot criteria.

7.3 Consideration of foundation embedment for frost depth is not required since the mean monthly temperature of the Thousand Oaks area is above freezing.

7.4 Shallow spread footings placed entirely on at least 2 feet of engineered fill, engineered fill extending to a depth of 5 feet below the existing site grades, or engineered fill that extends 1 foot below any former subsurface structures, etc., whichever provides the deeper fill, can provide adequate support for the proposed structure with regard to static settlements.

7.5 Combined settlements (static and seismic) of 1 inch total and 0.5 inch differential over a horizontal depth of 50 feet should be anticipated for design.

7.6 The analytical results of a soil sample analysis indicate that the near-surface soils exhibit a "moderately corrosive" corrosion potential to buried metal objects.

7.7 The analytical results of a soil sample analysis indicate a "negligible" potential for sulfate attack on reinforced concrete placed in the near-surface soils (CBC Table 19-A-3).

7.8 The near-surface soils exhibit poor support characteristics for pavements.

7.9 Groundwater was not encountered in any of the borings drilled at the subject site. Based on the lack of free water in the open boreholes and the moisture content of the collected soil samples, it is assumed that groundwater existed at a depth in excess of 50 feet at the time of our subsurface exploration. It should be noted, however, that the soils encountered at the boring locations possessed moisture contents in excess of the optimum moisture content. As a result, unstable soil conditions may impact construction and soil stabilization may be required for the project.

8.0 RECOMMENDATIONS

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

8.1 General

8.1.1 Although plans for the existing Kmart building were reviewed by Twining, it is recommended that these plans be reviewed by the contractor prior to beginning demolition and grading activities. The depth of existing foundations should be confirmed by the contractor to verify if they are consistent with the depths assumed in this report prior to earthwork operations. It is anticipated, based on a cursory review of the available plans, that the existing building foundations extend to a depth of approximately 18 inches below the existing slab-on-grade.

8.1.2 A demolition plan should be developed to identify existing improvements which will require removal. As a minimum, this plan should show the structural elements planned for removal. The structural elements shown on the demolition plan should be removed in their entirety and the resulting excavations backfilled with imported, non-expansive engineered fills under the observation of Twining.

8.1.3 Foundation plans for the proposed Home Depot were not provided for review at the time this report was prepared. When completed, our firm should be provided the opportunity to review the final grading plans and foundation details, and provide amended recommendations as necessary.

8.1.4 A preconstruction meeting including, as a minimum, the owner, general contractor, foundation and paving subcontractors, and 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 The contractor is responsible for including in the base bid the costs to perform the work required by the Geotechnical Report, the project plans, the project specifications, and the City of Thousand Oaks, whichever is most stringent. After review of the aforementioned documents, the contractor(s) bidding on this project should determine if the data are sufficient for accurate bid purposes. If the data are not sufficient, the contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect more data as required to prepare accurate bids.

8.1.6 The contractor is responsible for protecting existing facilities from damage including but not limited to subdrainage systems (if present), adjacent fences, buildings, streets, etc. Any damage shall be repaired by the contractor at no cost to Home Depot.

8.1.7 The contractor should use appropriate low-pressure equipment to achieve the required over-excavation and compaction and minimize subgrade rutting and subgrade instability.

8.1.8 Building pad over-excavation should include the building and garden center areas and a minimum of five (5) feet beyond the store building, and garden center, or by the depth of fill, whichever is greater. The building pad over-excavation should also include areas to be occupied by parking decks, vestibules, utility pads, building apron, front lumber doors and front lumber canopy area, rear lumber pad area, customer pick-up area, stairs, ramps, stoops, loading docks, truck loading wells (including excavation for truck loading wells) and trash compactors, and to a minimum of 5 feet beyond these improvements. The lateral overbuild distance for all of the above structures and improvements should be at least 5 feet, or equal to the depth of fill at those locations, whichever is deeper. If over-excavation laterally beyond the proposed footings is not feasible due to existing structures to be preserved, Twining should be contacted for additional recommendations on a case by case basis.

8.1.9 The contractor is responsible for compliance with the SWPPP requirements specified in the project plans, the project specifications, and the City of Thousand Oaks, whichever is most stringent.

8.1.10 Prior to placement of asphaltic concrete adjacent to slabs-on-grade, curbs, gutters, the contractor shall compact the area immediately adjacent to these features with equipment that can provide adequate compactive effort to the aggregate base adjacent to the vertical face of the concrete to achieve a dense, non-yielding condition. These compaction operations should be observed by Twining.

8.1.11 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for over-excavation, aggregate base, non-expansive soils and native soil moisture conditioning as recommended in this report for interior slabs-on-grade, AC pavements, and PCC pavements.

8.1.12 Contractors should be aware all existing undocumented fill soils (including those beyond the limits of the building pad) should be over-excavated and compacted as engineered fill as a part of site development.

8.1.13 Fly ash should not be used in the preparation of the building pad subgrade, import fill, aggregate subbase, aggregate base, etc. Use of flyash or materials containing flyash is prohibited.

8.1.14 If the existing Portland cement concrete, asphaltic concrete and aggregate base is considered for reuse on the site as an aggregate subbase or fill material, this use must be approved by Home Depot and Twining. If used as a subbase, the material shall be free from organic matter and other deleterious substances. Aggregate subbase may include material processed from reclaimed asphalt concrete, Portland cement concrete, or a combination of these materials. Aggregate subbase should meet the grading requirements of Section 8.10 of this report and should have a minimum R-value (CTM 301) of 50 and a minimum sand equivalent (CTM 217) of 18. The Contractor shall assume for the purpose of bid, that these materials cannot be reused as aggregate base or subbase. This material may be used as fill on the site provided it is processed to comply with the requirements of import fill as presented in this report, except that this material cannot be used in the preparation of the building pad due to the presence of asphaltic concrete.

8.1.15 Based on our review of the Grading Plan, it is anticipated that shoring will be necessary during grading to enable construction of the new retaining walls along the west side of the property. The contractor should determine the requirements for shoring and temporary excavations based on the recommendations of this report, any addendums, and the project plans. If the requirements for shoring cannot be determined from this report, the contractor should include in the bid the cost for any additional investigation deemed necessary. The base bid should include preparation and implementation of a shoring plan. Prior to the start of temporary excavation, a visual and photographic survey of adjacent structures and street improvements should be performed to document the condition of these features prior to the start of construction. A copy of the survey should be provided to the Owner, Architect, Structural Engineer and Twining.

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 structures or as necessary to establish positive drainage and preclude ponding of water adjacent to foundations. Adjacent exterior grades which are paved should be sloped at least one (1) percent away from the foundations.

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

8.2.3 Landscaping during and after construction should direct rainfall and irrigation runoff away from the structure and not promote ponding of water adjacent to the structures. Care should be taken to maintain a leak-free sprinkler system.

8.2.4 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended below the aggregate base section at least 4 inches into native subgrade soils. 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 is 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 structures and the resulting runoff directed away from the structures.

8.3 Site Preparation

8.3.1 The contractor is responsible for compliance with the SWPPP requirements specified in the project plans, the project specifications, and the City of Thousand Oaks, whichever is most stringent.

8.3.2 All construction debris, topsoil, vegetation, organics, and general debris should be removed from the proposed new building and pavement areas, as required. The general depth of stripping in existing landscape areas should be sufficiently deep to remove the root systems and organic topsoils. For estimating purposes, a minimum stripping depth of 6 inches should be used. The actual depth of stripping should be reviewed by our firm at the time of construction. Deeper stripping may be required in localized areas. 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.

8.3.3 The contractor should locate all foundations, slabs-on-grade, and subsurface structures to be demolished in the areas proposed for new footings, slabs-on-grade and new pavements. These structures should be entirely removed, where applicable, and the excavations should extend to at least 12 inches below the bottom of the structural element. The resulting excavations should be cleaned of all loose or organic material, the exposed native soils should be scarified to a depth of at least 8-inches, conditioned, i.e., wetted or aerated, to achieve a moisture content within two (2) to five (5) percent above optimum moisture content and compacted as engineered fill and the excavation backfilled with engineered fill. The contractor should note that the existing vapor barrier(s) should be removed completely in areas that will receive new slabs-on-grade.

8.3.4 The on-site soils should be over-excavated so that new foundations will be supported on a minimum of 2 feet of engineered fill or engineered fill extending to a depth of 5 feet below preconstruction site grades, whichever provides the deeper fill. The zone of over-excavation should be conducted throughout the entire building pad and overbuild zone. Slot cutting will not be allowed. The depth of engineered fill does not include the depth of scarification and compaction. Upon approval of the bottom of the over-excavation by a representative of Twining, the moisture content of the exposed soils should be conditioned (i.e., moisture conditioned or aerated) to approximately 2 percent over optimum moisture content, scarified to a minimum depth of 8 inches and compacted to a minimum of between 90 and 95 percent of the maximum dry density as determined by ASTM D1557.

8.3.5 Building pad over-excavation should include the building and garden center areas and a minimum of five (5) feet beyond the store building, and garden center, or by the depth of fill, whichever is greater. The building pad over-excavation should also include areas to be occupied by parking decks, vestibules, utility pads, building apron, front lumber doors and front lumber canopy area, rear lumber pad area, customer pick-up area, stairs, ramps, stoops, loading docks, truck loading wells (including excavation for truck loading wells) and trash compactors, and to a minimum of 5 feet beyond these improvements. The lateral overbuild distance for all of the above structures and improvements should be at least 5 feet, or equal to the depth of fill at those locations, whichever is deeper. If over-excavation laterally beyond the proposed footings is not feasible due to existing structures to be preserved, Twining should be contacted for additional recommendations on a case by case basis.

8.3.6 The project civil engineer should show the overbuild line on the project grading plan.

8.3.7 Areas to receive fill and all pavement areas (i.e., areas outside the building pad and overbuild zone) should be over-excavated to a minimum depth of 12 inches below preconstruction site grades, 12 inches below the existing subgrade elevation below pavements, to the depth necessary to remove all undocumented fill, and 12 inches below the bottom of any existing improvements to be removed (foundations, utilities, etc.), whichever provides the deeper fill. Upon approval of the over-excavation by Twining, the bottom of the over-excavation should be scarified a minimum of 12 inches in depth, moisture conditioned to near optimum and compacted to a minimum of 92 percent relative compaction.

8.3.8 Based on the nature of the subsurface soil conditions, it should be anticipated that unstable soil conditions will be encountered during excavations and installation of slabs-on-grade, foundations, utilities, etc. Therefore, the soils may require stabilization. The Contractor should note that the base bid should include stabilization in accordance with this report including the procedures in the report Appendices for Chemical Treatment of Soil. Stabilization of the subgrade soils should be performed in a uniform manner. If stabilization of the subgrade soils is necessary, it should be performed in the entire building area, including the overbuild zone.

8.3.9 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. Twining is not responsible for measuring and verifying the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and 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 Twining or excavating for foundations.

8.3.10 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for over-excavation, non-expansive soils, and native soil moisture conditioning as recommended in this report for interior slabs-on-grade, AC pavements, and PCC pavements.

8.3.11 The areas directly adjacent to vertical surfaces such as walls, slabs-on-grade, curbs, planters, etc. should be compacted using walk-behind vibratory rollers capable of achieving the required compaction.

8.3.12 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.

8.3.13 The use of shoring should be anticipated where slopes steeper than 2H to 1V are necessary to make the required excavations. The contractor should assess all areas requiring excavation to determine if shoring will be required. These costs should be included in bids for site preparation and building.

8.3.14 Shoring should, at a minimum, comply with the recommendations presented in this report for lateral earth pressures. It is recommended shoring plans and calculations be provided to Twining for review prior to the start of construction. All soils (if any) disturbed during the shoring installation / removal activity should be over excavated and compacted as engineered fill.

8.3.15 In no case should excavations extend below a 2H to 1V zone below existing utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs. Shoring should be designed and installed per the recommendations of a registered civil or structural engineer in the State of California. A shoring plan should be prepared by a registered engineer in the State of California. Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.3.16 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 adjacent public improvements occur, during or after excavation, the owners and Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury. All damage should be repaired by the Contractor at no additional cost to the Owner.

8.3.17 Any open graded gravel or rock material such as ¾-inch or ½- inch crushed rock used as backfill should be placed in 6-inch to 1-foot thick lifts and compacted using a vibratory compactor to a non-yielding condition as determined by our firm. Each lift must be approved by our firm prior to placing the next lift. All open graded materials should be encased in a geotextile filter fabric, such as Mirafi 140 N, to prevent migration of fine grained soils into the porous material and related settlement of surface improvements. The contractor should provide documentation that any imported material is free of any environmental contamination which may impact the project or that is regulated by local, state, or federal agencies. This documentation should be provided to Twining and Home Depot prior to importing to the site.

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

8.4 Engineered Fill

8.4.1 The on-site near surface soils encountered are predominantly clays. Native clay soils are not considered suitable for use as fill to a depth of 30 inches below the proposed slab-on-grade. The native clay soils will be suitable for use as fill material at depths in excess of 30 inches below the proposed slab-on-grade provided they are properly moisture conditioned and compacted. The native lean clay soils should be conditioned, i.e., wetted or aerated at necessary to achieve a moisture content of two (2) to five (5) percent above optimum moisture content and compacted to between 90 and 95 percent relative compaction. The upper 6 inches of fill below interior slabs should comprise imported, Class 2 aggregate base. If soils other than those considered in this report are encountered, Twining should be notified to provide alternate recommendations.

8.4.2 Imported, non-engineered fill to a depth of 30 inches below interior floor slabs and 12 inches below exterior slabs should be select non-expansive soils which meet the acceptance criteria in this report.

8.4.3 The onsite soils used for engineered fill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned or air dried to a minimum of 2 percent above optimum moisture content and compacted to a dry density of between 90 and 95 percent of the maximum dry density as determined by ASTM D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density of if soil conditions are not stable.

8.4.4 Recycled materials (such as asphaltic concrete or Portland cement concrete) should not be used within 10 feet of any improvement without approval by the Owner, and Twining. Contractors should not assume that recycled materials can be used in preparing bids for the project without approval by the Owner and Twining.

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

8.4.6 Imported fill soils should be non-contaminated, non-corrosive, non-expansive, granular in nature and contain enough fine grained material (binder) to allow cutting "neat" footing trenches with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	10 - 30

Plasticity Index	Less than 10
Expansion Index (UBC 18-2)	Less than 10
R-Value	Minimum 40
Organics	< 3% by weight
Sulfates	< 0.05 % by weight
Min. Resistivity	> 10,000 ohm-cm

Prior to being transported to the site, the import fill material should be tested and approved by Twining. Prior to importing fill, the contractor shall submit test data that demonstrates that the proposed import complies with the recommended criteria. Twining will test the material after receipt of this information. Also, prior to being transported to the site, the import material shall be approved by the Owner; and certified by the contractor and the supplier (to the satisfaction of the Owner and Twining) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies. This certification should consist of, at a minimum, analytical data specific to the source of the import material. The list of constituents to be tested for the fill source shall be submitted to Twining for review and approval prior to the contractor testing the fill. The contractor shall allow a minimum of seven (7) working days for each import source to be tested. Flyash is not allowed in import, aggregate base, subbase, or other materials used in the construction of the building pad or preparation of the site.

8.4.7 Non-expansive engineered fill (import and/or on-site) soil should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum and three percent above optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.8 Open graded gravels used as engineered fill and/or backfill should be completely encapsulated in an approved geotextile fabric (Mirafi 140N or equivalent), and vibrated and mechanically compacted to a dense, non-yielding condition under the observation of Twining. It is recommended the slurry type cut off collars be constructed at approximately 100 foot intervals along trenches backfilled with open graded material.

8.5 Foundations

8.5.1 Over-excavation and compaction for foundations, soil stabilization, shoring, etc. should be conducted as indicated in this report and the appendices of this report.

8.5.2 Structural loads may be supported on spread or continuous footings placed entirely on at least 2 feet of engineered fill, or engineered fill extending to at least 5 feet below preconstruction site grades, whichever provides the deeper fill. Exterior foundations should be supported at a minimum depth of 36 inches below the lowest adjacent finished grade, but not less than 42 inches below the finished slab surface. Interior footings should have a minimum depth of 30 inches below the finished slab surface. Footings should have a minimum width of 15 inches, regardless of load. Spread and continuous footings may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads.

8.5.3 Combined settlements (static and seismic) of 1 inch total and 0.5 inches differential over a horizontal distance of 50 feet should be anticipated for design consideration.

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 foundation loads.

8.5.5 A structural engineer experienced in foundation design should recommend the thickness, reinforcement, design details, and concrete specifications for the foundations based on the following estimated settlements: 1) a combined total static and seismic settlement of 1 inch, 2) a combined differential static and seismic settlement of ½-inch in 50 linear feet of continuous footings; 3) a differential settlement of ½-inch between new isolated column footings; and 5) a swell of ½ inch in 50 feet.

8.5.6 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. It should be noted that the contractor should take precautions not to allow the exposed soils to dry, including weekends and holidays. Our firm should observe the bottoms and sides of the foundations excavations, and exposed soils to verify that the excavations and exposed soils are properly moisture conditioned, and comply with the requirements of the geotechnical engineering investigation report prior to placement of concrete. If dry soils are noted, the contractor should request written recommendations from our firm to properly moisture condition the foundation excavations. The cost to mitigate the "dry" soils is the responsibility of the contractor.

8.5.7 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 placed entirely on at least 24 inches of engineered fill, the depth of engineered fill required to over-excavate and compact all existing undocumented fill or engineered fill extending to at least 12 inches below subsurface structures to be removed, whichever provides the deeper fill. The horizontal extent of the engineered fill should extend a minimum of 5 feet horizontally beyond both sides of the footings. Spread and continuous footings for miscellaneous structures may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.

8.5.8 The following factors were developed based on the tables in Chapter 16 of the 2001 CBC and the digitized active fault locations published by CGS.

Seismic Factor	CBC Value
Soil Type	S_D
Source Types	B
Near Source Acceleration Factor, Na	1.0
Near Source Velocity Factor, Nv	1.0
Seismic Acceleration Coefficient, Ca	0.44
Seismic Velocity Coefficient, Cv	0.64

8.5.9 Twining should observe the bottom of foundations excavations and slab subgrade prior to the placement of reinforcing steel.

8.6 Cast-In-Drilled-Hole Piers

8.6.1 The total axial load on the shaft should be determined by the project structural engineer. The total axial loads on the shaft should be determined by adding the compressive axial load plus the weight of the concrete shaft (@ 150 pcf) minus the weight of soil displaced by the concrete shaft (@ 100 pcf). A shaft depth should be selected by the structural engineer considering vertical downward and uplift loading.

8.6.2 The allowable downward values presented may be increased by one-third for short duration live loads such as wind and seismic loads.

8.6.3 Additional analysis and evaluations are required to prepare formal recommendations for allowable vertical loads for cast-in-drilled-hole pier foundations. Supplemental investigations and analysis will be performed and recommendations included in a future report. Casing will likely be required during construction due to the loose, near surface granular soils.

8.6.4 The capacity of the cast-in-drilled hole concrete pier in uplift (tension) may be taken as the total dead weight of the pier plus two-thirds (⅔) of the axial skin friction. The allowable uplift capacity of the cast-in-drilled hole concrete piers should be larger than the uplift or tension force on the cast-in-drilled hole concrete piers.

8.6.5 The lateral capacity of the piers may be computed using the procedures outlined in Section 1806 of the 2001 CBC. An allowable lateral soil-bearing pressure, S, of 150 pounds per square foot per foot of depth in granular soils or granular engineered fill can be used based on the soil type and Table 18-I-A of the CBC.

8.6.6 The cast-in-drilled hole concrete piers should be placed no closer together than three pier diameters, center-to-center.

8.6.7 Total and differential static and seismic settlements of 1 inch and ½ inch, respectively, should be anticipated for design of the cast-in-drilled hole concrete piers and grade-beam foundations.

8.6.8 A civil or structural engineer registered in the state of California should design the dimension of the cast-in-drilled hole concrete piers and reinforcement cages to resist shear, moment, and axial (tension and compression) loads.

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

8.6.10 The bottoms of the drilled shafts should be cleaned of all loose soils, cobbles, gravel, or other materials prior to installation of steel. The bottoms of the foundations should be observed by Twining to verify removal of these materials.

8.6.11 The pylon sign may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot per foot of embedment may be used to resist axial loads. Lateral load resistance may be estimated using the CBC non-constrained procedure (Section 1806.8.2.1). A value of 150 pounds per square foot per foot of depth may be used.

8.6.12 At the time of pier construction and until the concrete is placed, the shaft excavation should have stable sidewalls and all sloughed soil should be removed from the bottom of the hole. If the drilled hole exhibits instability, it should be cased. For the purpose of bidding, contractors should assume the excavations will require casing. Twining should observe the excavation to confirm that the pier was constructed as described above, and the soils encountered are similar to those indicated in this report.

8.6.13 Twining should drilling of the excavations foundations and the cleaned excavations prior to the placement of reinforcing steel.

8.7 Cast-In-Drilled-Hole Pier Construction

8.7.1 The contractor chosen to construct the cast-in-drilled hole concrete piers should have a minimum of 5 years of experience in the construction of cast-in-drilled hole concrete piers with a minimum of 5 similar projects (i.e., cast-in-drilled hole concrete piers) in the past 3 years. In addition, the contractor should be experienced in the use of casing to prevent sloughing of loose soils into the excavation.

8.7.2 Temporary casing should be used during drilled pier construction.

8.7.3 Drilling slurry shall not be used.

8.7.4 The type and strength of the concrete used for construction of the cast-in-drilled hole concrete piers should be specified by the structural engineer.

8.7.5 Concrete should be placed in the drilled shaft as soon as possible following drilling. In no case should the excavations be left open longer than eight hours.

8.7.6 Casing should be able to withstand the external pressures of the caving soils. The outside diameter of the casing should not be less than the diameter of the cast-in-drilled hole concrete pier.

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

8.7.8 The rebar cage should be designed (i.e., tied) with adequate space between the bars to allow concrete to flow.

8.7.9 Loose soils should be removed from the drilled shaft excavation prior to placement of reinforcing steel and concrete. The drilled shaft excavation, reinforcing steel, and concrete placement should be inspected by Twining during construction to verify that the soil conditions encountered are consistent with those assumed based on the results of the borings for pier design.

8.7.10 Dewatering techniques should be at the discretion of the contractor. Although not anticipated, the contractor should be prepared to dewater the excavations at any time during construction.

8.7.11 The design slump of the concrete at the time of placement should not be less than four inches. The slump should be specified by the project structural engineer and should be sufficient to flow between the reinforcing steel.

8.7.12 The concrete should be placed in the shaft through a hopper or chute placed centrally over the drilled shaft to direct it clear of the sides and the reinforcement. If this cannot be accomplished, tremie placement should be used. Regardless of the method employed, concrete should not be allowed to freefall a distance greater than 5 feet.

8.7.13 The piers should be placed no closer together than three pile diameters, center-to-center.

8.7.14 Shaft excavation should be drilled within 2 degrees of vertical. This condition should be verified and documented by the contractor.

8.7.15 The rebar cage should be suspended within 2 degrees of vertical in the center of the excavation. This condition should be verified and documented by the contractor. Minimum concrete cover should be maintained throughout the length of the excavation.

8.8 Frictional Coefficient and Earth Pressures

8.8.1 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads (areas of slabs underlain by a synthetic moisture barrier cannot be considered). An ultimate coefficient of friction of 0.3, reduced by an appropriate factor of safety, can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an ultimate coefficient of friction of 0.15, reduced by an appropriate factor of safety, can be used for design.

8.8.2 The ultimate 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 164 pounds per cubic foot. An appropriate factor of safety should be applied.

8.8.3 The passive pressure was calculated based on a minimum soil unit weight of 100 pounds per cubic foot. The soils within the passive zone at the foot of retaining walls (one footing width in front of the wall to a depth equal to the footing depth) should be tested to verify that the soils have the minimum unit weight of 100 pounds per cubic foot (with moisture). If the soils have a unit weight of less than 100 pounds per cubic foot, the soils within this zone should be over-excavated and replaced as engineered fill. These soils should be tested prior to backfilling behind the wall.

8.8.4 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 12 inches of subgrade should be neglected in determining the total passive resistance.

8.8.5 The active and at-rest pressures of the native soils and engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 63 and 80 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure. These values should be considered preliminary pending future investigations and evaluations.

8.8.6 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.

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

8.8.8 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a filter fabric encased gravel section and non-expansive fill to reduce the potential for hydrostatic pressures from acting on the walls. Drainage should be directed either into weep-holes or perforated pipe which can carry drainage from behind the walls.

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

8.8.10 Where stand alone retaining structures provide more than 6 feet of support, or for structures where the exterior grades on opposite sides differ by more than 6 feet, seismic factors or increments may need to be included in the retaining system design. The wall designer should determine if seismic increments should be used or not. If seismic increments are required, contact Twining for recommendations for seismic geotechnical design considerations for the retaining structures.

8.9 Retaining Walls

8.9.1 Retaining wall plans, when available, should be reviewed by Twining to evaluate the actual backfill materials, proposed construction, drainage conditions and other geotechnical design parameters.

8.9.2 Structural loads for retaining walls may be supported on spread or continuous footings placed entirely on at least 2 feet of engineered fil, or engineered fill which extends to a depth of at least 5 feet below preconstruction site grade existing at the time of Twining' field exploration. The engineered fill should extend horizontally a minimum of 5 feet beyond the limits of the foundations. Retaining wall footings may be designed for a maximum net allowable soil bearing pressure of 1,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.9.3 Retaining walls should be constructed with non-expansive granular free-draining backfill placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings. Granular backfill will reduce the effects of shrink and swell on the wall. Granular wall backfill should meet the following requirements:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	10 - 35
Plasticity Index	Less than 10
Expansion Index (UBC 18-2)	Less than 10

8.9.4 The import fill material should be tested and approved as indicated under subsection 8.4 of this report.

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

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

8.9.7 Retaining walls should be designed with a drain system including permeable backfill and drain pipes near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drains 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 and be spaced every 100 lineal feet along the wall, or as determined by the wall designer, whichever is more frequent. Drainage should be directed to the site storm drain system.

8.9.8 If open graded materials such as crushed rock are used as drain material, these materials should be fully encased in filter fabric and compacted to a non-yielding condition under the observation of Twining. 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 (CAL Test 216) using a vibratory plate.

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

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

8.10 Interior Slabs-on-Grade

The slabs on the project that should be prepared as interior slabs include: the floor slab of the Home Depot store, the front sidewalk, the Garden Center slab, sidewalks adjacent to the building, the entrance canopy slab, the lumber off-loading slab, the truck dock slab, customer pick-up porte-cochere, and the pickup lane slab.

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

8.10.2 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade for a differential vertical movement (total and differential settlements and swell) of the floor slabs of ½-inch in 50 feet horizontal distance.

8.10.3 Interior concrete slabs-on-grade should be supported on a minimum of 6 inches of Class 2 aggregate base over at least 24 inches of imported non-expansive soil over the depth of engineered fill recommended below foundations. The minimum 6 inches of AB is recommended directly below the slabs-on-grade to improve the slab support characteristics and for construction purposes. Aggregate base and all non-expansive fill should be compacted to a minimum relative compaction of 95 percent. Concrete should be placed by pump to reduce the potential for creating an unstable subgrade during placement operations.

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

8.10.5 The moisture content of the subgrade or engineered fill below the non-expansive section should be verified to be between two (2) and five (5) percent above optimum moisture content prior to placing non-expansive fill, and also within 48 hours of placement of the vapor retarding membrane or the concrete for the slab-on-grade if a vapor barrier is not used. The moisture content of the subgrade beneath the non-expansive section to a depth of at least 12 inches should be tested and confirmed prior to placement of the non-expansive fill section, vapor retarding membrane or slab-on-grade. If necessary to achieve the recommended moisture content, the clayey subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.10.6 In the event that the earthwork operations for this project are conducted prior to the construction of the individual structures such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) we recommend that the exposed subgrade to receive floor slabs be tested to verify adequate moisture content and compaction. If the moisture content just prior to placement of the floor slab is not at least two (2) percent above optimum moisture content, the soils should be moisture conditioned to at least optimum prior to placing a vapor barrier or concrete. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

8.10.7 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 sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 or equivalent should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The layer of Stegowrap 15 should overlay a minimum of 4 inches of compacted Class 2 AB for the Target store, junior major and smaller retail stores. 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 Mr. Eric Gerst with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the Class 2 AB and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with ASTM C 755-02, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to ASTM E 154-99 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor 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.

8.10.8 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.

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

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

8.10.11 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a water-cement ratio of 0.45 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.

8.10.12 The moisture vapor transmission through the slab should be tested at a frequency and method as specified by the flooring manufacturer. Vapor transmission results should be within floor manufacturers' specifications prior to placing flooring.

8.10.13 To avoid damaging slabs during construction the following recommendations are presented: 1) use perimeter pour-strips at tilt-wall locations to avoid damage to slab-wall connections; 2) design for a differential slab movement of ½ inch relative to interior columns; 3) provide at least 6 inches of aggregate base below the slabs, 4) it is expected that erection of concrete tilt-up wall panels and roof steel may require cranes. The loaded track and/or pad pressure of any crane which will operate on slabs or pavements should be considered in the design of the slabs and evaluated by the contractor prior to loading the slab. If cranes are to be used, the contractor should provide slab loading information to the slab design engineer to determine if the slab is adequate.

8.10.14 A perimeter pour strip between the wall footing and the adjacent interior slab should be incorporated into the project design. After the walls are erected and a majority of the differential movement has occurred, the pour strip should be placed.

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

8.10.16 To provide a design modulus of subgrade reaction of 150 psi/inch, the slabs should be supported on a minimum of 6 inches of Class 2 aggregate base material (R-value of 78). In addition, if concrete trucks will be traveling over the aggregate base material or the aggregate base will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The aggregate base section may be included in the non-expansive engineered fill recommended below the floor slabs. The proposed compacted subgrade can experience instability under high frequency concrete truck loads during slab construction resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. Often 6 inches of AB can reduce the potential for instability under the high frequency loading of concrete trucks. The improved support characteristics of the AB can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design with at least 6 inches of AB for constructability purposes and structural purposes.

8.11 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic. These recommendations are intended for lightly loaded sidewalks, curbs, and planters, etc. 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. Recommendations for concrete slabs subjected to vehicular traffic (impart a load on the subgrade soils of more than 150 pounds per square foot) are included in the Portland Cement Concrete section of this report.

8.11.1 A minimum of 6 inches of Class 2 aggregate base material underlain by a minimum of 6 inches of imported non-expansive engineered fill compacted to 95 percent should be provided below the exterior slabs. The non-expansive fill soil should be underlain by a minimum of 12 inches of moisture conditioned (wetted or aerated), and compacted subgrade. If any city, county, and/or state standards are cited on the plans or specifications, these standards should be in addition to the recommendations in this report.

8.11.2 The moisture content of the subgrade or engineered fill below the non-expansive section should be verified to be between two (2) and five (5) percent above optimum moisture content prior to placing non-expansive fill, and also within 48 hours of placement of the slab-on-grade. If necessary to achieve the recommended moisture content, the clayey subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.11.3 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 foundations and floor slabs. Twining or a qualified geotechnical engineer can provide alternative design recommendations for exterior slabs, if requested.

8.11.4 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 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.

8.12 Asphaltic Concrete (AC) Pavements

8.12.1 The existing pavement sections do not comply with the Home Depot standard or heavy duty designs and have exceeded their projected design life. These pavements are not anticipated to be suitable for the type and frequency of traffic stated in Home Depot's criteria. We understand the existing pavements will be removed and replaced with new pavement sections designed to meet the Home Depot criteria.

8.12.2 Contractors should be aware that areas proposed for pavements and slabs-on-grade adjacent to the proposed building and/or within the overbuild zone should incorporate the more stringent requirements for non-expansive soils and native soil moisture conditioning recommended in the interior slab-on-grade section of this report.

8.12.3 The contractor shall proof roll the subgrade of the areas to receive pavements prior to placement and compaction of the aggregate base (AB). All unstable areas should be removed, stabilized, and replaced with engineered fill under the observation of Twining.

8.12.4 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.

8.12.5 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the State of California Standard Specification Requirements.

8.12.6 The asphaltic-concrete should be compacted to an average relative compaction of 97 percent, with no single test value being below a relative compaction of 95 percent based on a 50-blow Marshall maximum density and a minimum joint density of 95 percent based on a 50-blow Marshall test.

8.12.7 The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. It is recommended that an asphalt concrete mix design(s) be prepared and approved prior to construction.

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

8.12.9 The upper 12 inches of subgrade beneath aggregate base should be excavated, conditioned, i.e., wetted or aerated as necessary to achieve the required moisture content and compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.12.10The following pavement sections are based on a R-value of 5 and a traffic index of 6.5 for the "Standard Duty Pavements," and a traffic index of 7.5 for the "Heavy Duty Pavements."

Traffic Index = 6.5 "Standard Duty Pavements"

ALTERNATIVE	AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	ASB Thickness, inches (Min. R-value = 50)	Compacted Subgrade, inches
Two-layer	3.5	14.5	--	12
Three Layer	3.5	7.5	8.0	12

Traffic Index = 7.5 "Heavy Duty Pavements"

ALTERNATIVE	AC Thickness, inches	AB Thickness, inches (Min. R-value = 78)	ASB Thickness, inches (Min. R-value = 50)	Compacted Subgrade, inches
Two-layer	4.0	17.5	--	12
Three Layer	4.0	8.5	9.5	12

AC	-	Asphaltic Concrete compacted to an average of 97 percent relative compaction
AB	-	Aggregate Base compacted to at least 95 percent relative compaction (ASTM D-1557)
ASB	-	Aggregate Subbase compacted to at least 95 percent relative compaction (ASTM D-1557)
Subgrade	-	Subgrade soils compacted to at least 95 percent with moisture contents within 2 to 5 percent above optimum for expansive (clay) soils and at least 95 percent relative compaction for non-expansive soils

8.12.11 A geotextile fabric of Mirafi 500X, or equivalent, placed below the AB section can extend the life of the pavements. This is a suggestion for Home Depot U.S.A., Inc. to consider and is not intended to become a project requirement unless elected by Home Depot U.S.A., Inc. A geotextile fabric would help prolong the life of the pavements by preventing fine grained subgrade soils from migrating into the AB section.

8.12.12 Alternative pavement sections, such as Portland cement concrete, or equivalent asphaltic concrete sections may be used.

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

8.12.14 It is recommended that the base 2 inch thick course of asphaltic concrete consist of a 3/4 inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation. Mix designs should be provided to Home Depot and Twining for review and approval prior to placement of concrete.

8.13 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. These recommendations should be used for design and construction of the slab, the customer pickup slab, and the seasonal sales area. The PCC pavement design assumes a minimum modulus of rupture of 550 psi. It should be noted that the Home Depot U.S.A., Inc. criterion requires that PCC slabs within the building pad overbuild area (i.e., 10 feet outside the building perimeter or to adjacent curblines, whichever is greater) should be designed as interior floor slabs or PCC pavements, whichever section is thicker or more stringent. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

8.13.1 The "standard duty" pavement section was designed based on an ADTT of 6 trucks per day. A design k-value of 150 psi/in was used considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over the native compacted soils.

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

8.13.2 The "heavy duty" pavement section was designed based on an ADTT of 26 trucks and a k-value of 150 psi/in considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78).

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

8.13.3 The minimum truck dock, per Home Depot U.S.A., Inc., requirements are as follows:

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

8.13.4 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.

8.13.5 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.

8.13.6 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.

8.13.7 Control joints should have a depth of at least one-fourth the slab thickness, e.g., 1-inch for a 4-inch slab.

8.13.8 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas. Construction joint location should be determined by the contractor's equipment and procedures.

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

8.13.10Pavement construction should conform to Sections 40 and 80 of the State of California Standard Specifications.

8.14 Temporary Excavations

8.14.1 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction.

8.14.2 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.

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

8.14.4 In no case should excavations extend below a 2H to 1V zone below existing utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.14.5 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 Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.15 Utility Trenches

8.15.1 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 Twining when these conditions occur and arrange for 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 pipe and backfill of the trench.

8.15.2 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, gas, cable, phone, irrigation, etc.) should be specified by the project Civil Engineer or applicable design professional in compliance with the manufacturer's requirements, governing agency requirements and this report, whichever is more stringent. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321 (a hydraulic condition must be used in the assessment of the type of backfill), whichever is more stringent. The width of the trench should provide sufficient space between the sidewall of the trench and the pipe to allow testing with a nuclear density gage (minimum 12 inches). As a minimum, the pipe bedding should consist of 4 inches of compacted (92 percent relative compaction) ASTM C-33 sand. 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 ASTM C-33 sand that is placed in maximum 6-inch thick lifts 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 non-expansive material compacted to a minimum of 92 percent relative compaction. All materials should be placed at optimum moisture content to 3 percent above optimum moisture content. The project civil engineer should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.15.3 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should extend to at least 1 foot above the top of pipe or as required by the manufacturer, whichever is greater, to prevent damage to the pipe by the compaction operations above the pipe. The pipe bedding, pipe zone and to 1 foot above the top of pipe shall consist of a 2-sack sand-cement slurry. The contractor will be required to arrange for the pipe manufacturer to observe the pipe installation and the completed system to certify, in writing, that the pipe was installed in accordance with the manufacturer's requirements.

8.15.4 Crushed gravel is not allow for use as backfill in trenches. Contractors should assume for the purpose of bid that no rock or gravel can be used for backfill on the project including utility trenches of any kind.

8.15.5 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.

8.15.6 When utility trench backfills are determined by Twining to be nonstructural backfills, they should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D1557.

8.15.7 Trench backfill should be placed in 8 inch lifts, moisture conditioned to within optimum to 3 percent above optimum and compacted to achieve the minimum relative compaction.

8.15.8 On-site soils and approved imported engineered fill may be used as final backfill in trenches.

8.15.9 Jetting of trench backfill is not recommended to compact the backfill soils.

8.15.10Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

8.15.11Storm drains and/or utility lines should be designed to be "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines be inspected by video prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are "watertight."

8.15.12The utility trenches for electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 92 percent per ASTM D-1557. This requirement should be noted on the plans.

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

8.15.14 The project Civil Engineer should include slurry type cutoff collars along utility trenches at critical locations to prevent the migration of surface water into the trench and along the trench backfill material. This is especially critical for utilities constructed on sloped surfaces. At a minimum, slurry cutoff collars should be constructed at maximum intervals of 100 feet along the utilities. The locations of the collars should be determined by the civil engineer and shown on the plans.

8.16 Corrosion Protection

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

8.16.2 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 project. If the manufacturer or supplier 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. Twining does not provide corrosion engineering.

9.0 DESIGN CONSULTATION

9.1 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, however, 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 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 Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that 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 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, we will provide a written summary of our observations, field testing and conclusions 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 Compaction tests should be conducted at a frequency of at least:

Area	Minimum Test Frequency
Mass Fills or Subgrade	1 test per 2,000 square feet per compacted 6-inch lift
Pavement Subgrade	1 test per 5,000 square feet per compacted 6-inch lift
Utility Lines	1 test per 100 feet per 6-inch lift

The above testing frequencies are suggested rates for tests. Testing frequency should be adjusted by the field technician and the engineer as needed based on continuous earthwork observation considering the methods used for compaction and the soil conditions.

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) we recommend that the exposed subgrade to receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

10.5 The construction monitoring is an integral part of this investigation. This phase of the work provides 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 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, Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. We recommend that if a firm other than 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. 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 Twining per the requirements of the Uniform Building Code, Chapter 33, "Excavation and Grading," Section 3318.1, "Final Reports." 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 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, 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 structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.6 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 Twining, its officers and employees harmless from any and all claims and provide 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 in the City of Thousand Oaks. 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 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 Home Depot U.S.A., Inc. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

THE TWINING LABORATORIES, INC.

DRAFT

Read L. Andersen, RCE
Manager
Geotechnical Engineering Division

DRAFT

Harry D. Moore, RCE, RGE
President

RA/amh

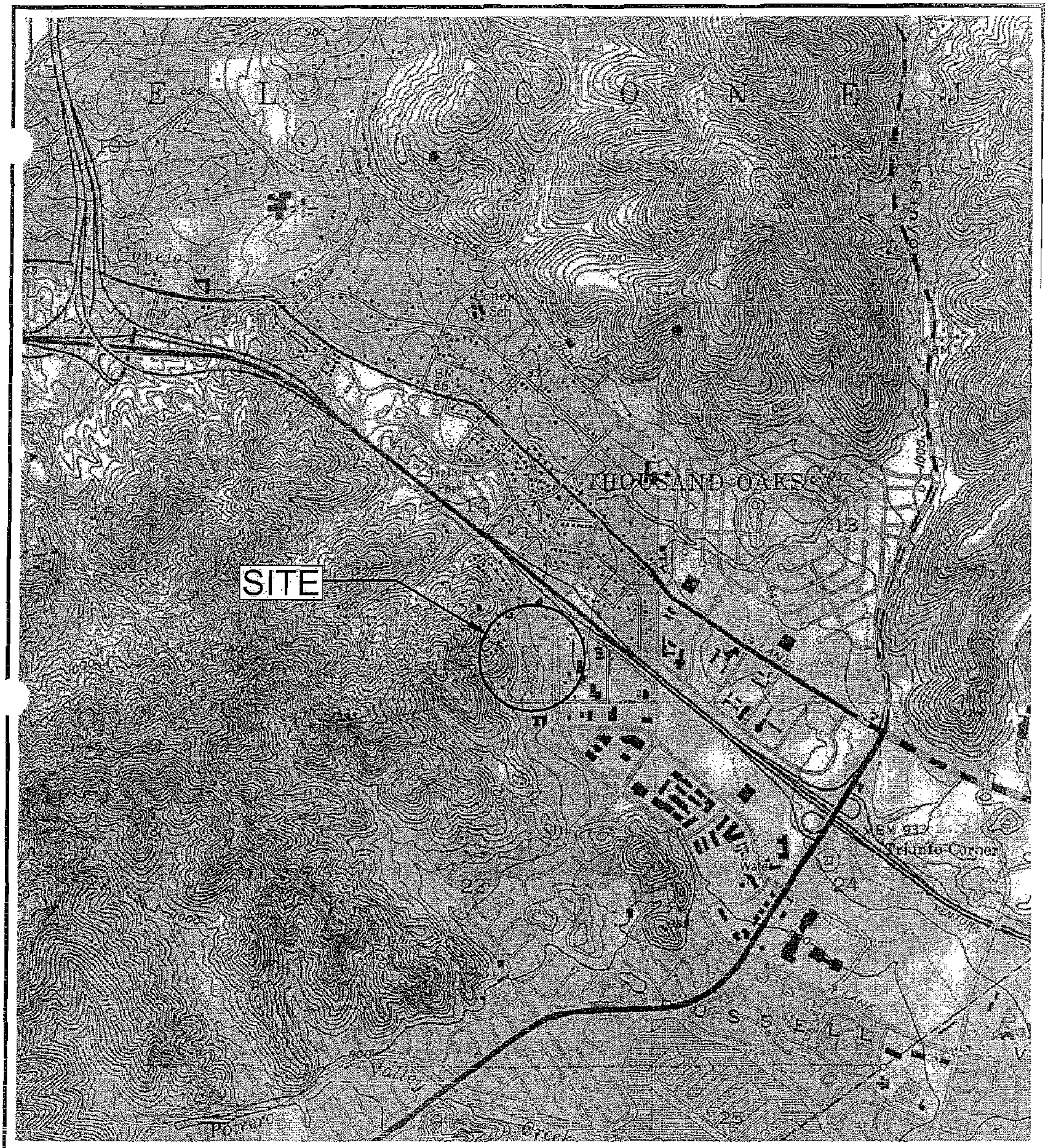
F:\HOME\BarbaraS\D050A31.01HDThousandOaksRA2.wpd

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map

Drawing No. 2 - Test Boring, and R-Value Location Map



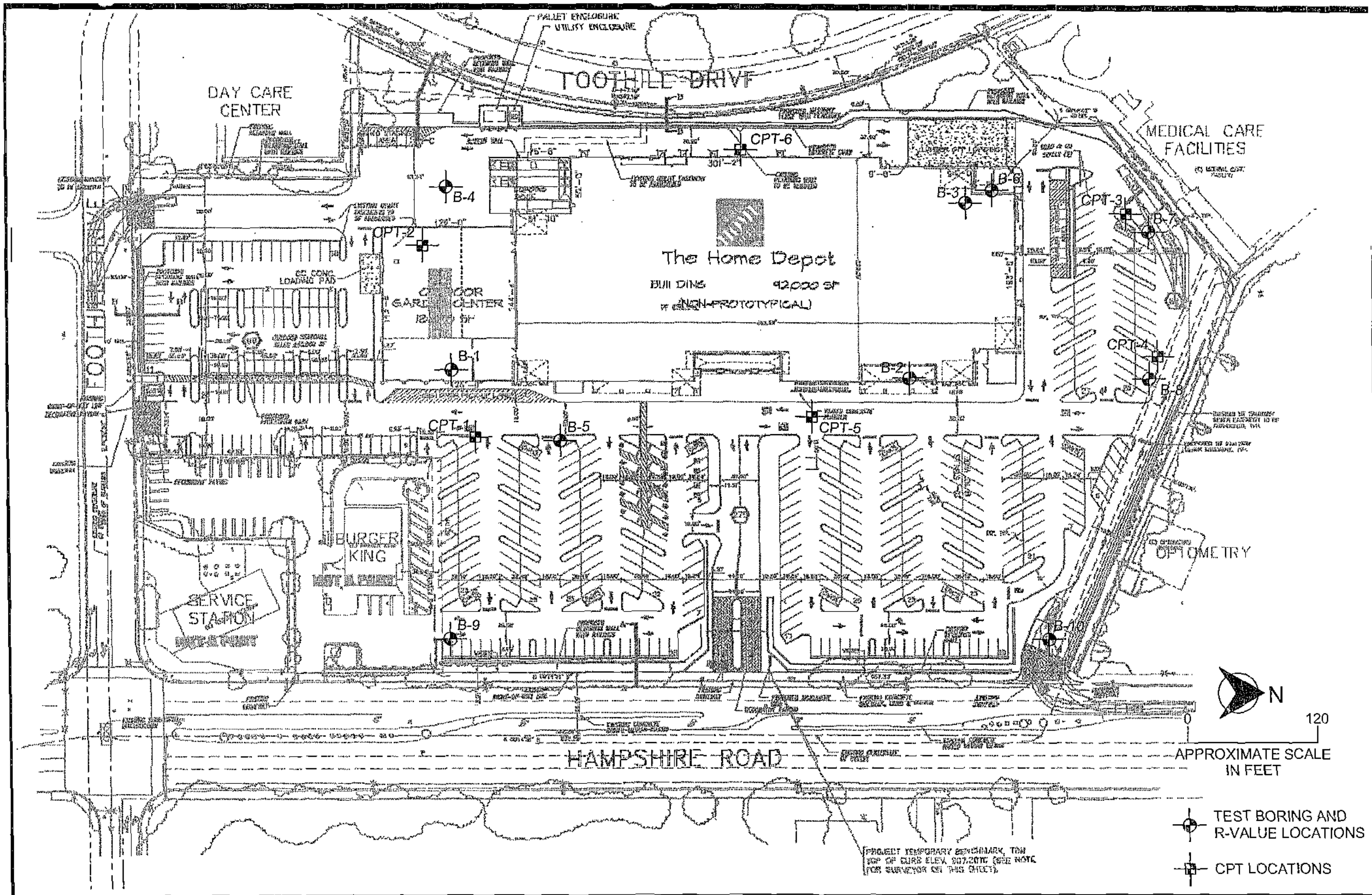
SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
THOUSAND OAKS, CALIFORNIA QUADRANGLE, PHOTOREVISED 1981

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP
PROPOSED HOME DEPOT STORE
325 HAMPSHIRE ROAD
THOUSAND OAKS, CALIFORNIA

FILE NO.: 050A3-01-01	DATE: 06/10/04
DRAWN BY: WME	APPROVED BY:
PROJECT NO. D050A3.01	DRAWING NO. 1

THE TWINING LABORATORIES, INC.
EST. 1898
FRESNO/MODESTO/VISALIA/BAKERSFIELD/MONTEREY



TEST BORING AND R-VALUE LOCATION MAP
PROPOSED HOME DEPOT
THOUSAND OAKS, CALIFORNIA

FILE NO. 050A3-01-02	DATE DRAWN: 09/15/05
DRAWN BY: RM	APPROVED BY:
PROJECT NO. D050A3.01-02	DRAWING NO. 2

TL EST.1898
THE TWINING LABORATORIES, INC.
FRESNO/MODESTO/VISALIA/BAKERSFIELD/MONTEREY

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.

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Home Depot Remodel

Location: Thousand Oaks, CA

Logged By: J. Thatch

Drilled By: Pacific Drilling

Drill Type: Beaver Tri-Pod

Auger Type: 6" O.D. Solid Flight Auger

Project Number: TL D050A3.01

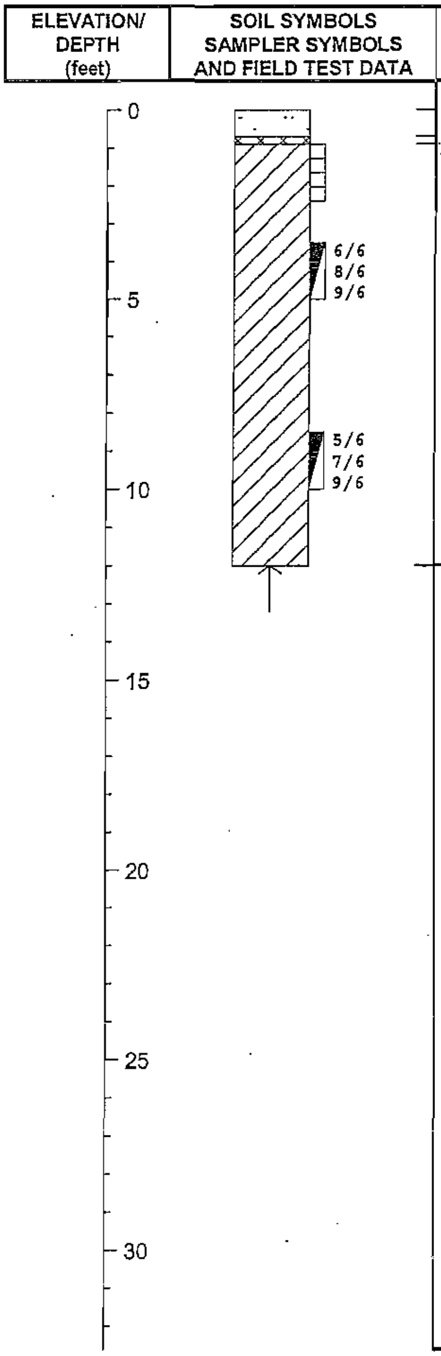
Date: 06/21/04

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: 140 Pound Donut



USCS	Soil Description	Remarks	N-value	Moisture Content %
PCC	Portland Cement Concrete = 7.5 inches			
FILL	At 0.7 Inches - Poorly Graded Sand, damp, fine to coarse, brown, with little gravel and trace silt		--	
CL	At 0.9 Inches - LEAN CLAY; hard, moist, low plasticity, brown to yellowish orange, with trace gravel		17	
	AT 5 Feet - Very stiff, olive gray to dark gray, with trace silt and sand		16	
	At 8.5 Feet - Dark brown, with gravel and cobbles			
	Drill refusal at 12 Feet			

Notes:

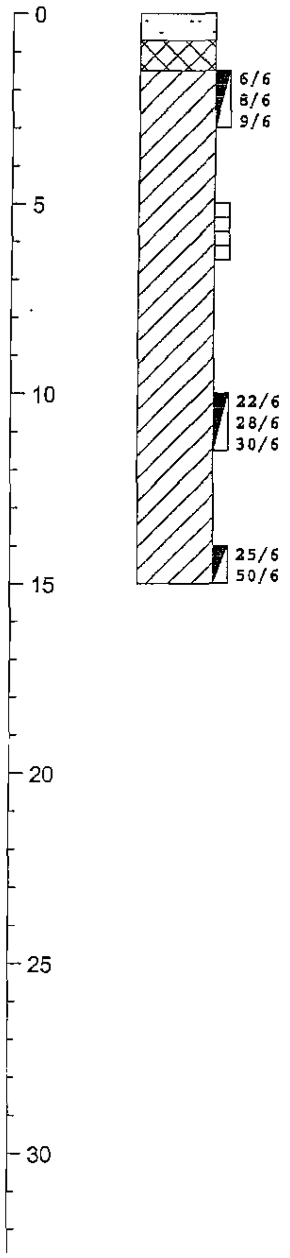
Figure Number B-1

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-2

Project: Home Depot Remodel
Project Number: TL D050A3.01
Location: Thousand Oaks, CA
Date: 06/21/04
Logged By: J. Thatch
Elevation: N/A
Drilled By: Pacific Drilling
Depth to Groundwater: N/E
Drill Type: Beaver Tri-Pod
Cased to Depth: N/A
Auger Type: 6" O.D. Solid Flight Auger
Hammer Type: 140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		PCC	Portland Cement Concrete = 6.5 inches			
		FILL	At 0.7 Inches - Poorly Graded Sand, damp, brown, with gravel and silt			
	6/6 8/6 9/6	CL	At 1.5 Feet - LEAN CLAY; very stiff, moist, low plasticity, brown to yellowish orange, with fine gravel At 3 Feet - Dark brown At 6.5 Feet - Hard, light brown to red		17	
5					-- --	
10	22/6 28/6 30/6				58	
	25/6 50/6		Cemented layer		>100	
15			Bottom of Boring at 15 Feet			
20						
25						
30						

Notes:

Figure Number B-2

SOIL TEST BORING SYMBOLIC LOG

BORING B-3

Project: Home Depot Remodel

Location: Thousand Oaks, CA

Logged By: J. Thatch

Drilled By: Pacific Drilling

Drill Type: Beaver Tri-Pod

Auger Type: 6" O.D. Solid Flight Auger

Project Number: TL D050A3.01

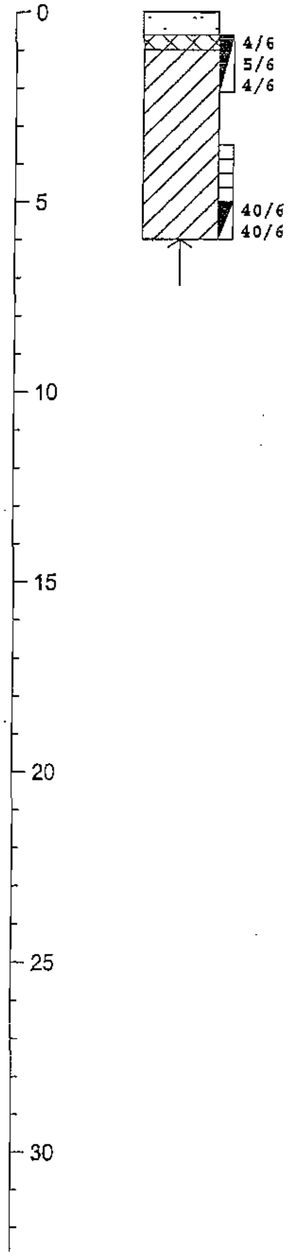
Date: 06/21/04

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: 140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		PCC	Portland Cement Concrete = 6.5 inches			
		FILL	At 0.6 Inches - Poorly graded sand and gravel, no vapor barrier		9	
		CL	At 1 Foot - LEAN CLAY; stiff, damp, low plasticity, brown to reddish brown, with sand and gravel		--	
		CL	At 3.5 Feet - Hard, with fine gravel		>100	
			Bottom of Boring at 6 Feet			

Notes:

Figure Number B-3

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-4

Project: Home Depot Remodel

Location: Thousand Oaks, CA

Logged By: J. Thatch

Drilled By: Pacific Drilling

Drill Type: Beaver Tri-Pod

Auger Type: 6" O.D. Solid Flight Auger

Project Number: TL D050A3.01

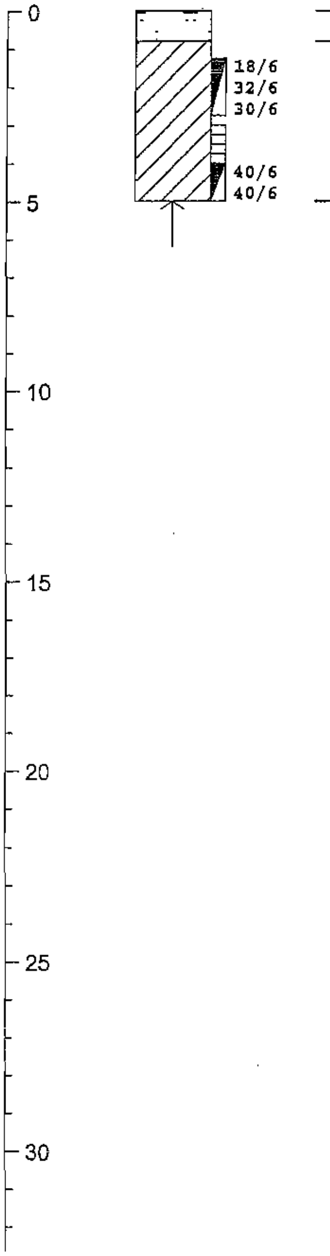
Date: 06/21/04

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: 140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		PCC	Portland Cement Concrete = 6 inches			
		CL	At 0.5 Inches - Poorly Graded Sand, damp, fine to coarse, brown with little gravel		62 –	
			At 0.8 Inches - LEAN CLAY; hard, damp, low plasticity, brown to yellowish orange, trace fine gravels		>100	
			Bottom of Boring at 5 Feet			

Notes:

Figure Number B-4

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-5

Project: Home Depot Remodel

Location: Thousand Oaks, CA

Logged By: D. Ledgerwood

Drilled By: T. Conley

Drill Type: CME 75

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

Project Number: TL D050A3.01

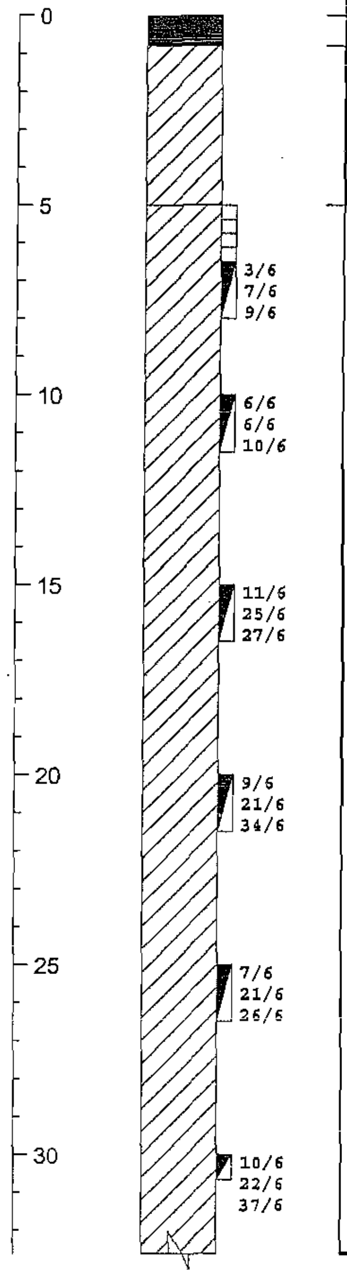
Date: 06/21/04

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: Trip



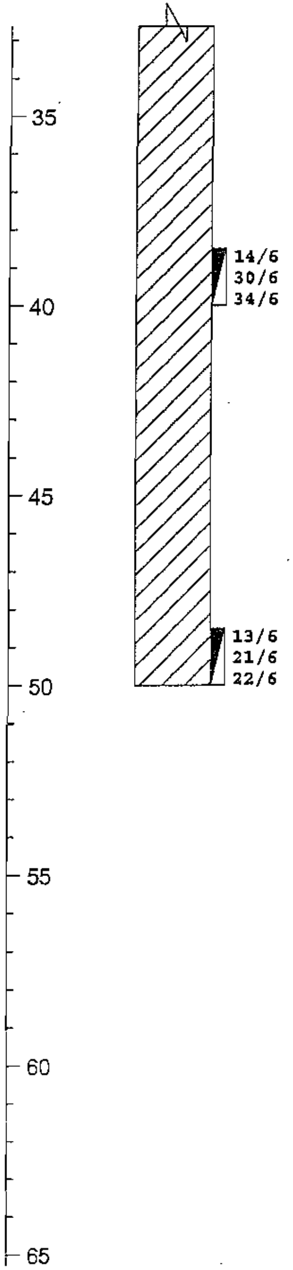
ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		AC	Asphaltic Concrete = 3 inches			
		CL	Aggregate Base = 6 inches LEAN CLAY, Sandy; very stiff, moist, low plasticity, black, with fine to medium rounded to subrounded cobbles and gravel			
5		CL			--	
	3/6 7/6 9/6				16	
			Dark brown, with fine subangular gravel			
10	6/6 6/6 10/6				16	
15	11/6 25/6 27/6		Hard, dry, light gray mottled with light reddish brown, increase in percent sand, no cobbles, with silt		52	
20	9/6 21/6 34/6		Light brown, fine subangular gravel, little to no silt		55	
25	7/6 21/6 26/6		Brown mottled with gray, no gravel		47	
30	10/6 22/6 37/6		Trace fine subangular gravel		59	

Notes:

SOIL TEST BORING SYMBOLIC LOG

BORING B-5

Project: Home Depot Remodel
Project Number: TL D050A3.01
Location: Thousand Oaks, CA
Date: 06/21/04
Logged By: D. Ledgerwood
Elevation: N/A
Drilled By: T. Conley
Depth to Groundwater: N/E
Drill Type: CME 75
Cased to Depth: N/A
Auger Type: 6 5/8" O.D. Hollow Stem Auger
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
	14/6 30/6 34/6		Grayish brown to brown		64	
	13/6 21/6 22/6		Damp, olive brown, trace coarse sand, trace fine gravels		43	
50			Bottom of Boring at 50 Feet			

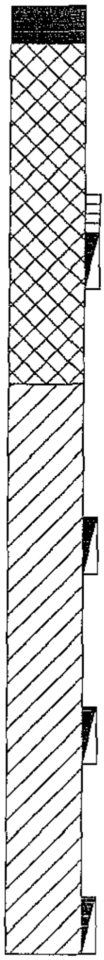
Notes:

SOIL TEST BORING SYMBOLIC LOG

BORING B-6

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		AC	Asphaltic Concrete = 4 inches			
		FILL	Aggregate Base = 7.5 inches			
			SANDY, Silty; very dense, damp, fine to medium subrounded, trace brick debris			
5			Dense, with clay, trace fine to medium subangular gravel		–	
	12/6 21/6 23/6				44	
10		CL	LEAN CLAY, Sandy; hard, damp, low plasticity, light brown			
	9/6 11/6 16/6		Increase in percent sand, with fine to medium subangular gravel		27	
15						
	9/6 11/6 16/6		Very stiff, moist, reddish brown		27	
20						
	6/6 11/6 16/6		Olive brown with reddish brown		27	
25			Bottom of Boring at 25 Feet			
30						

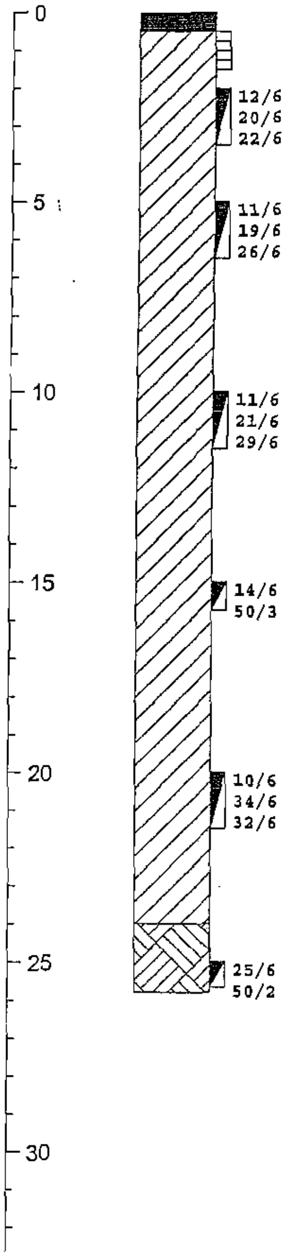
Notes:

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-7

Project: Home Depot Remodel
Project Number: TL D050A3.01
Location: Thousand Oaks, CA
Date: 06/21/04
Logged By: D. Ledgerwood
Elevation: N/A
Drilled By: T. Conley
Depth to Groundwater: N/E
Drill Type: CME 75
Cased to Depth: N/A
Auger Type: 6 5/8" O.D. Hollow Stem Auger
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		AC	Asphaltic Concrete - 3 inches			
		CL	Aggregate Base = 3 inches		--	
	12/6 20/6 22/6		LEAN CLAY, Sandy; hard, damp, low plasticity, reddish brown At 2 Feet - Trace fine subangular gravel		42	
5	11/6 19/6 26/6		Brown mottled with gray		45	
10	11/6 21/6 29/6				50	
15	14/6 50/3		Increase in fine to medium subrounded to subangular gravel		>100	
20	10/6 34/6 32/6		Gray mottled with reddish brown, weak to moderate cementation, decrease in gravel		66	
25	25/6 50/2	ROCK	Siltstone to sandstone, moderately weathered, light gray with light reddish brown		>100	
			Bottom of Boring at 25.8 Feet			
30						

Notes:

Figure Number B-7

THE TWINING LABORATORIES, INC.

SOIL TEST BORING SYMBOLIC LOG

BORING B-8

Project: Home Depot Remodel

Location: Thousand Oaks, CA

Logged By: D. Ledgerwood

Drilled By: T. Conley

Drill Type: CME 75

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

Project Number: TL D050A3.01

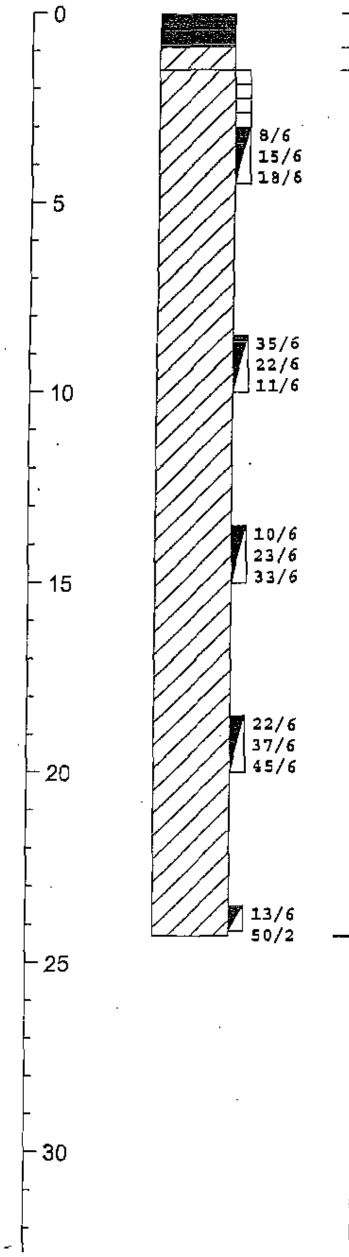
Date: 06/21/04

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		AC	Asphaltic Concrete = 4.5 inches			
		CL	Aggregate Base = 4.5 inches			
		CL	LEAN CLAY, Sandy; hard, damp, low plasticity, brown; with fine to medium subangular gravels with coarse sands		--	
	8/6 15/6 18/6				33	
			Drilling encountered dense coarse gravels			
	35/6 22/6 11/6				33	
	10/6 23/6 33/6		Reddish brown, trace coarse sands		56	
	22/6 37/6 45/6		Increase in fine to coarse angular to subangular gravel		82	
	13/6 50/2				>100	
			Bottom of Boring at 24.3 Feet			

Notes:

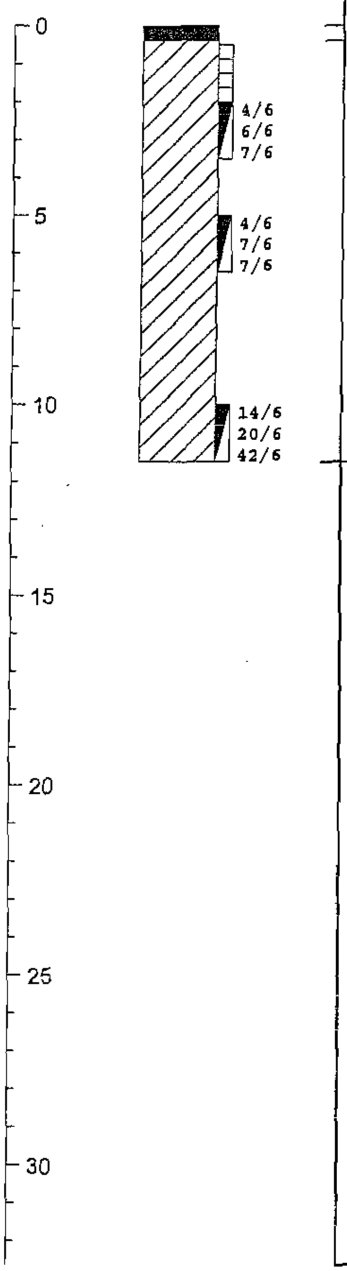
THE TWINING LABORATORIES, INC. EST. 1895

SOIL TEST BORING SYMBOLIC LOG

BORING B-9

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		AC	Asphaltic Concrete = 2 inches			
		CL	Aggregate Base = 2.5 inches		--	
	4/6 6/6 7/6		LEAN CLAY, Sandy; stiff, moist, low plasticity, brown, trace fine subangular gravel		13	
			At 1.5 Feet - Dark grayish green			
	4/6 7/6 7/6		At 2.5 Feet - Dark brown to black, trace fine sands, no gravel		14	
			At 4 Feet - Trace fine subangular gravel			
	14/6 20/6 42/6		Hard, damp, grayish brown to light brown, increase in fine to medium angular gravel, increase in percent sand		62	
			Bottom of Boring at 11.5 Feet			

Notes:

Figure Number B-9

THE TWINING LABORATORIES, INC. EST. 1898

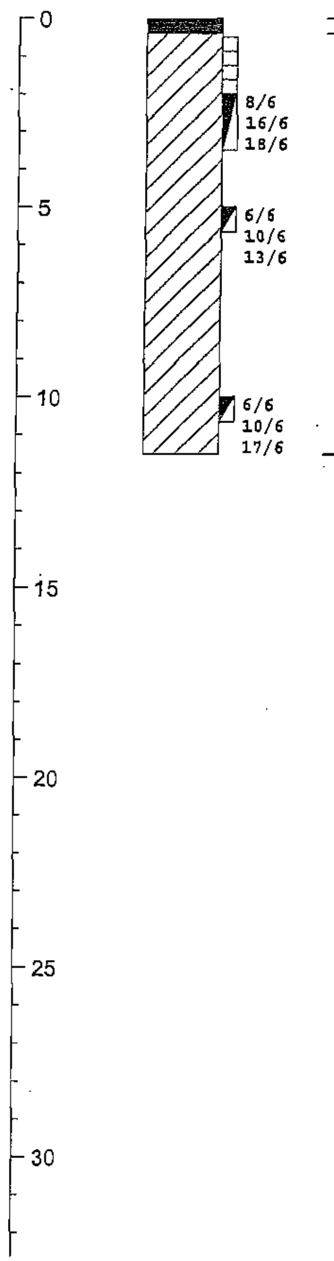
SOIL TEST BORING SYMBOLIC LOG

BORING B-10

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip

ELEVATION/ DEPTH (feet) — SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA



USCS	Soil Description	Remarks	N-value	Moisture Content %
AC	Asphaltic Concrete = 4.5 inches			
CL	LEAN CLAY, hard, moist, low plasticity, brown, trace coarse sand		--	
	At 3 Feet - Light brown, with subangular fine to medium gravel		34	
	At 4 Feet - Very stiff, damp, brown to grayish brown, no gravel		23	
	Sandy, brown, trace fine to medium subangular gravel		27	
	Bottom of Boring at 11.5 Feet			

Notes:

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
△	Portland Cement Concrete
▩	FILL
▨	LEAN CLAY (CL)
■	Asphaltic Concrete
▩	Basalt (or generic rock)
Misc. Symbols	
↑	Drill rejection

Symbol	Description
⌇	Boring continues
Soil Samplers	
☰	California Modified split barrel ring sampler
◩	Standard penetration test

Notes:

1. Test borings B-1 through B-4 were drilled on 06/21/04 using a Beaver Tri-Pod equipped with 140 Pound Hammer. Test borings B-5 through B-10 were drilled on 06/21/04 using a CME 75 equipped with Hollow Stem Auger.

2. Groundwater was not encountered during excavation of the test borings.

3. Test boring locations were located by measuring wheel with reference to the existing site features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

DD = Natural dry density
UC = Unconfined compression (psf)
-4 = Percent passing #4 sieve (%)
-200 = Percent passing #200 sieve (%)
SR = Soil resistivity (ohm-cm)
c = Cohesion (psf)
TS = Field Torvane Shear Strength test (tsf)

LL = Liquid limit (%)
PI = Plasticity index (%)
pH = Soil pH
SS = Soluble sulfates (%)
Cl = Soluble chlorides (%)
ø = Angle of internal friction (degrees)
N/A = Not applicable
N/E = None encountered

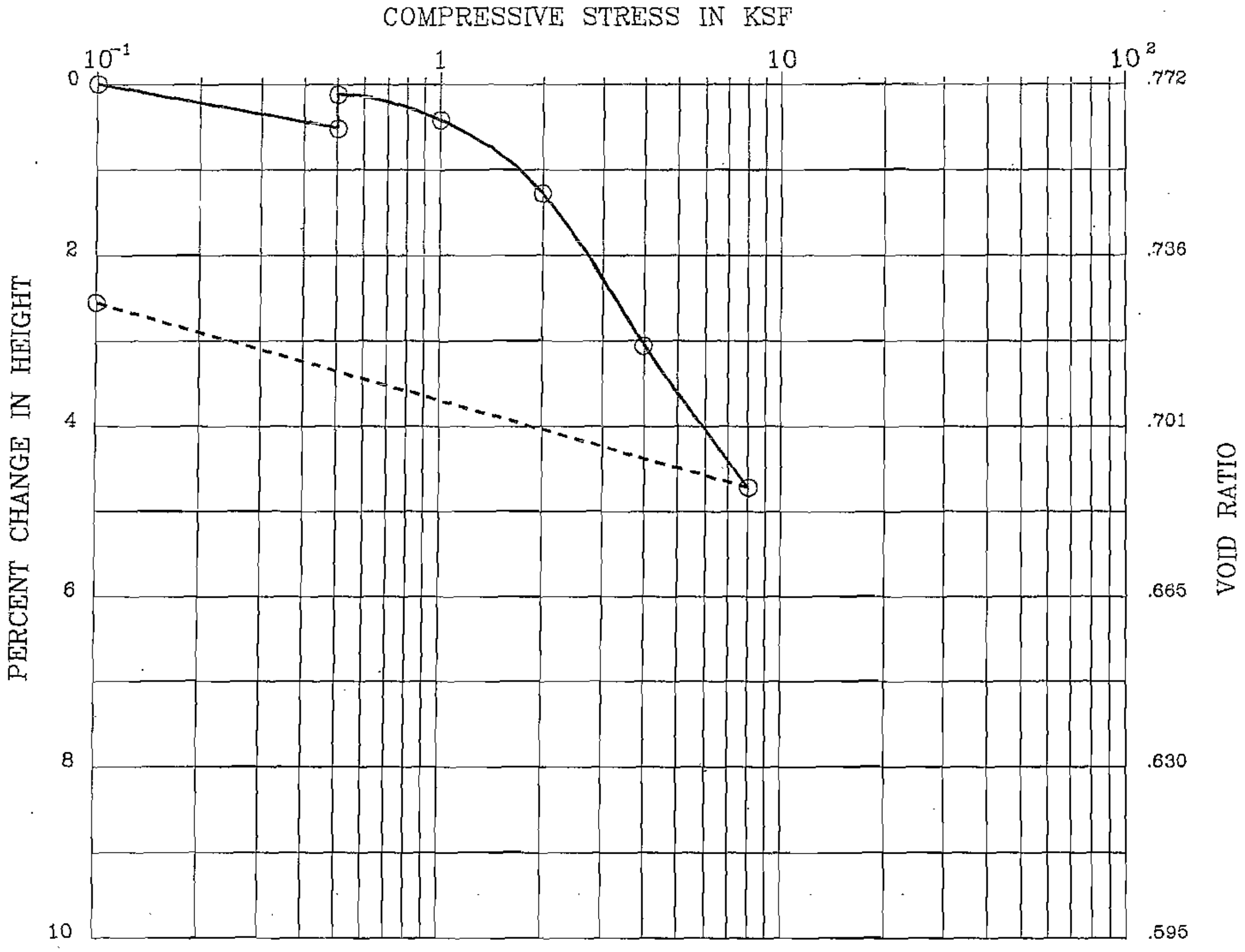
APPENDIX C

RESULTS OF LABORATORY TESTS

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

These Included:	Number of Tests:	To Determine:
Moisture Content (ASTM D2216)	34	Moisture contents representative of field conditions at the time the sample was taken.
Dry Density (ASTM D2216)	108	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Consolidation (ASTM D2435)	3	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
Direct Shear (ASTM D3080)	1	Soil shearing strength under varying loads and/or moisture conditions.
Expansion Index (UBC 18-2)	2	Measure of the swell potential when wetted
Atterberg Limits (ASTM D4318)	3	The consistency and "stickiness," as well as the range of moisture contents within which the material is "workable."
Grain-Size Distribution (ASTM D422)	2	Size and distribution of soil particles, i.e., clay, silt, sand, and gravel.

These Included:	Number of Tests:	To Determine:
R-Value (CTM 301)	5	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Sulfate Content (ASTM D4327)	2	Percentage of water-soluble sulfate as (SO4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	2	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM D1125)	2	The potential of the soil to corrode metal.
pH (ASTM D4972)	2	The acidity or alkalinity of subgrade material.



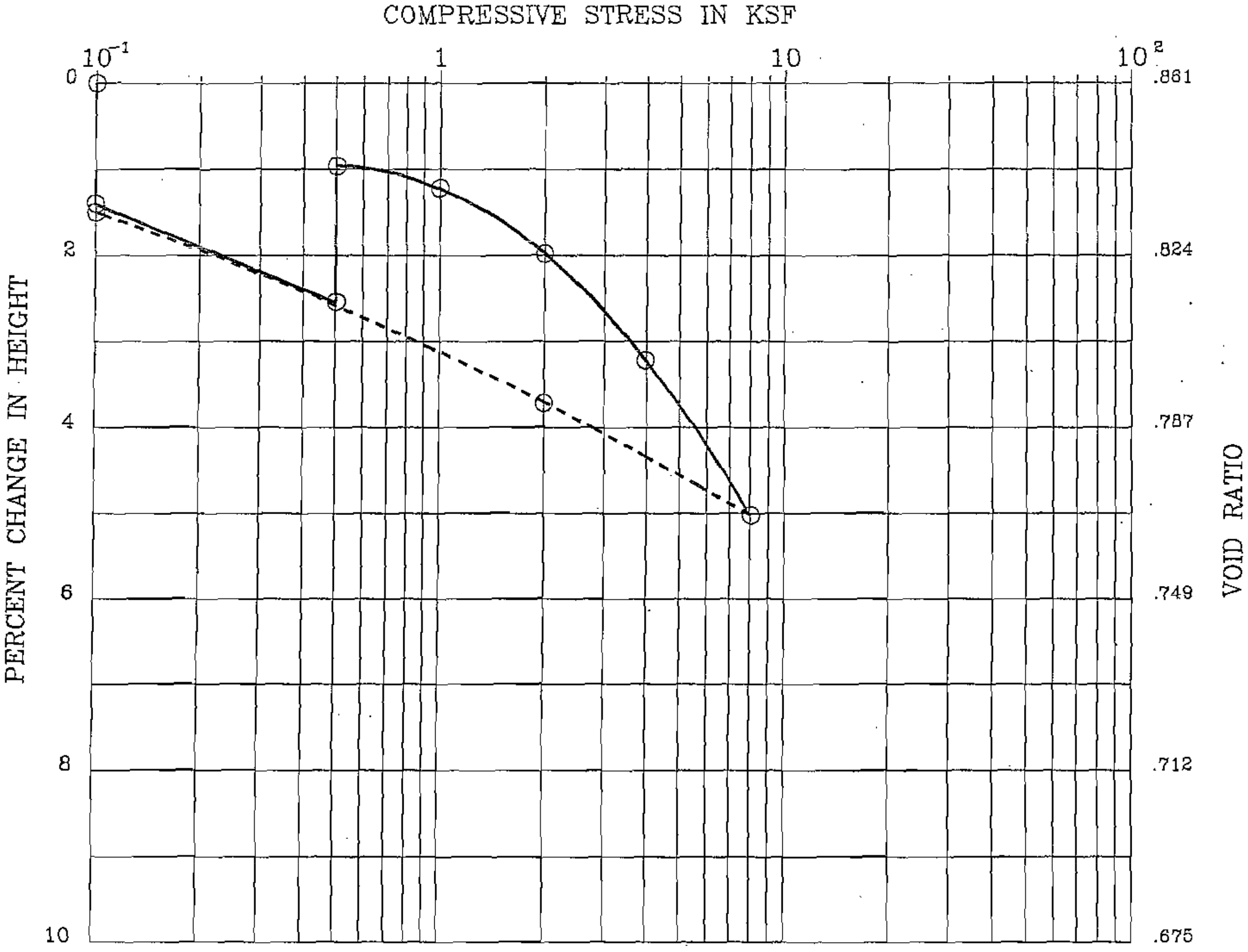
BORING : B-3
DEPTH (ft) : 3.5-5
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	22.0	93.4	76	.772
FINAL	25.5	95.9	94	.726

Remark : Saturated at 0.5 Ksf

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 1



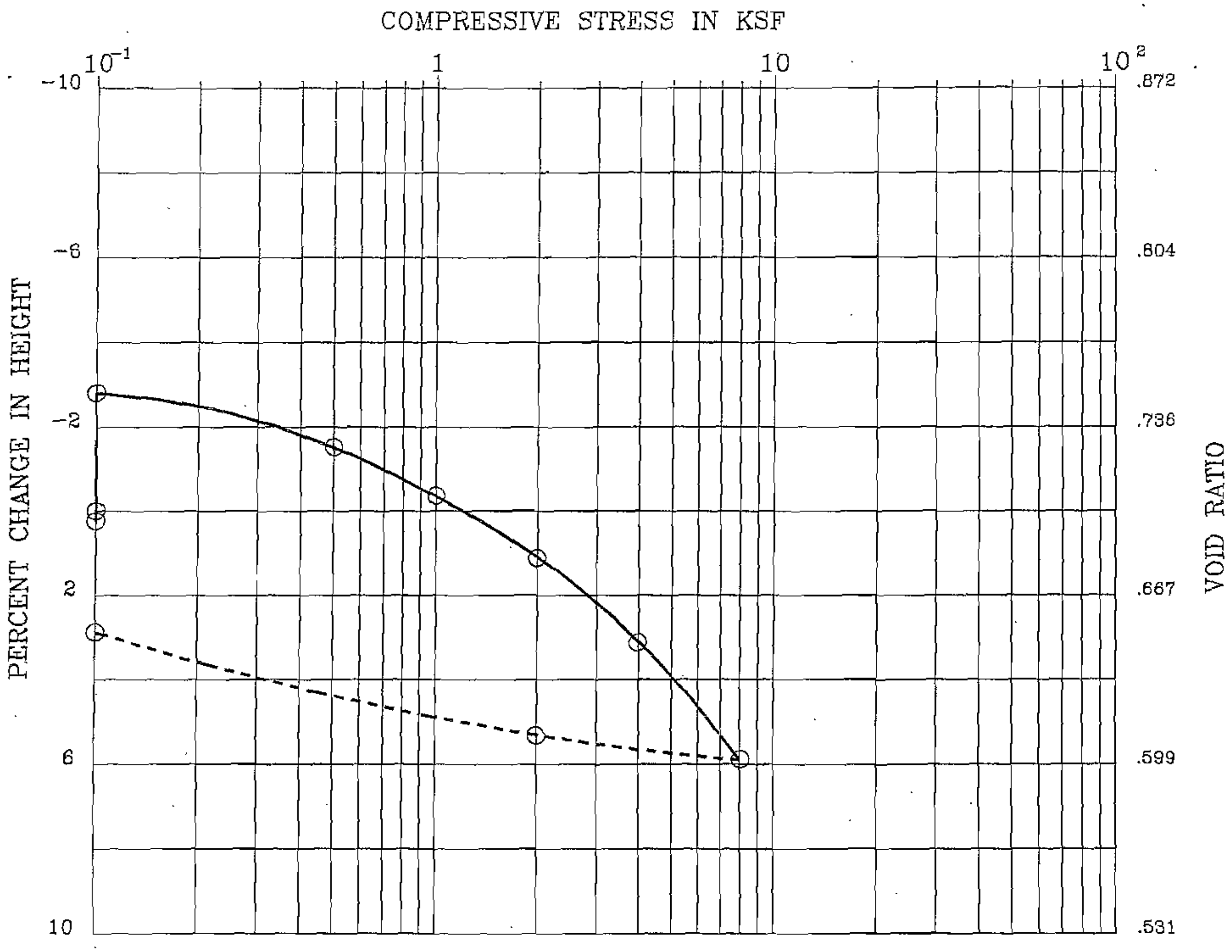
BORING : B-5
DEPTH (ft) : 5-6.5
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	29.2	89.0	90	.861
FINAL	31.3	90.3	100	.833

Remark : Saturated at 0.5ksf

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 2



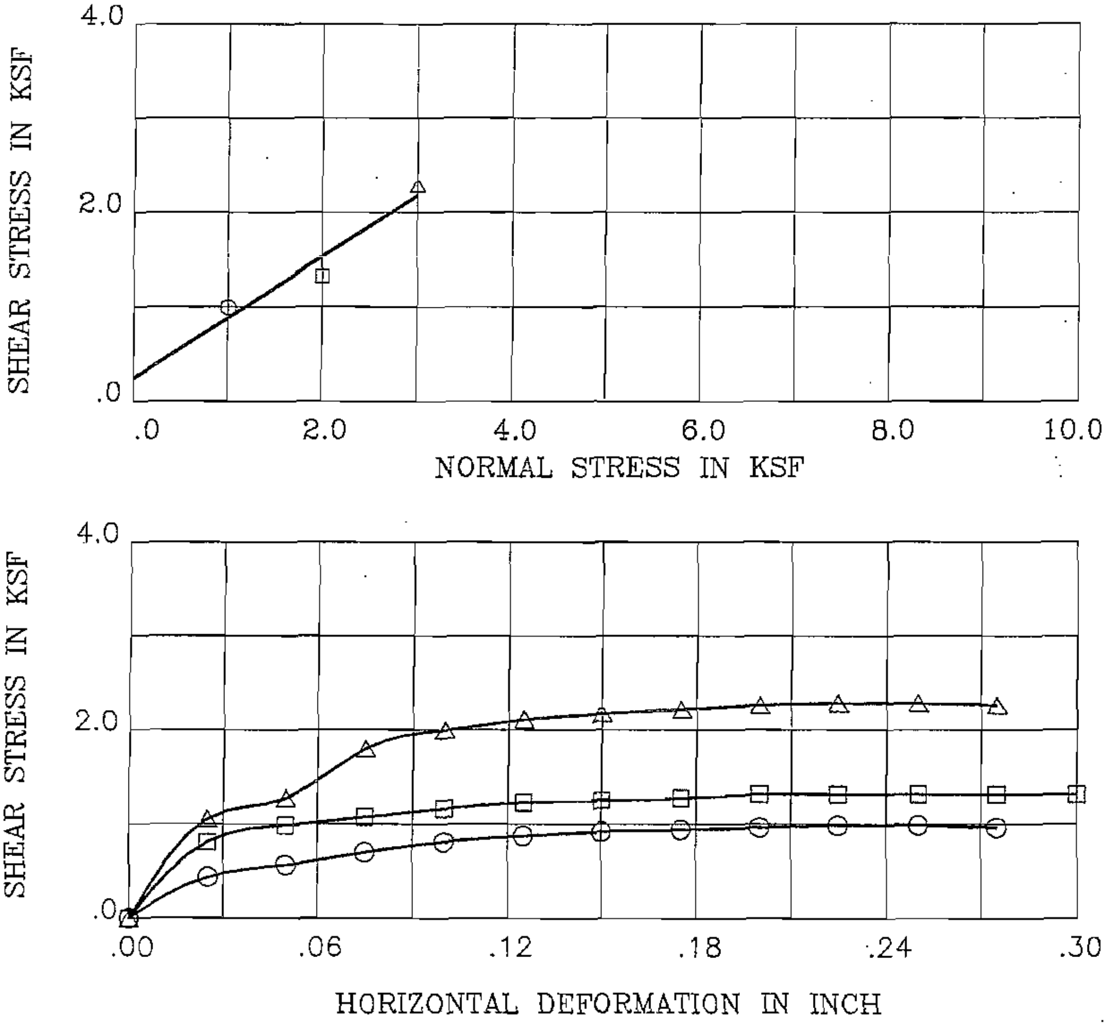
BORING : B-8
DEPTH (ft) : 1.5-3
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	17.9	97.3	68	.702
FINAL	24.0	100.2	98	.652

Remark : Saturated at 0.1ksf

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 3



BORING/SAMPLE : B-8 DEPTH (ft) : 1.5-3

DESCRIPTION :

STRENGTH INTERCEPT (C) : .238 KSF

FRICTION ANGLE (PHI) : 32.9 DEG

(PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	29.3	105.5	.567	1.00	.99	.97
□	28.9	107.0	.546	2.00	1.32	1.32
△	26.9	104.8	.579	3.00	2.29	2.26

Remark :

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	DIRECT SHEAR TEST Figure No. 4

COMPACTION TEST REPORT

Project No.: D050A3.01 **Date:** 7-6-04
Project: Home Depot Remodel

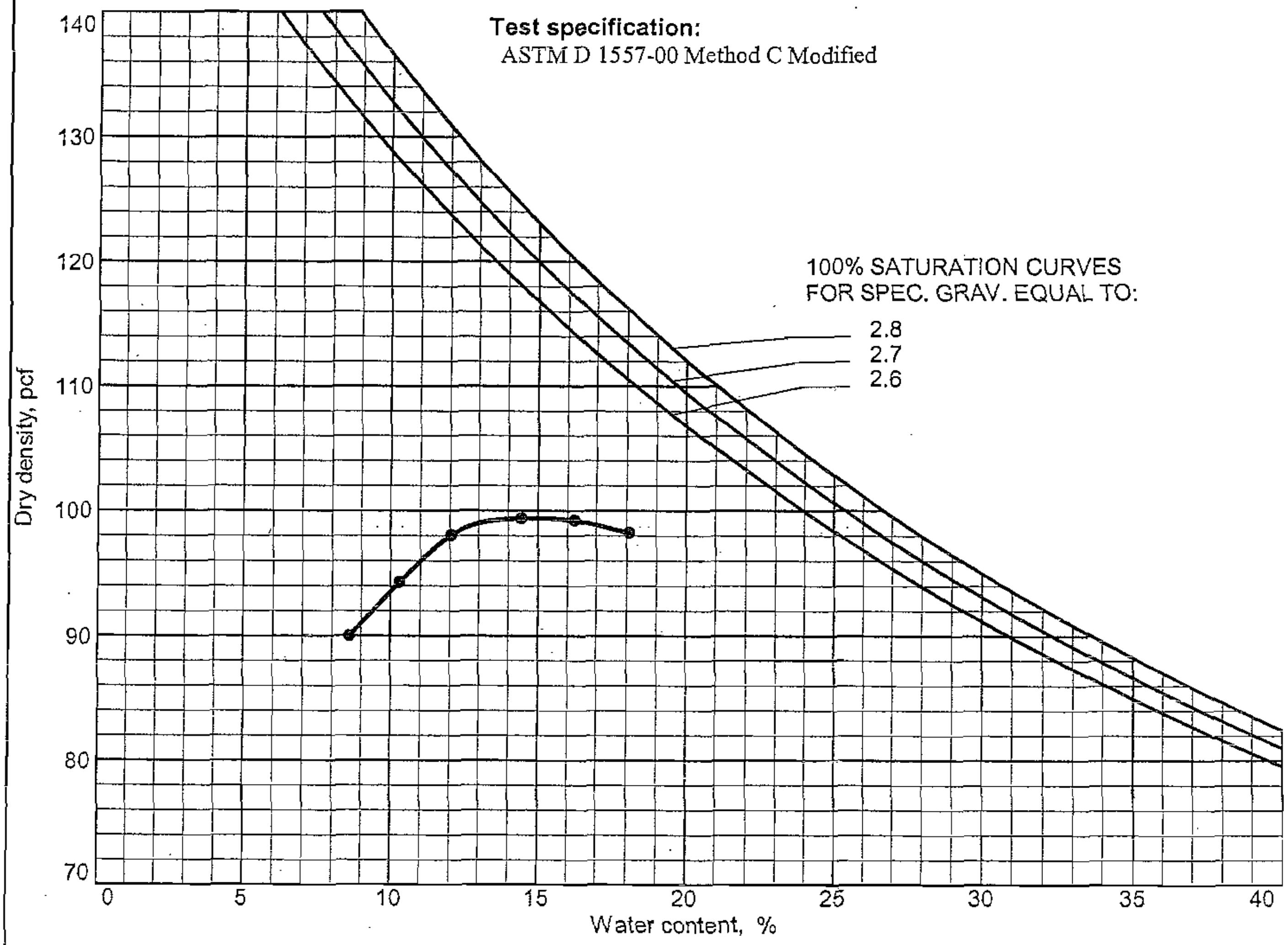
Location:
Elev./Depth: 0.7 **Sample No.**
Remarks: B-2, 0.7"-1.5'

MATERIAL DESCRIPTION

Description: At 0.7 Inches - Poorly Graded Sand, damp, brown, with gravel and silt

Classifications - **USCS:** FILL **AASHTO:**
Nat. Moist. = **Sp.G. =** 2.65
Liquid Limit = **Plasticity Index =**
% > 3/4 in. = % **% < No.200 =**

TEST RESULTS
Maximum dry density = 99.4 pcf
Optimum moisture = 15.0 %



Test specification:
ASTM D 1557-00 Method C Modified

Figure 5

THE TWINING LABORATORIES, INC.

EXPANSION INDEX TEST

Uniform Building Code (UBC) 29-2

Project Number: D050A3.01

Project: Home Depot(Thousand Oaks)

Sample Location: B-8

Depth: 1-3'

Date:07-5-04

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
B-8	13.6	28.5	98.6

Initial Thickness: 1.0000

Final Thickness: 1.0670

Expansion Index (EI): 67

Expansion Soil Classification: Medium

TABLE NUMBER 29-C
EXPANSIVE SOIL CLASSIFICATION

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

Figure No. 6

EXPANSION INDEX TEST

Uniform Building Code (UBC) 29-2

Project Number: D050A3.01

Project: Home Depot Thousand Oaks

Sample Location: B-2

Depth: 0.7"-1.5'

Date: 7-4-04

Sampled by:

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
B-2	11.9	21.9	101.9

Initial Thickness: 1.0000

Final Thickness: 1.0238

Expansion Index (EI): 23.8

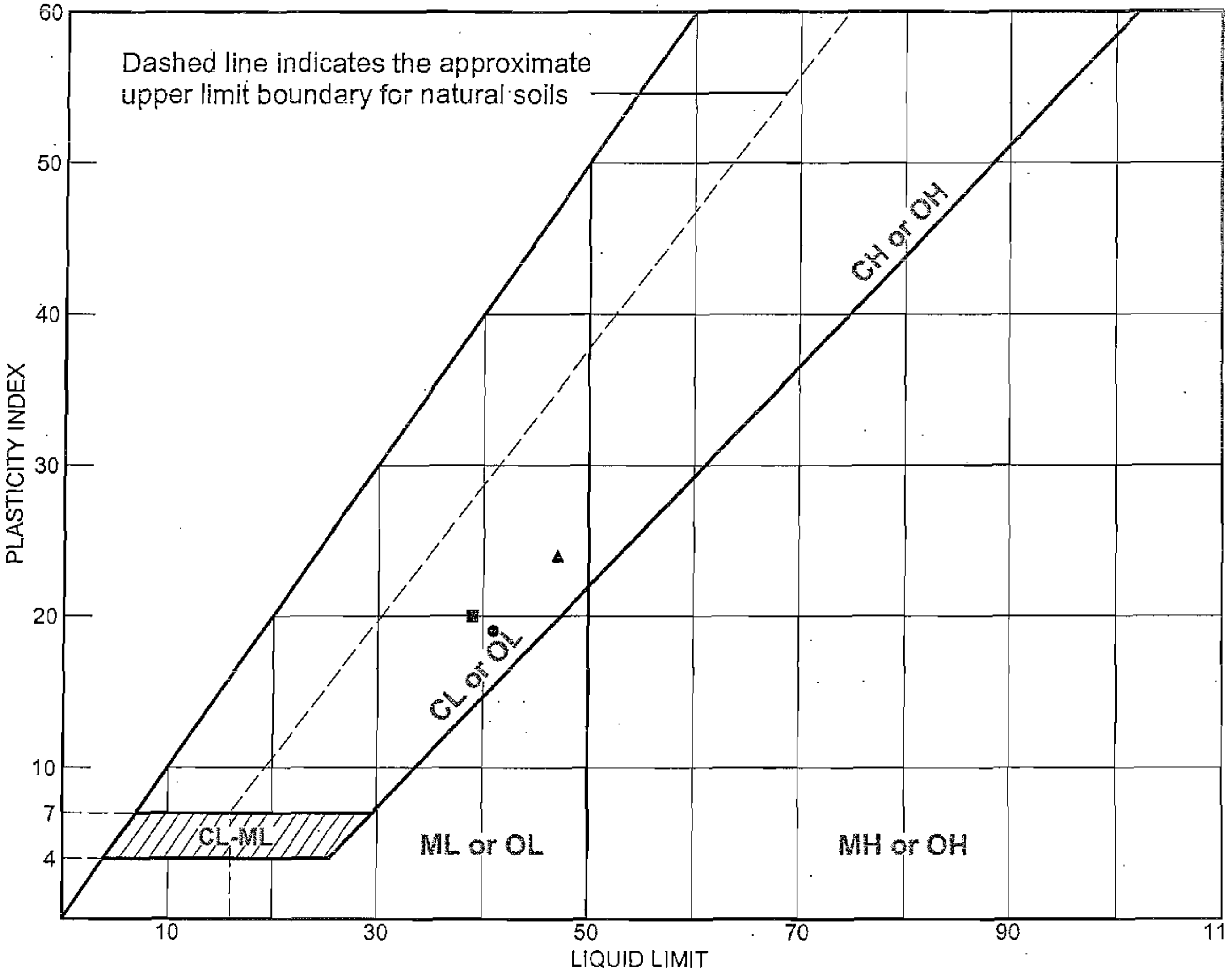
Expansion Soil Classification: Low

TABLE NUMBER 29-C
EXPANSIVE SOIL CLASSIFICATION

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

Figure No. 7

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-3		3.5		22	41	19	CL
■	B-5		5		19	39	20	CL
▲	B-8		1.5		23	47	24	CL

LIQUID AND PLASTIC LIMITS TEST REPORT

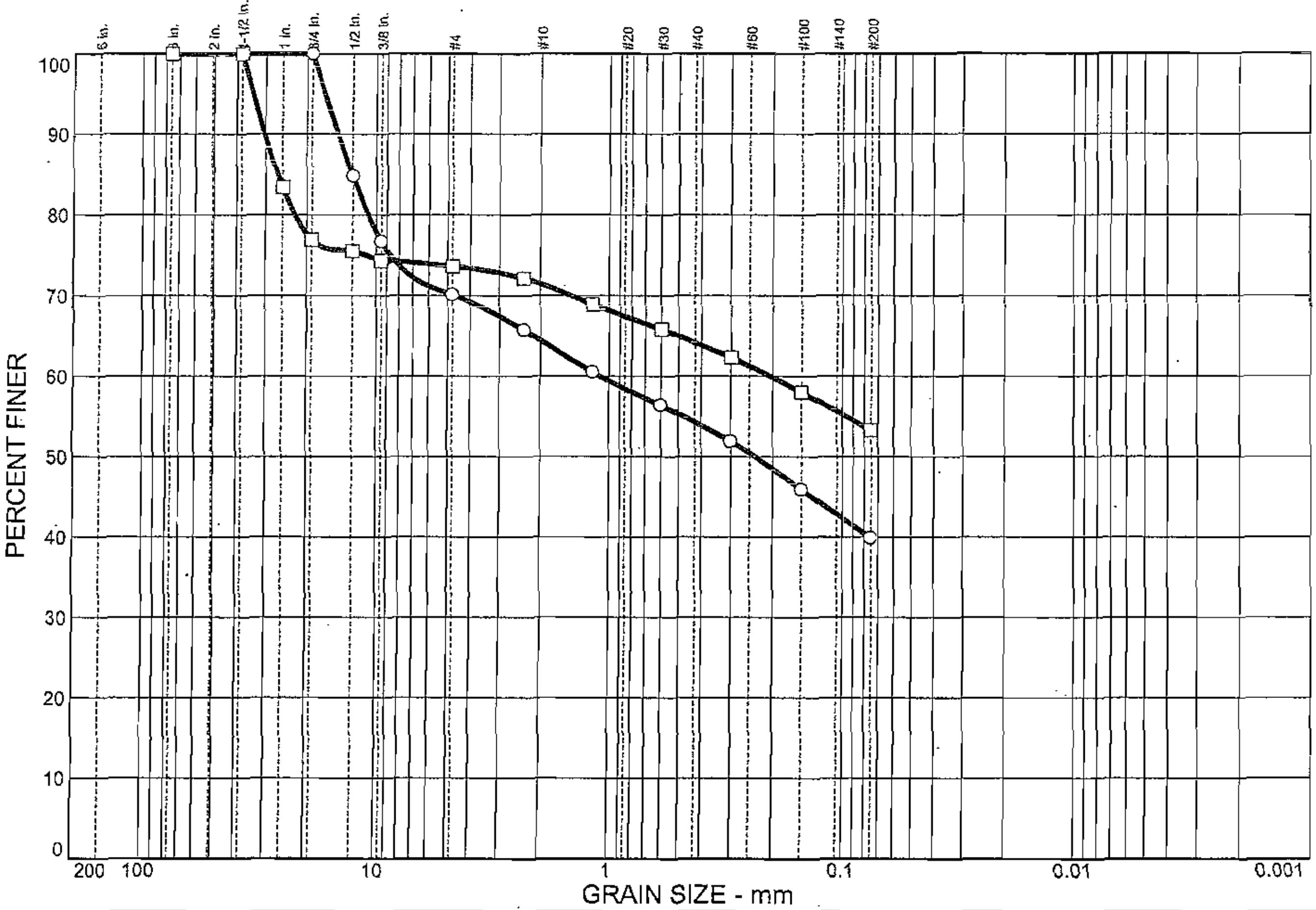
THE TWINING LABORATORIES, INC.

Client:

Project: Home Depot Remodel

Project No.: D050A3.01

Figure No. 8



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	29.9	5.7	10.1	14.4	39.9	
□	0.0	23.1	3.3	2.2	7.3	10.8	53.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○	B-3		3.5	At 3.5 Feet - Hard, with fine gravel	SM
□	B-5		5		CL

THE TWINING LABORATORIES, INC.

Client:

Project: Home Depot Remodel

Project No.: D050A3.01 **Figure No.** 9

Twining Geotechnical Department 2527 Fresno Street	Project: Home Depot- Thousand Oaks Project Number: D050A3.01	Reported:
Fresno CA, 93721	Project Manager: Vasiliy Parfenov	07/06/04

B-7 .5-2

4F29004-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method
Inorganics							
Chloride	**14**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D-4327-84
Chloride	**0.0014**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
Sulfate as SO4	**0.0028**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
pH	**7.7**		pH Units	T4G0102	07/01/04	07/01/04	ATSM D4972-89 Mod
Resistivity	**6200**		ohms/cm	T4G0102	07/01/04	07/01/04	ASTM D1125-82
Sulfate as SO4	**28**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D4327-84

THE
TWINING
LABORATORIES, INC.
EST. 1898
CALIFORNIA ELAP CERTIFICATE #1371

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

Twining Geotechnical Department
2527 Fresno Street

Fresno CA, 93721

Project: Home Depot- Thousand Oaks
Project Number: D050A3.01

Project Manager: Vasiliy Parfenov

Reported:
07/06/04

B-2 0.7-1.5

4F29004-02 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method
Inorganics							
Chloride	**26**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D-4327-84
Chloride	**0.0026**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
Sulfate as SO4	**0.0079**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
pH	**6.9**		pH Units	T4G0102	07/01/04	07/01/04	ATSM D4972-89 Mod
Resistivity	**6200**		ohms/cm	T4G0102	07/01/04	07/01/04	ASTM D1125-82
Sulfate as SO4	**79**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D4327-84

Notes and Definitions

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

Quality Control Data Available Upon Request

APPENDIX D
CHEMICAL TREATMENT OF SOIL

No change orders will be allowed for wet weather conditions, wet soil, soil instability, etc. including chemical treatment, geotextile fabric, rock, soil import, etc.

Contractors's bids shall include chemical treating of the site soils to a minimum depth of 12 inches below the bottom of the excavation for the building pad. Bids should include chemical treating of the site soils to a depth of 12 inches below the bottom of the excavation for building pad and overbuild zone.

Chemical soil treatment should include a minimum of 5 percent high calcium quick lime and/or cement or as required to stabilize the bottom of the excavation. The base bid shall include either 5 percent high calcium quick lime or 5 percent Portland cement. Twining will select the type of chemical, i.e., cement or lime, based on the type of soil encountered at the base of the excavations. Twining should be contacted to observe, sample, and test the soil at the bottom of the excavation to assess the relative suitability of lime or cement treatment.

This recommendation is based on the current soil conditions and may require revision if the on-site soil conditions change significantly.

Subgrade Preparation: Prior to addition of any lime to the subgrade soils, the proposed subgrade soils shall be free of organics and deleterious material. The presence of organic material in the lime treated soil may adversely affect the impact of the lime treating process and therefore provide less than the desired results. The subgrade soils should be graded to the proposed elevations. If necessary, the proposed subgrade soils should be scarified to a minimum depth of ten (10) to twelve (12) inches.

Application: As previously discussed, it is anticipated that the current soil conditions will require a minimum concentration of five (5) percent (by dry weight) of either lime or cement depending on the type of soil encountered. The actual weight of high calcium quicklime or cement may vary slightly depending on the actual weight of the quicklime or cement and the soil placed in the area to be treated. The anticipated weight of quicklime or cement should be determined prior to application in order to verify application rate and concentration. It is recommended that the spread rate and application rate of the lime or cement be verified both prior to and during application. This can be accomplished using a tarp or temporary hard surface placed beneath the application equipment.

Mixing: Immediately after the lime or cement has been applied, the lime or cement shall be thoroughly mixed into the subgrade soils. The soil shall be mixed to a depth of at least twelve (12) inches. It is recommended that an additional two (2) to four (4) inches of soil be lime treated to allow for trimming of disturbed soils in the event the surficial soils are disturbed by traffic or weather. The lime shall be mixed into the subgrade soils with a minimum number of two (2) passes to ensure a thorough and uniform blending of the lime. Mixing should be performed in two (2) directions (i.e., perpendicular to one another) across the treated subgrade. All of the applied lime shall be mixed into the proposed subgrade soils to the full depth of treatment. Mixing of the lime or cement and soil should result in a homogeneous mixture, free of and lumps or clods in excess of two (2) inches across the largest diameter.

Water shall be added to the lime/soil or cement/soil mixture as necessary to maintain and ensure that the resulting mixture possesses a minimum of the optimum moisture content as determined by the geotechnical engineer at the time of lime or cement treatment. Water shall be gradually added to the mixture and thoroughly blended. The resulting moisture content of the lime/soil or cement/soil mixture shall be between one (1) to three (3) percent above the optimum moisture content.

Mixing of the subgrade soils should be performed to the satisfaction of the geotechnical engineer. Subsequent to the mixing process, the resulting lime or cement treated subgrade soils shall be regraded to the approximate proposed elevations. The proposed subgrade should be lightly compacted to decrease the potential for evaporation loss prior to final compaction. In addition, the proposed subgrade should be graded to ensure adequate surface runoff and avoid standing water.

Preliminary Curing: Subsequent to completion of the initial mixing process, the lime or cement treated soil shall be allowed to cure for one (1) to four (4) days. The treated soil should not be allowed to cure for more than four (4) days prior to the final mixing process. During the preliminary curing, the moisture content in the treated soil should be maintained in order to avoid cracking or dessication.

Final Mixing and Pulverizing: As previously discussed, the lime treated soils should not be allowed to cure in excess of four (4) days. Subsequent to the preliminary curing period, the treated subgrade soils once again be thoroughly mixed and pulverized to the full depth of treatment. The final mixing and pulverizing process should continue until the treated material entirely passes a one-half inch sieve, and a minimum of eighty percent of the treated soil passes a No. 4 sieve. Additional water should be gradually and uniformly added at this point in order to insure the resulting treated soil possesses at least the optimum moisture content.

Compaction and Finishing: Compaction of the treated soil should be performed immediately following the final mixing process. The full depth of the treated soil shall be compacted to a minimum of 95 percent of the maximum dry density. It is recommended that the entire surface area of the treated soils be rolled using a rubber tired roller to ensure uniform and consistent compaction.

Final grading and finishing of the proposed subgrade soils should be performed within 24 hours of final compaction activities. Delayed trimming may result in the need for additional processing and lime treatment. The moisture content in the treated soil should be maintained for a minimum of seven (7) days. The treated subgrade should be protected from standing water and excessive traffic until covered. Prior to placement of fill, aggregate base material, etc., the lime or cement treated subgrade should be proofrolled in order to identify any soft or yielding areas. Soft or yielding areas shall be repaired prior to backfilling.

Quality Control: It is recommended that the contractor submit a quality control plan prior to commencement of the lime or cement treatment process. This plan should include but not necessarily be limited to verification of the spread and application of the lime or cement, verification of the scarification and treatment depth, and verification of the resulting lime or cement percentage in the soil.

Quality Assurance: Subsequent to the final compaction and grading activities, it is recommended that soil samples be collected in order to verify the resulting engineering properties of the lime or cement treated soil. This testing should include R-Value testing, unconfined compressive strength, plasticity index testing, soluble sulfates in soils and determination of the expansive potential of the treated soil. In addition, the depth of treated soil should be verified using phenolthalein.

In addition, the contractor shall provide verification of the unit weight as well as the actual quantity of lime or cement used. Verification should include providing a representative sample of the lime or cement used. Certified load tickets should be provided at the time of delivery in order to verify the actual quantity of lime or cement used at the subject site. The contractor shall also provide a certificate of compliance that the high calcium quicklime or cement and treatment process meet project specifications.

Additional Items to be Considered: It is recommended that underground utility lines be installed prior to the lime or cement treatment process in order to avoid trenching or excavating through the lime treated soil. Trenching or excavation through lime or cement treated soil should be backfilled with aggregate base material with a section equal to the lime or cement treated soil. Any backfill placed in lime o+r cement treated areas consisting of previously excavated soil should be retreated with high calcium quicklime or cement.

It should be noted that lime or cement treatment of the on-site soils will result in an increase in the pH of these soils. The contractor should address any concerns regarding surface water runoff from the treated soils. In addition, lime or cement treatment of soil will hinder plant growth. As a result, areas to be lime or cement treated should be carefully identified in order to avoid treatment of proposed landscaped areas.

The contractor should conform with the requirements of the storm water pollution prevention plan (SWPPP). In addition, the lime or cement treatment process should comply with all local, state, and federal regulatory requirements.

GEOTECHNICAL EVALUATION AND INFILTRATION TESTING, 325 HAMPSHIRE ROAD CITY OF THOUSAND OAKS, CALIFORNIA

Prepared for:

imt Residential
15303 Ventura Boulevard, Suite 200
Sherman Oaks, California 91403

Work Order: 3196-0-0-100
October 18, 2021

GORIAN
& ASSOCIATES, INC.

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Attachments:	Figure 1	Site Location Map
	Figure 2	Regional Geologic Map
	Figure 3	Seismic Hazard Map
	Appendix A	Logs of Subsurface Data
	Appendix B	Laboratory Testing
	Plate 1	Geotechnical Map
	Plates 2 & 3	Geotechnical Cross Sections

GORIAN
& ASSOCIATES, INC.

Applied Earth Sciences
Geotechnical Engineers
Engineering Geologists
DSA Accepted Testing Laboratory
Special Inspection and Materials Testing

3595 Old Conejo Road
Thousand Oaks
California 91320-2122
805 375-9262
805 375-9263 fax

October 18, 2021

imt Residential
15303 Ventura Boulevard, Suite 200
Sherman Oaks, California 91403

Work Order: 3196-0-0-100

Attention: Adam Thomas

Subject: **GEOTECHNICAL SITE EVALUATION AND INFILTRATION TESTING, PROPOSED MIXED-USE DEVELOPMENT, 325 HAMPSHIRE ROAD, THOUSAND OAKS, CALIFORNIA**

1.0 INTRODUCTION

Presented herein is our geotechnical site evaluation and infiltration testing for the proposed mixed-use development at 325 Hampshire Road in Thousand Oaks, California (Site). The approximately 8½± acre overall Site and proposed improvements are shown on the 1" = 40' scale *Preliminary Grading Plan, 325 Hampshire Road, Thousand Oaks, CA* prepared by Stantec, dated April 19, 2021. This plan serves as the base for our attached Geotechnical Map (Plate 1).

Currently, the property (formerly known locally as the K-Mart shopping center) has several unoccupied/vacant buildings, landscape areas, and surface parking and drive areas. Additionally, at the rear of the property, an existing 2- to 22-foot-high retaining wall is present, which will remain. The other existing improvements will be demolished and the new mixed-use project constructed.

2.0 PROPOSED DEVELOPMENT

As shown on the *Preliminary Grading Plan*, and *Site Sections A-A and B-B* by ktgy Architecture + Planning, the proposed mixed-use development will include retail, co-working space, and residential development. Adjacent to Hampshire Road and the main entrance, retail and co-working with residential above and subterranean parking is proposed. Surrounding this area, to the north, south, and west several 3-story residential buildings with at-grade parking garages are planned. Toward the center of the site a swimming pool with adjacent amenity building is located. Driveway, isolated parking, and landscape areas are proposed throughout the Site. The development includes a pocket park at the northern end and a dog park at the southern end.

3.0 GEOTECHNICAL SCOPE OF WORK

The scope of services described below was performed to provide pertinent geotechnical engineering recommendations for the design and construction of the proposed development. This geotechnical evaluation was conducted by or under the direct supervision of a State licensed geotechnical engineer and certified engineering geologist. This evaluation included the following:

Archival Review
Pertinent geologic/geotechnical data in our files was reviewed including regional geologic maps, geotechnical/geologic hazard maps, and Alquist-Priolo earthquake fault-rupture hazard zone maps. In addition, *Draft Geotechnical Engineering Investigation, Proposed Home Depot, 325 Hampshire Road, Thousand Oaks, California* prepared by Twining Laboratories, Inc., (Twining, 2004) and Section 4.4, Geology and Soils presumably from the *Thousand Oaks Home Depot EIR,* were reviewed.

Geologic Mapping
Reconnaissance level geologic mapping of the Site and the existing surficial exposures on the ascending cut slope west of Foothill Drive was performed by a geologist from this office.

Geotechnical Subsurface Exploration
Six infiltration borings (IB-1 through IB-6) and geotechnical boring B-1 were drilled and sampled to total depths ranging from 11 to 51½ feet below the existing ground surface (bgs) as the first phase of our subsurface exploration. A subcontractor supplied and operated truck-mounted hollow-stem auger drill rig equipped with 8-inch diameter augers was used to advance these borings to the total depths and an automatic hammer weighing 140 pounds with a 30-inch drop was used to collect drive samples.

At the conclusion of drilling and sampling, infiltration borings (IB-1, IB-2, IB-4, IB-5, and IB-6) were converted to infiltration test wells. [IB-3, which was to be a deep test encountered groundwater shallow, so the test was not performed.] The wells consisted of placing a minimum 1-foot layer of bentonite in the bottom of the boring and 2-inch diameter Schedule 40 PVC pipe in the boring; the upper 5 to 20 feet of the pipe was non-perforated and the lower 5 to 24 feet of pipe was slotted (0.020”). The annular space between the pipe and the wall of the excavations was backfilled using sand from the top of the basal bentonite layer to the top of the slotted portion of the pipe. Individual well development details are presented on the pertinent boring logs attached in Appendix A. After developing the infiltration test wells, the holes were pre-saturated overnight.

For the second phase of exploration, four geotechnical borings (B-2 through B-5) were advanced to depths of 26 to 46 feet bgs utilizing a subcontractor supplied and operated truck-mounted bucket auger drill rig. Samples from the bucket auger borings were obtained using a telescoping kelly bar weighing 3390 lbs. from 0 to 26 feet and 2260 lbs. from 26 to 53 feet with an approximate 12-inch drop. The borings were observed by an engineer and/or geologist from this office, who logged the underlying materials, and obtained both bulk and relatively undisturbed soil samples for laboratory analyses in order to characterize the subsurface soil and bedrock conditions. In addition, a geologist entered several of the bucket auger borings for detailed down-hole observation and logging of encountered stratigraphic and geologic structural data.

At the conclusion of exploration and logging / infiltration testing, the exploratory borings were backfilled with spoils from the excavations. Some compactive effort to the backfill was applied; however, the backfill may settle over time and the property owner or designated representative should periodically observe the exploration locations and fill any depressions should they develop.

Prior to initially mobilizing the drilling equipment for the field exploration, the proposed exploratory boring locations were located and marked in the field and per State mandated protocol, Underground Service Alert "Dig Alert" contacted.

Laboratory Testing
A program of laboratory testing was performed on selected soil samples obtained during the field exploration operations. The laboratory testing included in-situ moisture and density determinations, shear strength parameters, expansion potential, consolidation characteristics, grain size (Hydrometer),

and maximum dry density/optimum moisture content relationships. In addition, corrosion testing of two selected samples was performed by an independent subcontracted laboratory.

Geotechnical Engineering Analysis and Report Preparation
The results of our archival review, field exploration, and laboratory testing programs were used in engineering analyses to develop geotechnical recommendations for the design and construction of the proposed mixed-use development. Geotechnical findings, conclusions, and recommendations provided in this report include:

a) A description of soil, bedrock, and groundwater conditions, as encountered during the current and referenced subsurface exploration, including Logs of Subsurface Data (Appendix A), a Geotechnical Map (Plate 1), and Geotechnical Cross Sections A-A’ through D-D’ (Plates 2 and 3).

b) A description of the laboratory testing program, including test results (Appendix B).

c) A description of our soil infiltration testing and results.

d) Discussion and geotechnical recommendations regarding:
   i. Seismic setting of the site and seismic design criteria;
   ii. Soil expansion and collapse potential;
   iii. Site preparation, grading, placement of fill and backfill;
   iv. Conventional foundation design and construction, including preliminary settlement analysis;
   v. Retaining wall design parameters;
   vi. Swimming pool design parameters;
   vii. Slab-on-grade and hardscape design;
   viii. Preliminary pavement design; and
   ix. Soil chemistry analysis (by subcontract) and corrosion recommendations.

4.0 BACKGROUND

A geologic/geotechnical site evaluation was conducted by Twining Laboratories, Inc. in 2004 for the proposed redevelopment of the site as a Home Depot (Twining 2004). Twining drilled, sampled, and logged ten (10) borings. Four borings were drilled inside the existing building using a tripod mounted 6” solid flight auger and six were drilled in areas surrounding the building using a truck mounted CME 75 hollow stem auger drill rig equipped with 6-5/8” diameter augers. The interior borings ranged in depth from 5 feet (B-4) to 15 feet (B-2), while the exterior borings ranged in depth from 11½ (B-9 and B-10) to 50 feet (B-5) below the existing ground surface (bgs). All of the borings encountered artificial fill varying in thickness from approximately 6 inches to 1½ feet to 5 feet (B-5). However, in boring B-6 ten feet of artificial fill was encountered, which Twining opined could be related to backfill from a former underground storage tank. The artificial fill was observed to be mantling geologically unassigned soil deposits (that we are herewith classifying as Older Alluvium deposits) that were reported to be at least 50 feet in thickness (B-5). Bedrock of the Modelo Formation (classified by Twining as “rock”) appears to have been encountered at 15 feet (?) (B-2) and at 24 feet (B-7) bgs. In addition, six cone penetrometer test soundings (CPTs) were advanced for the Twining investigation. While the logs of the soundings were not provided, the text of their report reported refusal in CPT-2 and CPT-3 at depths of 16 and 17 feet, respectively, which has been interpreted herein to be bedrock. No groundwater was reported to have been encountered to the maximum depth explored of 50 feet bgs (B-5).

5.0 SITE LOCATION AND DESCRIPTION

The property is located at 325 Hampshire Road in the Thousand Oaks area of Ventura County (Figure 1). The approximately 8½-acre property is bounded by Hampshire Road to the east, Foothill Drive to the south and west, and an assisted living and commercial properties to the north. A preschool is located outside of the southwestern portion of the site, as well as gas stations adjacent the southeastern corner and beyond a medical office northeast of the property. The property is accessed by three driveways from Hampshire Road leading up sloping ground to the existing buildings and one driveway from Foothill Drive.

The western half of the site includes the existing approximately 104,500 square-foot one-story vacant building previously occupied by K-Mart, as well as the smaller attached vacant buildings to the south, which will be demolished. An existing retaining wall with a maximum height of 22½ feet separates the western edge of the property from Foothill Drive above. The wall footing may be encountered as shallow as one foot below the pavement and may extend out from the wall up to 13 feet. The eastern half of the site has surficial parking and drive areas. Surface drainage of the site is accomplished by sheet flow towards Hampshire Road and Foothill Drive.

5.1 SITE GEOLOGY

The site is underlain at depth by bedrock of the Miocene-age Modelo Formation (after Weber 1984) overlain with a thin to relatively thick section of Quaternary-age Older Alluvial sediments and an upper mantle of artificial fill deposits (Figure 2). The areal distribution and spatial relationships of these earth units are shown on the attached Geotechnical Map, Plate 1, and Geotechnical Cross Sections A-A through D-D, Plates 2 and 3. Descriptions of these earth materials are presented below with exploratory excavation specific details presented on the attached Logs of Subsurface Data (Appendix A).

5.1.1 Modelo Formation (Tm)

Sedimentary bedrock of the Miocene-age Modelo Formation underlies the property at depths ranging from 3 feet (B-5) to 24 feet (Twining B-7) to deeper than 51 feet (B-1) below the existing ground surface. [Note that Weber's Modelo Formation nomenclature is equivalent to Monterey Formation of Dibblee and Ehrenspeck, 1993.] As encountered, the bedrock generally consists of light yellowish brown to light olive brown to pale yellow diatomaceous siltstone interbedded with pale yellow silty fine-grained sandstone in a moist condition. The bedrock is typically thinly bedded to laminated and fractured with common iron oxide staining.

Structurally, the bedrock is inclined towards the east-southeast at low angles (4º to 9º) based on the downhole logging conducted in boring B-5.

Geologic field mapping of road cuts along Foothill Drive west of the Site encountered local bedrock outcroppings mantled with relatively thin veneers of artificial fill and topsoil/colluvium (undifferentiated). As observed in the bedrock outcrops, the Modelo Formation consists of thinly bedded to laminated yellowish brown diatomaceous clayey siltstone in a damp, fractured and slightly weathered condition. Structurally, the bedrock observed along Foothill Drive is generally warped with bedding inclined towards the southwest at a moderate angle (18º) at the northern end and then inclined towards the northwest at low to moderate angles (7º to 17º) or towards the northeast to southeast at low angles (6º to 10º) just west of the highest portion of the existing retaining wall on the subject Site. Consequently, it can be surmised that the bedrock underlying the Site near surface and at depth is generally relatively shallow dipping but undulating in different directions.

This office prepared a *Geotechnical Site Investigation* for a 16-unit apartment complex located offsite and west of Foothill Drive, southwest of the subject site (Gorian 1987). Three (3) of the borings (B-1 through B-3) from that site evaluation are shown on the attached Geotechnical Map (Plate 1). The geologic discussion from this report is reiterated herein for completeness. "*As encountered on the lowland portion of the site, in the area of the proposed development, the bedding of the Modelo Formation is inclined to the north to northwest at shallow to moderate inclinations. However, westward in the upper hillside areas and offsite, the structure becomes complexly folded into a series of tight, irregular synclines and anticlines often crenulated within the axial areas*".

5.1.2 Older Alluvium (Qoal)

A thin to relatively thick section of Quaternary-age Older Alluvium overlies the Modelo Formation for the preponderance of the Site. The Older Alluvium was found to extend deeper than 51½ feet based on our recent borings. As encountered, the Older Alluvium generally consists of strong brown to reddish brown to yellowish brown silty clay to clayey silt with varying amounts of sand to gravel and cobbles in a damp to saturated and stiff to hard condition interstratified with strong brown silty fine to medium sand in a damp to saturated and medium dense to very dense condition. The layers of clays, silt, and sand are often gradationally interlayered in rhythmic fine to fining upwards sequences with local concentrations of gravel to cobbles towards the base of the coarser strata. Some calcium carbonate veinlets and iron oxide staining were observed mainly in the finer grained sediments.

5.1.3 Artificial Fill (af)

Artificial fill deposits were encountered mantling the bedrock and Older Alluvium in all of our recent borings, except B-5, and was observed to vary in thickness from 2 feet (B-3) to 10½ feet (B-2 and IB-6). As encountered, the artificial fill generally consists of very dark grayish brown to dark grayish brown to dark yellowish brown to olive brown silty clay mottled with clayey silt and varying amounts of sand, gravel and cobbles in a moist and medium stiff condition.

5.1.4 Groundwater

Groundwater was encountered within the Older Alluvium in borings B-1, B-3, IB-1, and IB-3 at depths ranging from 26 feet (IB-3) to 31 feet (B-1) to 33 feet (IB-1) to 36 feet (B-3) below the existing ground surface. It should be noted that groundwater was encountered at deeper depths during the drilling operation and was allowed to stabilize before recording the depth to the groundwater. The elevation of the groundwater encountered in the eastern half of the site varies from elevation 885½ (B-1) to 886½ (B-3 and IB-3) to 889½ (IB-1). Groundwater was not encountered in the borings by Twining in 2004. Historic high groundwater may be as shallow as ten feet (CGS 2000).

5.1.5 Landslides

No landslides are present within or near the site nor are any shown on regional geologic maps (Dibblee 1993, Weber 1984).

5.2 FAULTING AND SEISMICITY

The subject site, like most in Ventura County, is located in a seismically active region prone to occasional damaging earthquakes. The destructive power of earthquakes can be grouped into fault-rupture, ground shaking (strong motion), and secondary effects of ground shaking such as tsunami, liquefaction, seismic settlement, mass wasting, and flooding from dam failures.

No faulting was observed during the site exploration of the property nor are any faults known to cross the site in the regional geologic literature. The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined, pre-existing, active faults. No doubt there is and will be exceptions to this, because it is not possible to predict the precise location of a new fault where none existed before (CDMG, 1975).

No Holocene-active faults are known to cross the site nor is the project site currently located within an Alquist-Priolo (A-P) Earthquake Fault Zone as defined by the State Geologist (CGS 2018). The closest mapped Holocene-active faults are the Simi Santa Rosa Fault Zone approximately 9.9 kilometers northwest of the site and the Malibu Coast Fault Zone, 15.5 kilometers south southeast. As such, the potential for ground rupture due to faulting during the lifetime of the project is considered remote.

However, the property, like any in the greater Southern California area will be subjected to strong ground motion generated from the occasional damaging earthquakes occurring within the life expectancy of the proposed project. Based on the United States Geological Survey (USGS) interactive web application, *Unified Hazard Tool*, <https://earthquake.usgs.gov/hazards/interactive/> probabilistic seismic hazard analyses (PSHA) predict the Design Basis Earthquake (10% chance of being exceeded in 50 years [475-year return period]) peak horizontal ground acceleration will be on the order of 0.424g for the stiff soil / soft rock conditions on Site (Site location Latitude 34.16369° North and Longitude -118.83946° West; Site Class C/D - assumed V_s=360 m/sec). The mean magnitude from this PSHA is 6.63 (Mw) with a mean distance of approximately 18.44 km from the property and a modal magnitude of 7.52 (Mw) with a modal distance of 22.85 km from the property.

The secondary effects of strong ground motion include tsunami, seiche, liquefaction, settlement, earthquake triggered landslides, and flooding from dam failures. Tsunamis are impulsively generated water waves that can cause damage to shoreline areas. A seiche is an oscillation wave within an enclosed body of water. The property is not near the ocean or adjacent a body of water and, therefore, is not subject to tsunami and seiche hazards nor is the property in the vicinity of any dam failure inundation zone (Thousand Oaks Safety Element 2014). Furthermore, based on the relatively flat, although gently sloped nature of the proposed development portion of the Site, it is not prone to earthquake triggered landslides. The property is shown to be outside of an area having a potential for liquefaction on the State's *Earthquake Zones of Required Investigation, Thousand Oaks Quadrangle, Seismic Hazard Zones Quadrangle Official Map* (CGS 2000); therefore, the potential for liquefaction is not considered a constraint on development.

Structures within the site may be designed using a simplified code-based approach and ground motion procedures for seismic design using the procedures in the California Building Code (CBC). Geotechnical seismic design criteria for the simplified Code based approach are presented below in Section 6.5.

5.3 INFILTRATION TESTING

Infiltration testing for stormwater infiltration BMP design was proposed in the vicinity of the proposed dog park and in the driveway areas towards the center and eastern portions of the property at shallow and deep depths. On the day of testing, water was still present in the test wells. The water level was refilled to above the slotted pipe and readings were taken at 30 minute intervals. However, no significant drop in the water level was measured. This is likely due to the near surface moist clayey fill soils, dense to hard fine-grained Older Alluvial soils, and encountered groundwater in IB-1, IB-3, and IB-6 at depths ranging from 29 feet to 45 feet below the existing ground surface. Consequently, infiltration BMPs are not recommended at the site.

6.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, the site may be developed with the proposed improvements; however, geotechnical recommendations presented in the following sections of this report should be incorporated into the design and construction of the project.

6.1 SITE PREPARATION AND GRADING

6.1.1 Site Cleanup and Demolition

Any existing structures and improvements, such as utility lines, foundations, hardscape, not to be a part of the project should be demolished and removed from the site. All existing vegetation, paving, debris, etc. should be removed from the proposed construction area. Rootballs and large root systems from the existing trees in the area of the proposed construction should be removed. Debris generated from the demolition should be hauled away.

6.1.2 Soil Removals

All old fill and any soils disturbed by demolition activities are unsuitable for support of the proposed structures.

Within Basement Excavations

The basement may be cut to the proposed grade and no additional soil removal should be necessary. However, the excavation bottom should be observed to confirm no additional removals are required. Soils disturbed during the basement excavation should be removed to firm in-place soils at the bottom of the excavation.

Outside Basement Excavations

Soil removals, as a minimum, should extend below any soils disturbed during the site clearing. For areas supporting shallow continuous and isolated footings outside of the basement areas, as well as those supporting structural fill or lightly loaded footings, the soil removals should extend to firm native soil.

Additionally, within the footprint of the proposed buildings removals should be made such that the disturbed soils and at least two feet of native soil is removed below existing ground or to a depth where three feet of newly compacted soils is provided below the foundations, whichever is deeper. Removals should extend to a horizontal distance of equal to the depth of removal or five feet, whichever is greater. Due to possible variations in the subsurface materials, local areas of deeper removals may be necessary.

After these removals are completed as addressed above, the exposed ground surface should be observed by a field representative of this office to confirm that it is suitable for placement of certified fill. No fill soils may be placed until completion of the geotechnical observation.

6.1.3 In-Place Soil Processing

Following removals, the underlying 8 inches should be scarified, moisture conditioned to above the optimum moisture content, and re-compacted to at least 90% of the maximum dry density as determined by ASTM D 1557.

6.1.4 Fill Placement

On-site soils may be reused as fill soils providing the soils are free of major vegetation, trash and debris. Rocks greater than 8 inches in diameter should be excluded from all fills placed. To facilitate footing excavations, the maximum size of rock should be 6 inches or less. Suitable fill soils should be placed in thin (8 to 12 inches maximum) lifts, brought to above optimum moisture content, and compacted to at least 90% of the maximum dry density as determined by ASTM D 1557.

6.1.5 Utility Trenches and Interior Slab Subgrade

All utility trenches within building areas, parking and drive areas, and backfill beneath interior slabs should be compacted to at least 90% of the maximum dry density as determined by ASTM

D1557. In landscape areas where settlement of the trench could be detrimental to proposed usages backfill should also be compacted to at least 90% of the maximum dry density as determined by ASTM D1557. Utility trench excavations within the zone of influence of the footings (2[horizontal]:1[vertical] projection from the bottom of footing) should be backfilled with compacted soil or concrete slurry and/or the foundations deepened such that the utility trench is not within the zone of footing influence. Utilities should be constructed in accordance with current practice and standards (such as the current Green Book).

6.2 EXCAVATIONS

6.2.1 General

The mixed-use development is being proposed with one level of subterranean parking, adjacent to Hampshire Road, which will have a finished floor approximately 10 feet below the existing grade. Therefore, the bottom of the basement footings could be 12 to 13 feet below the ground surface; and the maximum excavation depth is anticipated to be on the order of 13-14 feet.

Shallow excavations for construction made in properly engineered fill or firm natural soils should stand with vertical sides to a depth of 5 feet.

During construction, the contractor is responsible for the excavation and maintenance of safe and stable slope angles considering the subsurface conditions and the methods of operations. Temporary excavations should be made per the applicable requirements of the current Cal/OSHA excavation regulations. Surcharge loading, such as construction equipment or vehicle traffic, should be kept back from the top of temporary excavations a minimum horizontal distance of 10 feet.

6.2.2 Temporary Slopes

Shear tests performed on samples of the soils encountered during the subsurface exploration are relatively high in cohesion. Temporary slopes, above groundwater, to a maximum height of 20 feet may be constructed at a maximum ¾(horizontal):1(vertical) gradient. Surcharge loading, such as construction supplies or equipment or vehicle traffic, should be kept away from the top of slope 10 feet or the depth of excavation, whichever is more.

6.3 SHORING RECOMMENDATIONS

Excavation shoring may consist of soldier beams that are cantilevered, internally braced or with tieback anchors, or other recognized methods. Alternatively, soil nails may be considered. Shoring should be designed by a shoring engineer. Shoring will encounter soils as previously addressed herein. Lagging should be used to support the cut between the beams. Grouting is the preferred method to fill voids between the cut and lagging. Shoring should be designed to include the lowest construction elevation.

6.3.1 Soil Nailing

Soil nail walls may be considered as an option for shoring. Soil strength may be used for the design of soil nails. For the Older Alluvium soils an average shear strength of 25° friction and cohesion of 900 pounds per square foot may be used. A unit weight of 120 pounds per cubic foot may be used in the wall design.

6.3.2 Shored Depth

The shoring depth should be designed to facilitate building foundation construction and the slab underdrain described below.

6.3.3 Shoring Soil Pressure

Shoring should be designed for lateral earth pressure plus lateral pressure imposed by adjacent surcharges. Cantilevered shoring systems should be designed for an active earth pressure equal to 35 pounds per cubic foot for level conditions adjacent the excavation. This value is an ultimate value without a factor of safety. The width of active pressure acting on the pile below the bottom of the excavation may be taken as the pile diameter.

6.3.4 Surcharge Loading

Surcharge on the shoring from construction equipment or loading adjacent the top of a shored cut will occur. This office can evaluate the surcharge loading or the loads can be analyzed using the procedure presented in the city of Los Angeles Retaining Wall Design (Document No: P/BC 2020-083). In general, lateral pressure on the shoring due to a uniform area surcharge of intensity q (force/area) is equal to a uniform pressure of 0.4q over the entire height of the wall.

Construction surcharging should be maintained, as a minimum, a horizontal distance equal to the depth of excavation from the shoring unless the shoring is specifically designed for surcharge loading.

6.3.5 Soldier Beam Passive Pressure

A passive earth pressure of 300 pounds per cubic foot (pcf) below the base of excavation may be used for design of soldier beams spaced at least three diameters center to center. This value may be increased to a maximum of 3,000 pounds per square foot (psf). The surface area (pile diameter if encased in structural concrete) may be doubled for soldier beams a minimum of three diameters apart center to center in the evaluation of passive resistance.

For vertical support, a unit friction value of 300 pounds per foot of embedment may be used for that portion of the soldier beam encased in structural concrete extending below the lowest depth of excavation. The unit of friction is independent of the shaft diameter providing the shafts are at least 24 inch diameter. Fixity may be assumed at 5 feet below the lowest unsupported grade.

6.3.6 Lagging

Lagging consisting of treated timber will be required the entire depth of the shored excavation. Wood lagging should be new rough timber (full dimension) Douglas fir, straight, free of bends, and free from defects that might impair structural strength. Lagging to be left in-place shall be pressure treated for contact with soil. The upper two feet of shoring and lagging measured from the adjacent grade may be removed when the shoring is no longer needed for support of the excavation.

Lagging should be designed to resist an equivalent fluid pressure equal to 35 pcf measured below the ground surface. A maximum lagging pressure of 300 psf may be assumed where the maximum soldier beam spacing does not exceed 8 feet center to center. An alternate to installing lagging would be to construct the shoring as a continuous gunite/shotcrete wall descending as the excavation proceeds. Cavities behind the lagging and retained soils should be filled, preferably with sand/cement slurry.

6.3.7 Pile Construction

The shoring contractor should be prepared to provide methods to prevent caving where low cohesive soils are encountered. Construction should be performed in general accordance with ACI (American Concrete Institute) Manual of Concrete Practice, Section ACI 336.3R Design and Construction of Drilled Piers. Slurry should be used to support the pile excavations when adding water to the excavations is not sufficient to prevent caving of the excavations. The slurry may consist of a mixture of water and chemical polymers. The method selected by the contractor for excavation and construction of the drilled shafts should be discussed with this office. Piles should be drilled and cast in the same day.

Care should be exercised when casting adjacent piles to avoid blowout from one excavation into the other. Excavated materials from drilled shaft construction should not be spread over any areas of construction unless properly placed and compacted. Drilled pile excavations should be observed by this office prior to setting reinforcing steel to verify the anticipated geotechnical conditions.

6.3.8 Shoring Plan Review and Construction Inspection

A structural engineer with shoring experience should prepare the shoring plans. Our office should be provided the proposed shoring plans and calculations for review. Variations in subgrade conditions or construction techniques exercised by the Shoring Contractor should be reviewed by this office. This office should also perform geotechnical observations during the shoring construction.

6.3.9 Barricades

Appropriate barricades should be placed at the top of all temporary excavations that are approached by pedestrians or construction vehicle traffic.

6.4 SOIL EXPANSIVENESS

Soil expansion tests were performed on representative soil samples obtained from the property. Test results indicate the underlying materials have a medium expansion potential, in the 51-90 Expansion Index range.

Expansive soils contain clay particles that change in volume (shrink or swell) due to a change in the soil moisture content. The amount of volume change depends upon the soil swell potential (amount of expansive clay in the soil), availability of water to the soil, and the soil confining pressure. Swelling occurs when soils containing clay become wet due to excessive water from poor surface drainage, over-irrigation of lawns and planters, and sprinkler or plumbing leaks. Swelling clay soils can cause distress to structures, walks, drains, and patio slabs.

Swelling clay soils can cause distress to construction (generally as uplift). Construction on expansive soil has an inherent risk that should be acknowledged and understood by the developer/property owner. The geotechnical recommendations presented herein are intended to reduce the potential for expansive soil action. However, these recommendations are not intended, nor designed to provide complete and full mitigation of expansive soil conditions. If requested, additional recommendations can be provided to further reduce the risk of expansive soil movement. The following should be maintained within the site.

a) Positive drainage should be continuously maintained away from structures and slopes. Ponding or trapping of water in localized areas near the foundations can cause differential moisture levels in subsurface soils. Plumbing leaks should be immediately repaired so that the subgrade soils underlying the structure do not become saturated.

b) Trees and large shrubbery should not be planted where roots can grow under foundations and flatwork when they mature.

c) Landscape watering should be held to a minimum; however, landscaped areas should be maintained in a uniformly moist condition and not allowed to dry-out. During extreme hot and dry periods, adequate watering should be provided to keep soil from separating or pulling back from the foundations.

d) According to the County of Ventura Building Code where buildings are located on soils having an Expansion Index of greater than 50, gutters and downspouts should be installed on the buildings to collect roof water and direct the water away from the structure. Downspouts should drain into PVC collector pipes that will carry the water away from the building.

6.5 CORROSION AND CHEMICAL TESTING

Two near surface soil samples were tested for corrosivity by an independent testing laboratory (Project X). The results of the preliminary corrosion testing and discussions are provided in Appendix B, Laboratory Testing, of this report. Based upon the test results the soil is classified as S0. As such, concrete in contact with site soils should have a minimum strength of 2,500 psi. The soils are also classified as severely corrosive to metals. For specific recommendations a corrosion engineer should be consulted. Additional testing should be performed subsequent to rough grading.

6.6 GEOTECHNICAL SEISMIC DESIGN

The site may experience strong ground shaking from seismic events generated on regionally active faults. Structures within the site may be designed using a simplified code-based approach and ground motion procedures for seismic design using the procedures in the California Building Code (CBC). Seismic ground motion values based on ASCE/SEI 7-16 are adjusted to obtain the maximum considered earthquake (MCE) spectral acceleration values for the site based on its site class of D for the stiff soil site. The seismic design parameters for the site's coordinates (Latitude 34.16369° North and Longitude 118.83946° West) were obtained from the USGS web based spectral acceleration response maps and calculator: <https://seismicmaps.org/>.

Seismic Parameters based on ASCE/SEI 7-16

SEISMIC PARAMETER	**VALUE PER ASCE/SEI 7-16**
Site Class Definition	D
MCE_R ground motion (for 0.2 second period), S_s	1.439g
MCE_R ground motion (for 1.0 second period), S_1	0.512g
Site amplification factor at 0.2 second, F_a	1.0
Site amplification factor at 1.0 second, F_v	--
Site-modified spectral acceleration value, $S_{MS} = F_aS_s$	1.439g
Site-modified spectral acceleration value, $S_{M1} = F_vS_1$	--
Numeric seismic design value at 0.2 second, $S_{DS} = 2/3S_{MS}$	0.959g
Numeric seismic design value at 1.0 second, $S_{D1} = 2/3S_{M1}$	--
Site Modified peak ground acceleration, PGA_M	0.596g

The purpose of the building code earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage nor maintain function. Therefore, values provided in the building code should be considered minimum design values and should be used with the understanding site acceleration could be higher than addressed by code-based parameters. Cracking of walls, cosmetic damage and possible structural damage should be anticipated in a significant seismic event.

6.7 SHALLOW FOUNDATION DESIGN RECOMMENDATIONS

Foundations for all new at-grade construction should be founded in engineered compacted fill materials. Basement foundations may be supported on either engineered fill or competent native materials, but not both. All building foundations should maintain slope setback distances for foundations. Geotechnical recommendations are presented below. These recommendations are based on soils with a medium expansion potential (51-90 Expansion Index range).

6.7.1 Conventional Footings

The proposed new development will include improvements with varying foundation needs from a geotechnical standpoint; the proposed development includes one-story structures, three-story residential structures, and three-story above ground, mix-use residential/commercial structures with subterranean basement-level parking. Proposed improvements may also include trash enclosures, lighting fixtures in parking and walkway areas, and a retaining wall adjacent the pool area.

Due to the expansive nature of the onsite soils the embedment should be a minimum of 30 inches for perimeter and interior footings. Where basement walls are retaining more than 2 feet of soil, the footings may be founded at a depth of 24 inches. The lowest adjacent grade is the lowest soil grade adjacent the footings. However, the embedment of interior footings may be measured from the top of the interior concrete slab-on-grade.

Light-weight structures and one-story structures may be supported on continuous and isolated footings a minimum of 12 and 24 inches wide, respectively. Multi-story structures may be supported on continuous and isolated footings a minimum of 15 and 24 inches wide, respectively. The footings may be designed to impose an allowable soil bearing pressure of 2,000 psf provided they are a minimum of 30 inches below the lowest adjacent grade. The above net allowable bearing capacity may be increased by one-third for short-term wind and seismic loads.

Steel reinforcement should be per the structural engineer's recommendations. However, minimum reinforcement for continuous footings should consist of two number five bars in the top and bottom (minimum total of four bars).

Footings should be set back from ascending and descending slopes per the 2019 California Building Code Section 1808. Footings located behind a retaining wall should be embedded below a 2(h):1(v) line extending up from the base of the wall or the wall should be designed to support the footing surcharge.

6.7.2 Lateral Resistance

Passive soil pressure and friction may be used to resist lateral forces on the foundations. Passive earth pressure in newly compacted fill may be taken as an equivalent fluid pressure equal to 250 pcf for level ground (maximum passive pressure should not exceed 2,000 psf). Friction between the bottom of the footings and soil may be taken as 0.25. These values have a factor of safety of 1.5.

6.7.3 Settlement

Preliminary settlement analyses were performed based on a bearing capacity of 2,000 psf. Estimated settlements of shallow foundations due to static loading are anticipated to be on the order of ¾ inch. These estimates should be confirmed when the actual foundation loads become available. Differential settlement between adjacent, similarly loaded new foundations may be on the order of one half the total settlement.

6.7.4 Footing Excavations

All footings and edges should be cut square and level. A representative from this office should observe the footing excavations prior to placing reinforcing steel. Prior to concrete placement the footing excavations should be cleaned of slough and soils silted into the excavations during the pre-moistening operations. Soil excavated from the footing trenches should not be spread over any areas of construction unless placed as a properly compacted fill.

6.7.5 Conventional Slab-on-Grade

Slabs-on-grade should be designed to support the anticipated loading. The slab should be at least 5 inches thick and reinforced with #3 bars at 24 inches on center both ways, or per the structural engineer's recommendations. The slab reinforcement should be extended into the footings to within 3 inches of the bottom. The slab thickness and reinforcement design recommendations herein are provided from a geotechnical standpoint only and are not intended to address any structural engineering requirements.

6.7.6 Mat Slab Design Data

Mat slabs may be designed using an allowable soil bearing pressure of 1,500 pounds per square foot or a modulus of subgrade reaction "K" of 100 pounds per cubic inch (pci) at the surface of a properly prepared building pad. The project structural engineer should determine the steel reinforcement and concrete compressive strength. The project structural engineer should determine the slab thickness; however, the slab should be a minimum of 10 inches thick.

6.7.7 Under-Slab Treatment

Where moisture sensitive floor coverings will be utilized, an appropriate moisture vapor retarder layer should be installed and maintained below the concrete slab to reduce moisture vapor transmission through the slab. Fifteen-mil plastic sheeting is commonly used as a moisture vapor retarder layer. The sheeting should be installed and perforations through the moisture vapor retarder such as at pipes, conduits, columns, grade beams, and wall footing penetrations should be sealed per the manufacturer's specifications or ASTM E1643 Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill under Concrete Slabs. Proper construction practices should be followed during construction of the slab-on-grade.

Minimizing shrinkage cracks in the slab-on-grade can further minimize moisture vapor emissions. A properly cured slab utilizing low-slump concrete will reduce the risk of shrinkage cracks in the slab as described herein.

The slabs should be tested for moisture content prior to the selection of the flooring and adhesives. Moisture in the slabs should not exceed the flooring manufacturer's specifications. The concrete surface should be sealed per the manufacturer's specifications if the moisture readings are excessive. It may be necessary to select floor coverings that are applicable to high moisture conditions.

6.7.8 Slab and Foundation Pre-Moistening

Slab and Foundation subgrade soils should be pre-moistened to 3 percent over the optimum moisture content for a minimum depth of 18 inches. A representative of this office should observe pre-moistening of the subgrade before sand base or concrete placement.

6.7.9 Concrete Placement and Cracking

Minor cracking of concrete slabs is common and generally the result of concrete shrinkage continuing after construction. Concrete shrinks as it cures resulting in shrinkage tension within the concrete mass. Since concrete is weak in tension, development of tension results in cracks within the concrete. Therefore, concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing, and curing should be performed per the American

Concrete Institute Guide for Concrete Floor and Slab Construction (ACI 302.1). Concrete slump during concrete placement should not exceed the design slump specified by the structural engineer. Where shrinkage cracks would be unsightly, concrete slabs on grade including post-tensioned slabs should be provided with tooled crack control joints at 10-15 foot centers or as specified by the structural engineer.

6.7.10 Flooring Coverings

Tile flooring can crack, reflecting cracks in the concrete slab below the tile. Therefore, the slab designer should consider additional steel reinforcement of concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile. A vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) is recommended between tile and concrete slabs-on-grade performed per the Portland Cement Association Specifications. Concrete slabs-on-grade should be tested for moisture if moisture sensitive floor coverings such as wood flooring or wool carpet are used over the slabs. The slabs should be sealed if the moisture is higher that recommended by the flooring manufacturer.

6.8 SUBTERRANEAN DRAINAGE

The historic groundwater level (CGS 2000) is near the base of the proposed basement level and the encountered soils were above optimum moisture; therefore, subterranean construction may encounter moist to wet soil conditions over the life of the structure. As such, it is recommended that a subdrain be placed below the slab within the basement. In addition, the retaining walls should be waterproofed and provided with backdrains.

Below slab drains are intended to provide drainage of groundwater from below the basement floor, should groundwater rise to the basement level. However, drains will not drain water naturally held by the soils or stop vapor migration.

The interior slab should be constructed on 6 inches of 3/4± rock. An acceptable gradation would be as specified in the Standard Specifications for Public Works Construction (Greenbook) Table 200-1.2, Crushed Rock and Rock Dust for 3/4-inch rock. However, the rock may be rounded or crushed. The rock should be placed on a properly prepared subgrade as addressed herein and should be separated from the subgrade by a single layer of filter cloth. Filter cloth having a maximum equivalent opening of 0.212 mm (70 U.S. sieve size) should be lapped at least 12 inches at the seams and the seams sealed per the manufacture’s specifications.

Directly above the filter cloth within the rock, rows of 4-inch PVC (SDR 35 or approved equivalent) perforated pipes should be placed with holes down at a maximum pipe spacing of 25 to 30 feet and preferably with a slight slope to drain (or horizontal if necessary). Where piping must cross a structural element, a sleeve should be constructed per the structural engineer’s design. Manifold piping or solid piping connecting the drains to the sump system or storm drain may be 4 inch or larger PVC (SDR 35 or approved equivalent) that is non-perforated with glued connections. Drainpipes should be connected to a single outlet pipe prior to exiting the building or into a sump. Connector pipes should be placed preferably with a slight slope to drain (or horizontal if necessary). Rock should be carefully placed over the piping so as not to disturb the pipe layout or distort the piping.

6.9 RETAINING WALLS

6.9.1 Foundations

Allowable bearing capacities and lateral resistance provided herein for conventional footings may be used for retaining/subterranean wall design.

6.9.2 Lateral Earth Pressures

Retaining walls should be designed to resist an active pressure exerted by compacted backfill or retained soil. Walls that may yield at the top should be designed for an equivalent fluid pressure equal to 40 for a level condition behind the wall. The above active pressures do not contain provisions for adverse geologic conditions, possible expansion of the backfill, nor for hydrostatic pressure building up behind the walls. If water is allowed to collect behind the wall or to saturate the backfill, the actual pressure may exceed the active pressure provided.

Surcharges may be treated as additional height of backfill. Assume one foot of additional height for each 125 psf of areal surcharge. Vehicle wheel loads (light to moderate) should be taken as two feet of additional surcharge. Lateral loads imposed by adjacent shallow foundations should be added to the lateral earth pressure.

Retaining walls restrained at the top should be designed for a minimum lateral earth pressure equal to 28H with a trapezoidal distribution. The lateral pressure is an ultimate value with no factor of safety included. The lateral earth pressure at the ground surface may be taken as zero. The pressure will increase with depth to 28H at a depth of .2H below the ground surface. The shoring pressure would then extend uniformly at 28H to a depth of 0.8H and decrease uniformly to zero at the base of the excavation. H is the supported height of the wall. The resultant of 28H is in units of psf. Surcharges from adjacent loading should be added to the wall active pressure. The basement wall should be designed for the appropriate factor of safety as determined by the structural engineer.

6.9.3 Earth Pressures-Seismic

Walls greater than 6 feet in height should be designed for a seismic lateral pressure. The seismic equivalent fluid pressure may be taken as 10 pounds per cubic foot for a cantilever wall with level backfill and 16 pcf for a restrained wall with level backfill. This force is added to the static earth pressures.

$\Delta P_{AE} = ½*\gamma* H^2*(.42*PGA/g) = ½*120*H^2*(.42*0.384) = 10$ pcf for cantilever wall, level backfill

$\Delta P_{AE} = ½*\gamma* H^2*(.68*PGA/g) = ½*120*H^2*(.68*0.384) = 16$ pcf for restrained wall, level backfill

Where $PGA = S_{DS}/2.5 = 0.959/2.5 = 0.384$

6.9.4 Waterproofing

Basement walls should be waterproofed on the exterior in addition to installing the drainage system and wall backfill. The waterproofing and backdrain system should be designed by a waterproofing consultant experienced with this type of structure.

6.9.5 Drainage

Retaining walls should be provided with a drainage system behind the wall consisting of a manufactured composite drain board or a section of aggregate drain material. An aggregate drain should consist of a minimum 1-foot wide continuous section of drain material (clean 3/4 inch crushed rock) extending from the base of wall to the top of wall for interior walls or to within 2 feet of the top of exterior walls. The upper 2 feet of exterior wall backfill should consist of compacted native soils. A layer of filter cloth should be placed between the drain material and soil to minimize the migration of fines into the drain material. A composite drain board may consist of a prefabricated drainage composite consisting of a filter fabric bonded to a corrugated panel.

Composite drain boards or aggregate sections should be drained by a perforated drainpipe. For exterior walls, weep holes may be used or a perforated drainpipe (perforations 3/8" or smaller, perforations down) may be placed in the bottom of the drain material (2 inches above the bottom of the gravel). The invert of the drainpipe should be at least 6 inches below any adjacent slab-on-grade or grade. The outlet pipe from the basement perimeter drain should be a non-perforated 4-inch diameter PVC (SDR 35 or approved equivalent) pipe that is sloped to and connected to a storm drain system or sump. An as-built plan should be prepared detailing the location of the wall drainage system.

6.9.6 Backfilling

Retaining walls should be backfilled where necessary with granular material or soils having a low expansion potential. Onsite soil may not be suitable. The backfill should be placed in 6-inch lifts at slightly over optimum moisture content and compacted to at least 90% relative compaction. If the backcut is flatter than ½(h):1(v), the backfill should be benched into the backcut slope. Light equipment should be used immediately behind the walls to prevent possible over-stressing. Bracing needed to resist basement wall movement should be in-place prior to placing the backfill.

6.10 SWIMMING POOL

6.10.1 General

A swimming pool may be constructed on the property from a geotechnical standpoint if the following geotechnical recommendations are followed and incorporated into the design. Risks associated with pool construction, such as pool or deck movement, cannot be completely eliminated, especially if proper construction practices, drainage, maintenance of landscaping, pool plumbing and pool equipment are not provided. A representative of this office should review pool plans to confirm conformance to the recommendation presented below and observe all geotechnical aspects of the pool construction addressed herein. Pools and decking should be designed for soils with a medium expansion potential, 51-90 Expansion Index range.

6.10.2 Pool Excavation

All aspects of grading for the pool including site preparation, excavation, and fill placement should be per the City of Thousand Oaks Building Code. Soils exposed in the pool excavation should be kept moist until the concrete placement. The concrete should be cast as soon as possible after excavation to avoid desiccation of the subgrade material. Completion of the pool excavation and construction should be performed in a timely manner so the excavation is open for a maximum of only two weeks. Soil excavated from the pool area should not be spread over areas of construction or slopes unless properly placed and compacted as previously described in the referenced reports.

Due to the presence of expansive soils, the pool perimeter should be deepened such that the shallow and deep ends are founded at the same depth and in the same material. The proposed supporting material for the pool construction is anticipated to be artificial fill. The bottom and sides of the pool excavation must be observed by the project geotechnical consultant, before placing structural steel or concrete to verify suitability of foundation bearing materials.

6.10.3 Pool Walls

The pool shell should be designed for medium expansive soils condition (51-90 Expansion Index range). Additionally, the walls should be designed for an equivalent fluid pressure equal to 65 pcf. Water should not be allowed to saturate the soils behind the walls as the expansion pressures could exceed the active pressure provided. The requirements of the City of Thousand Oaks Building Code regarding setback to a slope should be followed however, the setback to a descending slope should not be reduced by 1/2.

The pool walls near a structure should be designed to support loads imposed by the structure on the pool wall. Foundations located below a 2(h):1(v) line extending up from the base of the pool wall should not impose loads on the pool wall. A structural engineer should evaluate the impact of an adjacent structure on the pool wall and design pool walls accordingly.

6.10.4 Pool Drainage

A hydrostatic popup valve is suggested in the pool bottom to relieve possible hydrostatic pressure should the pool be drained of water.

6.10.5 Pool Plumbing

Piping should be protected so that the piping is not damaged during backfilling around the pool. It is imperative that any leaks in the pool plumbing or drainage system be repaired immediately. Leaks in the plumbing can cause saturation of the soils adjacent the pool resulting in possible deck or pool movement.

6.10.6 Concrete Deck

Concrete decking and hardscape surrounding the swimming pool should be constructed on engineered compacted fill or Older Alluvium. Soil excavated from the swimming pool area or elsewhere, should not be used underneath the deck unless properly compacted and moisture conditioned. Joints between adjoining sections of pool decking and between the pool decking and the pool walls should be caulked. Periodic inspection by the owner and subsequent re-caulking, if necessary, are important maintenance procedures that will help prevent water from migrating into the supporting subgrade.

Drainage should be collected at area drains that will convey the water to paved drainage surfaces. Drainage water should not be disposed of on any of the adjacent descending slopes.

6.11 EXTERIOR SLABS AND WALKWAYS

Exterior slabs and walkways for pedestrian use may be 4 inches thick, reinforced with #3 bars at 24 inches on center each way and underlain by a minimum of 4 inches of sand or aggregate base. Where adjacent to landscape areas the slabs/walkways should be provided with a deepened edge. Subgrade soils should be pre-moistened to a minimum of 3 percent over the optimum moisture content to a depth of 18 inches. Uplift of the slabs and walkways can occur if premoistening is not performed prior to concrete placement or if excessive irrigation is applied to soils supporting the hardscape.

Aggregate base materials should be placed in thin (6 inch maximum) lifts, moistened to slightly above the optimum moisture content and compacted to at least 95% of the laboratory standard.

Concrete slabs on grade should be provided with tooled crack control joints at 10-to-15-foot centers or as specified by the slab designer. Additional contraction and isolation joints should be installed, as necessary, where hardscape layout or protrusions of the structure result in reentrant, or inside, corners in the hardscape. Installation of a felt or similar separator should be considered within all areas where concrete hardscape meets the structure.

Shrinkage cracking can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing and curing should be performed per the Portland Cement Association guidelines.

6.12 PRELIMINARY PAVEMENT RECOMMENDATIONS

Presented herein are preliminary structural section recommendations for pavement within the subject project. The structural sections are based on a range of traffic conditions and an assumed Resistance value ("R" value) of 5 based on the referenced site evaluations and observed soil conditions. R-Value testing should be performed after grading. Actual traffic indices should be confirmed by the project design civil engineer.

PRELIMINARY PAVEMENT SECTIONS	
	"R" Value ~ 5
Traffic Index	**Asphaltic Concrete Sections**
4.5	3" AC / 9" AB
5.0	3" AC / 10" AB
6.0	3" AC / 14" AB or 4" AC / 12" AB
AC = Asphaltic Concrete AB = Aggregate Base	

6.12.1 Subgrade Preparation

The subgrade soils within areas of proposed paving should be moistened to slightly above the optimum moisture content and compacted to at least 95% of the laboratory standard prior to placing aggregate base.

6.12.2 Aggregate Base Preparation

The aggregate base materials should be placed in thin (6 inch maximum) lifts, moistened to slightly above the optimum moisture content and compacted to at least 95% of the laboratory standard.

7.0 SITE DRAINAGE

Positive drainage should be provided away from structures during and after construction per the grading plan or applicable building codes. Water should not be allowed to gather or pond against foundations. In addition, planters near a structure should be constructed so that irrigation water will not saturate footing and slab subgrade soils.

8.0 PLAN REVIEW

As the project development design process continues, grading and foundation plans should be reviewed by the project geotechnical consultant as they become available. Revised and/or additional geotechnical recommendations will be provided as required.

9.0 CLOSURE

This report was prepared under the direction of a licensed geotechnical engineer and certified engineering geologist. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Gorian and Associates, Inc. disclaim any and all responsibility and liability for problems that may occur if the recommendations presented in this report are not followed.

This report was prepared for imt Residential and their design consultants solely for the design and construction of the project described herein. This report may not contain sufficient information for other uses or the purposes of other parties. These recommendations should not be extrapolated to other areas or used for other facilities without consulting Gorian and Associates, Inc.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions that can vary horizontally and vertically across the site. Due to possible subsurface variations, representatives of this office should observe all aspects of field construction addressed in this report.

Any person using this report for bidding or construction purposes should perform such independent investigations as they deem necessary.

We recommended all earthwork be observed and tested by the project geotechnical consultant including site stripping, removals and placement of compacted fill as well as floor slab subgrades, and footing excavations. The work should be performed per the current City of Thousand Oaks Building Code. However, the services of the geotechnical consultant should not be construed to relieve the owner or contractors of their responsibilities or liabilities.

-oOo-

We appreciate the opportunity to submit this geotechnical report and look forward to continuing our service on the project design and construction team. Please call if you have any questions concerning this report, or require any additional information.

Respectfully submitted,

GORIAN AND ASSOCIATES, INC.

By: Sheryl N. Shatz, GE 2288
Senior Geotechnical Engineer

By: William F. Cavan, Jr., CEG 1161
Principal Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
SHERYL N. SHATZ
No. 2288
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
WILLIAM F. CAVAN, JR.
No. EG1161
CERTIFIED ENGINEERING GEOLOGIST
EXP. 9/30/2023
STATE OF CALIFORNIA

Distribution: Addressee, via e-mail
Stantec, via e-mail

10.0 REFERENCES

California Division of Mines and Geology (CDMG) now California Geological Survey (CGS), 1975, *Guidelines for Evaluating the Hazard of Surface Fault Rupture.* CDMG Note Number 49.

California Division of Mines and Geology (CDMG) now California Geological Survey (CGS), 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*. California Division of Mines and Geology Special Publication 117A.

California Geological Survey (CGS), *Earthquake Zones of Required Investigation Thousand Oaks Quadrangle* SEISMIC HAZARD ZONES Released: November 17, 2000.

California Geological Survey (CGS), 2000, *Seismic Hazard Zone Report for The Thousand Oaks 7.5-Minute Quadrangle, Ventura and Los Angeles Counties, California*, Seismic Hazard Zone Report 042 latest revision 1/17/06.

California Geological Survey (CGS), 2018, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California*. California Geological Survey Special Publication 42, Revised 2018. <https://www.conservation.ca.gov/cgs/Documents/Publications/Special-Publications/SP_42.pdf>.

Dibblee, Jr., Thomas W., and Helmut E. Ehrenspeck, 1993, *Geologic Map of the Thousand Oak Quadrangle, Ventura and Los Angeles Counties, California*. Dibblee Geological Foundation, Map #DF-49.

Gorian and Associates, Inc., August 24, 1987, *Geotechnical Site Investigation, Proposed 16 Unit Apartment Complex, Lots 9, 10 and Portions of Lots 16 and 17, Block 21, Thousand Oaks Tract, Foothill Drive, City of Thousand Oaks, California.* Work Order: 1607-1-10.

Jennings, C.W. and Bryant, W.A. with Assistance from Saucedo, G.J., 2010, *Fault Activity Map of California.* California Geological Survey, California Geologic Data Map Series, Map No. 6. <http://maps.conservation.ca.gov/cgs/fam/>

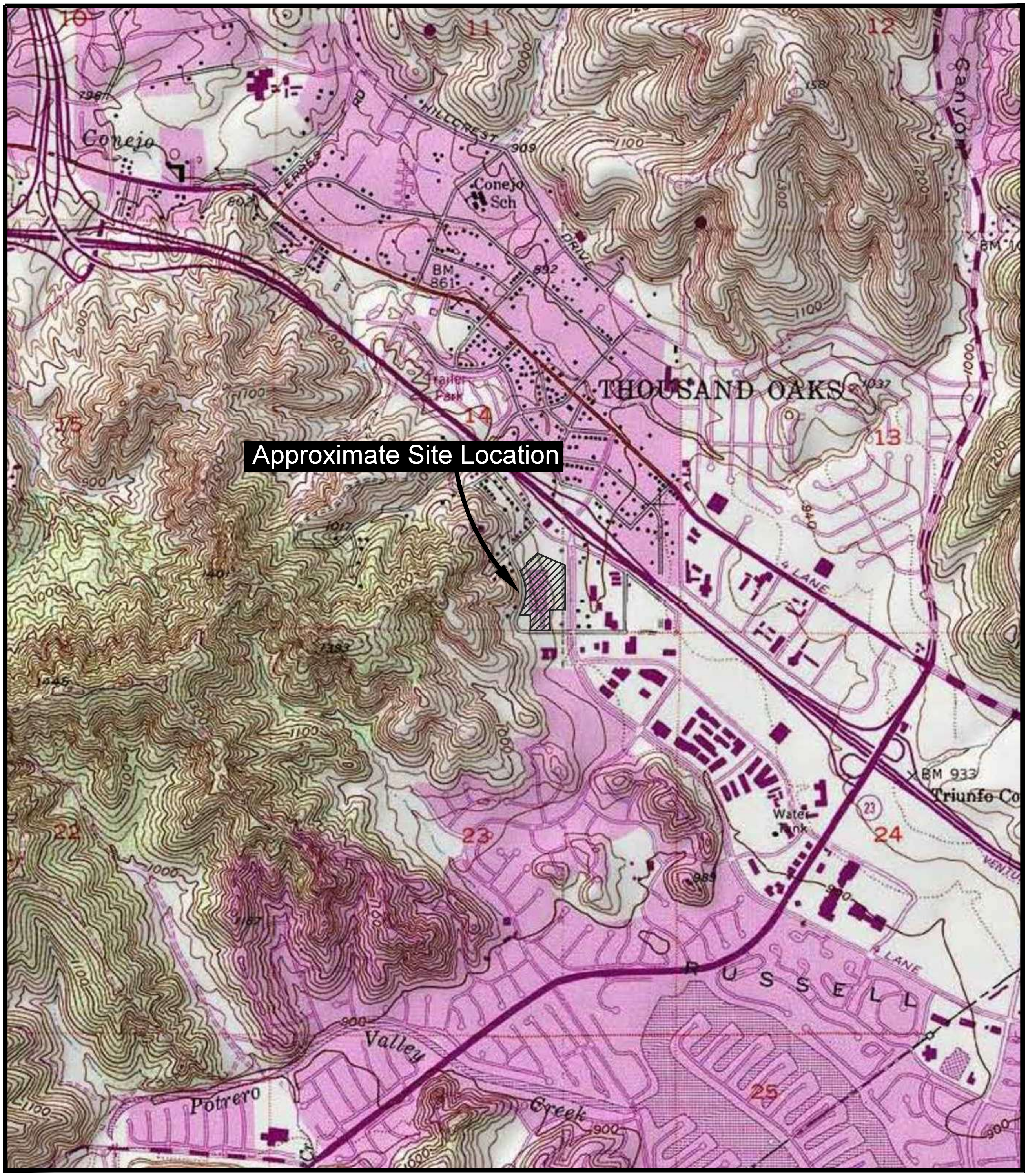
SEAOC/OSHPD, *Seismic Design Maps Tool*, <https://seismicmaps.org/>.

Twining Laboratories, Inc., September 13, 2005, *Draft Geotechnical Engineering Investigation, Proposed Home Depot, 325 Hampshire Road, Thousand Oaks, California.* Project Number: TL D050A3.01-02.

City of Thousand Oaks, 2014, *Thousand Oaks General Plan, Safety Element 2014 Update*, dated March 2014 <https://www.toaks.org/home/showpublisheddocument?id=344>.

United States Geological Survey (USGS) interactive web application, *Unified Hazard Tool*. <https://earthquake.usgs.gov/hazards/interactive/>.

Weber, F. H., 1984, *Geology of the Calabasas-Agoura-Eastern Thousand Oaks Area, Los Angeles and Ventura Counties, California*. California Division of Mines and Geology Open-File Report 84-1 LA.



Source

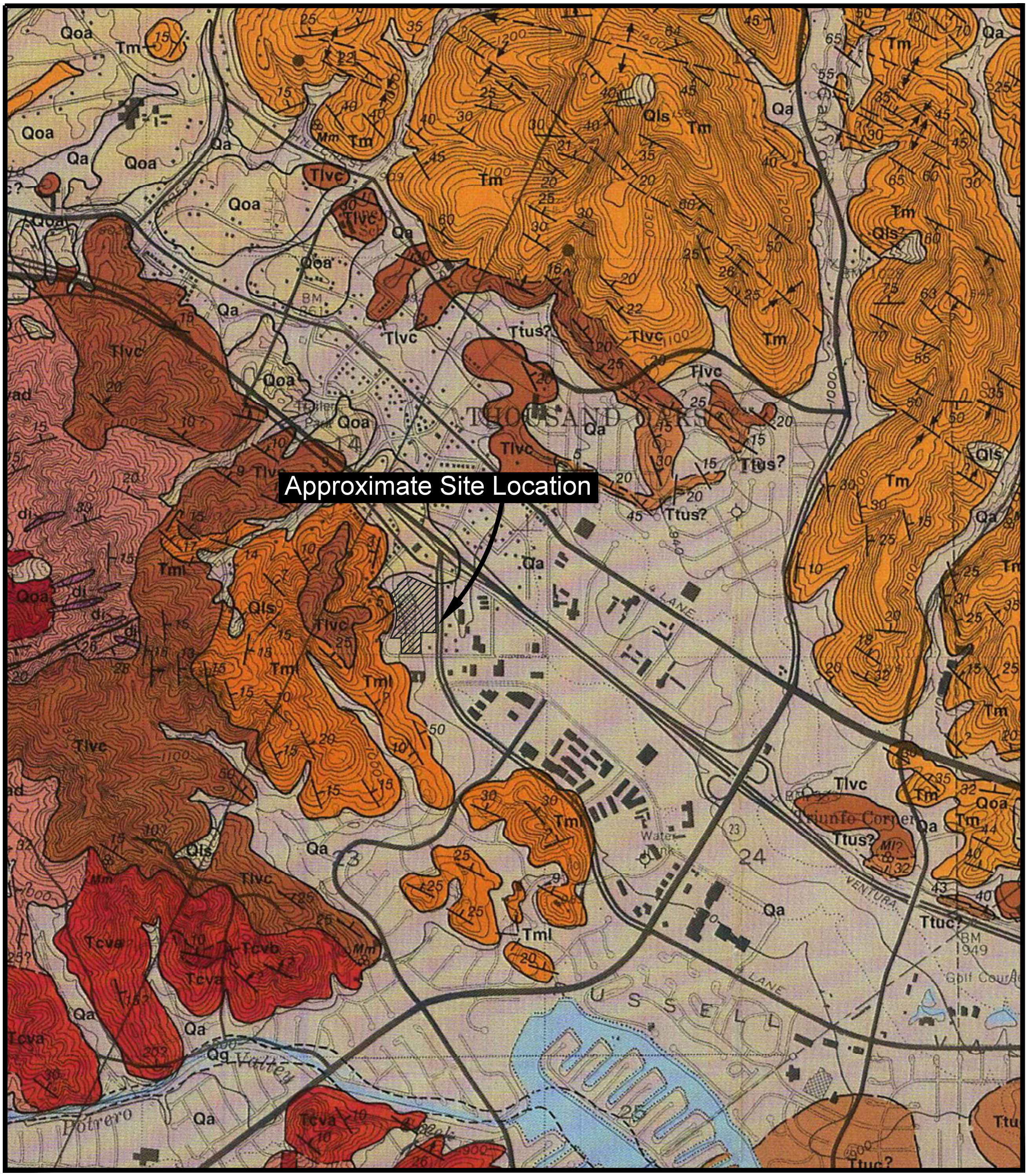
United States Geological Survey, Thousand Oaks Quadrangle, California - Ventura County. 7.5 Minute Series (Topographic)

VICINITY MAP

325 Hampshire Rd,
Westlake Village, CA 91361

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3196-0-0-100	Date: September 2021
Scale: 1" = 2000'	Drawn by: Approved by:
	FIGURE 1



Source: Dibble Jr., Thomas W., Ehrenspeck, Helmut E., 1993, *GEOLOGIC MAP OF THE THOUSAND OAKS QUADRANGLE, VENTURA AND LOS ANGELES COUNTIES, CALIFORNIA.* Dibblee Geological Foundation Map #DF-49.

Explanation

Qa - Alluvial gravel, sand, and clay of valley areas

Tml - Monterey Formation (Modelo Formation and in part Upper Topanga Formation of Weber 1984.) Shale - fissile to punky; includes scattered thin, hard calcareous layers and concretions, middle Miocene Age

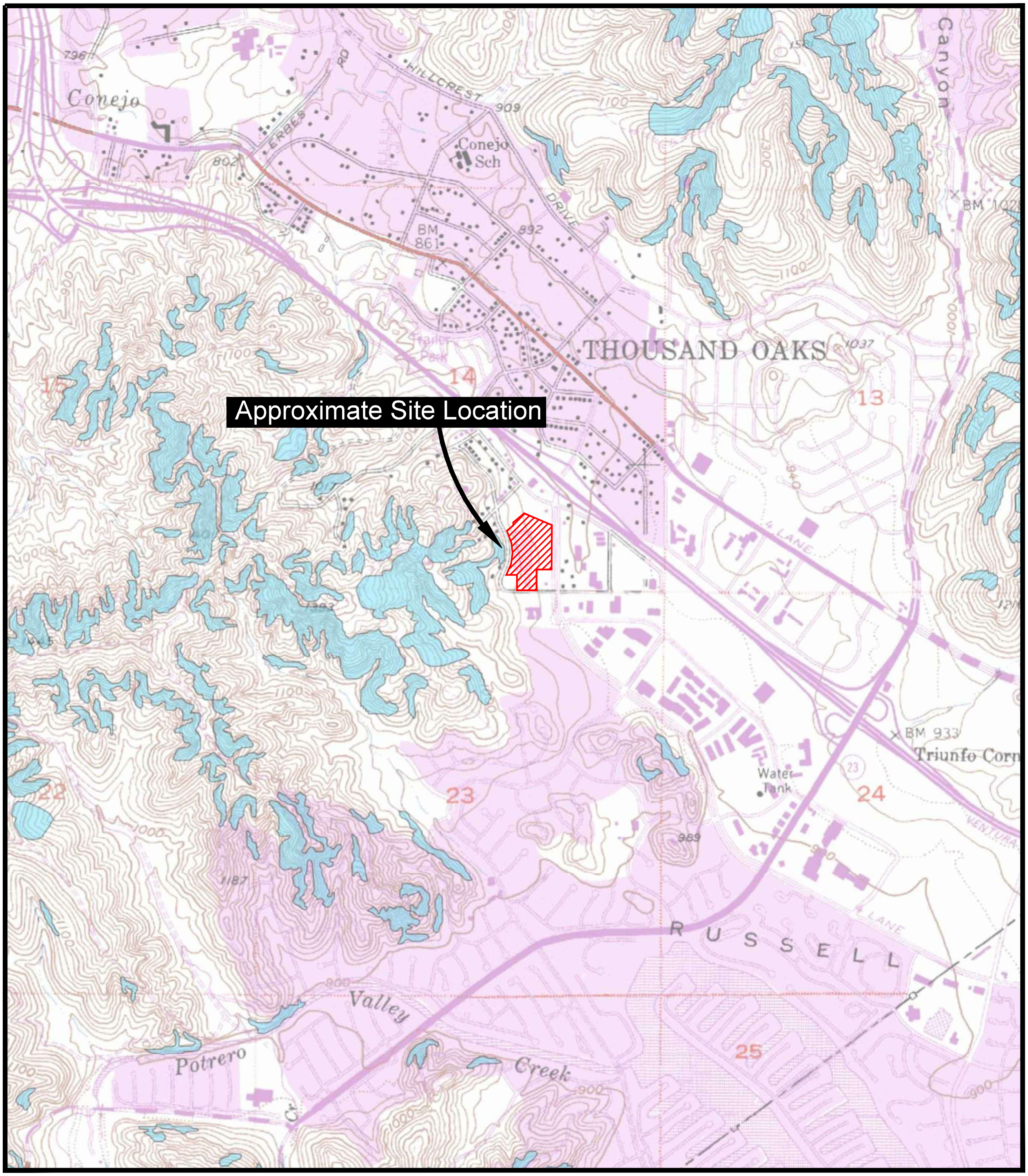
Tlvc - Detrital Sediments of Lindero Canyon (Upper Topanga of Weber 1984.) Basal epiclastic (reworked) conglomerate of detritus derived from Conejo Volcanics andesitic/basaltic rocks

REGIONAL GEOLOGIC MAP

325 Hampshire Rd,
Westlake Village, CA 91361

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3196-0-0-100
Date: September 2021
Scale: 1" = 2000'
Drawn by:
Approved by:
FIGURE 2



Source: California Geological Survey (1995), EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, THOUSAND OAKS QUADRANGLE. Seismic Hazard Zones Official Map Released November 17, 2000

SEISMIC HAZARD ZONES

🟩 **Liquefaction Zones**
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

🟦 **Earthquake-Induced Landslide Zones**
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

SEISMIC HAZARD MAP

325 Hampshire Rd,
Westlake Village, CA 91361

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3196-0-0-100
Date: September 2021
Scale: 1" = 2000'
Drawn by:
Approved by:
FIGURE 3

APPENDIX A

LOGS OF SUBSURFACE DATA

GORIAN
& ASSOCIATES, INC.

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation
Number: B-1

Page Number: 1

Date(s) Excavated 7/14/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 916½±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE 5" OVER 3" BASE	
915	■	■	17	23.9	98	CL	▨	FILL: Dark grayish brown to very dark grayish brown silty CLAY, trace fine to coarse Sand, trace fine gravel (moist, medium stiff).	
						CL	▨	Very dark grayish brown silty CLAY, trace fine to coarse Sand, trace fine gravel (moist, medium stiff).	
5 910		■	22	24.3	96				
		■	36	23.1	96	ML	▥	Olive brown clayey SILT mottled with dark grayish brown silty CLAY, trace fine gravel (moist, hard). At shoe of sampler, calcium carbonate veinlets.	
10 905		■	80	25.5	93	ML	▥	OLDER ALLUVIUM: Strong brown very clayey SILT with fine to coarse Sand, trace fine gravel (damp to moist, hard). Iron oxide staining, carbon nodules.	
		■	66	25.8	94			At 12½'; fine to coarse gravel.	
15 900		■	52	15.7	104	CL	▨	Strong brown sandy to very silty CLAY, fine to coarse gravel (damp to moist, hard). Carbon nodules.	
		■	59	22.5	98	CL	▨	Strong brown fine sandy to very silty CLAY (moist, hard). Carbon nodules.	
20 895		■	59	25.6	97				
		■	63	23.4	98			At 22½'; iron oxide staining.	
25 890		■	58	25.1	96				
		■	58	25.8	98			At 27½'; becoming very moist.	
30 885		■	30	24.8	98				
		◩	7/10/14			ML	▥	Strong brown clayey SILT, trace fine Sand (very moist, very stiff). Carbon nodules.	
						CL	▨	Strong brown to reddish brown silty CLAY (moist, hard). Carbon nodules.	
35 880		■	45	23.7	99			At 35'; strong brown.	
		◩	9/16/21						
40 875		◩	5/7/11					At 40'; saturated, becoming stiff, some fine to coarse Sand.	At 40': FC = 68%

GORIAN & ASSOCIATES, INC.

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: B-1

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
		◣	3/5/8				▨		
45		◣	3/5/10				▨		
870							▨	At 46'; becoming reddish brown to dark strong brown.	
		◣	3/4/7				▨		
50		◣	3/5/6			SM	░	Strong brown silty fine to medium SAND (saturated, medium dense).	
865						CL	▨	Strong brown silty CLAY (saturated, stiff). Carbon nodules.	
55									
860									
60									
855									
65									
850									
70									
845									
75									
840									
80									
835									
85									
830									

Total Depth 51½'
Groundwater Encountered at 40', measured at 31' after 5 minutes.
No Caving Observed

Backfilled with cuttings, topped with AC patch.

GORIAN
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Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation
Number: B-2

Page Number: 1

Date(s) Excavated 8/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 920±
Excavation Dimension 24" Dia.	Equipment Contractor Tri Valley Drilling	Equipment Type Bucket Auger	Hammer Data *SEE NOTE

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
920 0							■	ASPHALT CONCRETE: 5" OVER 3" BASE	
	■					CL	▨	FILL: Dark yellowish brown mottled with dark grayish brown sandy to silty CLAY (moist, medium stiff). Trace fine to coarse gravel.	
		■	6	24.6	97		▨		
							▨	At 4'; becoming dark gray to black.	
915 5		■	4	20.7	103		▨		
		■	PUSH	25.3	96		▨		
910 10		■	10				▨		
				25.1	101	CL	▨	OLDER ALLUVIUM: Olive gray silty CLAY, trace fine to coarse gravel (moist, hard).	
	■						▨		
		■	8	21.7	95		▨	At 13'; becoming light olive brown, iron oxide staining, damp.	
	■					ML	▥	Yellowish brown fine sandy SILT, trace Clay (damp, hard).	
905 15		■	8						
				23.1	98	CL	▨	Yellowish brown to strong brown mottled with grayish brown fine sandy silty CLAY (moist, hard).	
900 20		■	5	23.1	101		▨	At 20'; dark yellowish brown to dark olive brown.	
						CL	▨	Strong brown sandy to silty CLAY, fine to coarse gravel-sized clasts (moist, hard).	
895 25		■	5	24.1	98		▨		
							▨	At 27'; mottled with grayish brown silty Clay.	
							▨	At 29'; gravel, shale fragments.	
890 30		■	8	24.5	100		▨	At 30'; micaceous, manganese oxide nodules.	
885 35		■	10	26.5	96		▨		
								Total Depth 36', No Groundwater Encountered, No Caving Observed *NOTE - KELLY BAR WEIGHTS 0'-26' = 3390# 26'-53' = 2260#	
880 40									

GORIAN & ASSOCIATES, INC.

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: B-3

Page Number: 1

Date(s) Excavated 8/13/21	Logged By DM/BC	Excavation Location See Map	Approximate Surface Elevation 922½±
Excavation Dimension 24" Dia.	Equipment Contractor Tri Valley Drilling	Equipment Type Bucket Auger	Hammer Data *SEE NOTE

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE	
	■					CL		FILL: Dark yellowish brown mottled with grayish brown silty CLAY, trace fine to coarse Sand, trace fine to coarse gravel (moist, medium stiff). Trace fine to coarse gravel.	
920		■	15	19.8	97	ML/ SM		OLDER ALLUVIUM: Yellowish brown sandy SILT to silty fine SAND, fine to medium gravel-sized clasts, fragmented shale (damp, very dense to hard).	
5	■	■	7	22.3	89	CL		Olive brown sandy to silty CLAY, fine to coarse gravel-size clasts (moist, hard).	
915	■							At 7'; increasing percentage of sand, color chaning to strong brown with gray mottling.	
						CL		Strong brown very sandy to silty CLAY, scattered fine to medium gravel-size clasts throughout (moist, hard).	
10	■	■	6	20.5	99				
910		■	6	19.1	94				
	■							At 14½'-15'; occasional small cobbles.	
15		■	11	16.8	98	CL- ML		Gradational sequences of strong brown very sandy silty CLAY grading to clayey SILT and SAND with scattered fine to medium gravel-size clasts (moist, hard).	
905									
	■							At 19'; increasing percentage of sand.	
20		■	11	18.4	103			At 20'; becoming yellowish brown to strong brown; calcium carbonate veinlets, scattered cobbles.	
900	■								
								At 23'-25'; cobbles up to 6" in diameter.	
25	■	■	8	22.8	102	CL		Brown to reddish brown very silty to sandy CLAY, trace fine to medium gravel-size clasts and occasional cobbles (moist, hard).	
895									
						ML- CL		Strong brown to yellowish brown very sandy clayey SILT to very sandy silty CLAY (moist, hard). Fine to coarse gravel-sized clasts.	
30		■	14	20.3	97				
890									
								At 34'; becomes mottled with olive brown silty Clay.	
35		■	10	26.8	92	ML		Yellowish brown sandy SILT, fine to coarse gravel-size clasts, trace Clay (very moist, hard). Iron oxide staining. At 36'; saturated, water seeping into bottom of 35' drive.	
885									
								At 38'; becoming mottled with grayish brown silty Clay.	
40		■	9	33.8	88	ML		Yellowish brown fine sandy to clayey SILT, fine to coarse gravel-size clasts and fragmented shale (saturated, hard).	

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: B-3

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
880							▥	At 42'; becoming strong brown.	
45		■	11	33.7	91		▥		
875								Total Depth 46' Groundwater Encountered at 36' No Caving Observed Downhole logged to 33' *NOTE - KELLY BAR WEIGHTS 0'-26' = 3390# 26'-53' = 2260#	
50									
870									
55									
865									
60									
860									
65									
855									
70									
850									
75									
845									
80									
840									
85									

GORIAN & ASSOCIATES, INC.

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: B-4

Page Number: 1

Date(s) Excavated 8/12/2021	Logged By DM/BC	Excavation Location See Geotechnical Map	Approximate Surface Elevation 927½±
Excavation Dimension 24" Dia.	Equipment Contractor Tri Valley Drilling	Equipment Type Bucket Auger	Hammer Data *SEE NOTE

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALTIC CONCRETE: 3" OVER 8" BASE	
925		■	6	28.3	94	CL	▨	FILL: Yellowish brown sandy to silty CLAY, trace fine to coarse gravel (moist, medium stiff).	
	■								
		■	5	21.1	97	CL-ML	▨	OLDER ALLUVIUM: Dark yellowish browm to brown sandy silty CLAYS gradationally interlayered with clayey SILT and fine sand (moist, stiff). Scattered to abundant angular to subangular shale, siltstone, sandstone, and volcanic fragments throughout and as local concentrations.	
5	■								
		■	5	19.1	97			At 4'; matrix supported clasts, pebble to coarse gravel size shale and siltstone.	
920								At 6'; brownish yellow to dark yellowish brown. At 7'-8½'; local cobble zone.	
	■								
10								At 10'; becoming sandier with only scattered rocks.	
		■	8	23.3	96				
915								At 11½'; minor light olive brown mottling.	
								At 13'; local cobbles up to 9".	
15		■	7	21.9	99				
910									
								At 19'; local cobbly zone.	
20	■	■	8	21.8	96				
905								At 22'; local cobbles up to 9".	
25		■	9	23.2	100			At 25'; increasing percentage of subangular cobbles and siltstone, angular volcanics.	
900								Total Depth 26' No Groundwater Encountered. No Caving Observed. Downhole logged to 24' *NOTE - KELLY BAR WEIGHTS 0'-26' = 3390#	
30									
895									
35									
890									
40									

GORIAN & ASSOCIATES, INC.

Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: B-5

Date(s) Excavated 8/12/2021	Logged By DM/BC	Excavation Location See Geotechnical Map	Approximate Surface Elevation 929½±
Excavation Dimension 24" Dia.	Equipment Contractor Try Valley Drilling	Equipment Type Bucket Auger	Hammer Data *SEE NOTE

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALTIC CONCRETE: 4" over 3" base	
	■					CL	▨	FILL: Yellowish brown sandy to silty CLAY, trace fine gravel (moist, medium stiff)	
		■	6	28.3	90		▒	MODELO FORMATION: Light yellowish brown fine sandy SILTSTONE interbedded with pale yellow silty fine-grained SANDSTONE (moist). Iron oxide staining. Slightly weathered but highly fractured. Thinly bedded.	
925 / 5	■	■	11	23.3	96		▒		ATTITUDE ON BEDDING At 5½'; N55°E/9°SE
920 / 10		■	8/9"	DIST	DIST		▒	Light olive brown and pale yellow thinly bedded SILTSTONE, minor SANDSTONE (moist). Becoming well-bedded, platy. Common iron oxide staining. Becomes slightly less fractured with depth.	At 10'; N9°E/4°-5°SE
915	■						▒		
15		■	7/6"	19.0	88		▒		At 17'; N15°E/6°SE
910 / 20		■	10/ 9"	21.4	88		▒		At 21'; N5°W/7°NE
905 / 25		■	7	23.7	101		░	Pale olive silty fine-grained SANDSTONE (moist). Micaceous.	
900 / 30								Total Depth 26' No Groundwater Encountered No Caving Observed Downhole logged to 25' *NOTE - KELLY BAR WEIGHTS 0'-26' = 3390# 26'-53' = 2260#	
895 / 35									
890 / 40									

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Project: imt Residential
325 Hampshire Road, Westlake Village

Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation
Number: IB-1

Page Number: 1

Date(s) Excavated 7/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 923±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE 4" OVER 4" BASE	
	■					CL	▨	FILL: Very dark grayish brown silty CLAY with fine to coarse Sand (moist, medium stiff).	
920		■	15	24.1	100		▨		
5		■	18	28.8	87		▨	At 5½'; some fine to coarse gravel.	
						ML	▥	OLDER ALLUVIUM: Strong brown clayey SILT with fine to coarse gravel, trace fine gravel (moist, hard). Iron oxide staining, carbon nodules.	
915		■	42	23.5	95		▥		
10		■	58	23.2	97		▥	At 10'; becoming reddish brown to strong brown.	
910							▥		
15		■	44	23.8	90		▥		
905							▥		
20		■	71	26.5	93		▥	At 20'; becoming mottled with light olive brown fine sandy Silt.	
900							▥		
25		◤	8/13/ 18				▥		
895							▥		
30		■	56	29.0	93		▥	At 30'; becoming very moist.	
890							▥		
35		◤	5/8/ 13				▥		
885							▥		
40		■	29	23.0	100		▥	At 40'; saturated.	

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Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
880							▥		
45		◩	11/ 16/ 23				▥		
875							▥		
50		■	67	31.9	93		▥		
								Total Depth 51' Groundwater Encountered at 40', measured at 33' after 5 minutes, at 29' after conclusion of exploration, at 26' after adding 4 bags of Sand. On 7/14 at 6:50 AM, water at 27.8' No Caving Observed Boring backfilled to 19' Boring converted into infiltration test well. -19' to -18'; medium bentonite chips -18' to -11'; 2" Slotted (.020) Schedule 40 PVC -11' to -1'; 2" Blank PVC ANNULAR SPACE -18' to -11'; #3 Sand -11' to -9'; bentonite chips Hydrated	
870									
55									
865									
60									
860									
65									
855									
70									
850									
75									
845									
80									
840									
85									

GORIAN & ASSOCIATES, INC.

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SUBSURFACE LOG

Excavation Number: IB-2

Page Number: 1

Date(s) Excavated 7/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 922±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE 4" OVER 4" BASE	
920	■					CL	▨	FILL: Vary dark grayish brown silty CLAY, trace fine to coarse Sand (moist, medium stiff).	FC = 76%
		■	15	24.1	100		▨		
5		■	18	28.8	87		▨		
915						ML	‖	OLDER ALLUVIUM: Strong brown clayey SILT, trace fine Sand (damp to moist, hard). Few calcium carbonate veinlets, carbon nodules.	
		■	42	23.5	95		‖		
10		■	58	23.2	97		‖		
910 15 905 20 900 25 895 30 890 35 885 40								Total Depth 11' No Groundwater Encountered No Caving Observed Boring converted into infiltration test well. -11' to -10'; medium bentonite chips -10' to -5'; 2" Slotted (.020) Schedule 40 PVC -5' to +3"; 2" Blank PVC ANNULAR SPACE -10' to -5'; #3 Sand Hydrated	

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SUBSURFACE LOG

Excavation Number: IB-3

Page Number: 1

Date(s) Excavated 7/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 912½±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE 4" OVER 4" BASE	
910						CL	▨	FILL: Very dark grayish brown silty CLAY, trace fine to coarse Sand (moist, medium stiff).	
						ML	▥	OLDER ALLUVIUM: Olive brown clayey SILT, some fine to coarse Sand, fine gravel (moist, stiff).	
5		■	27	24.2	96				
905		■	53	23.1	95	ML	▥	Strong brown clayey SILT, trace fine to coarse Sand, trace fine gravel (moist, hard). Iron oxide staining, carbon nodules.	
10		■	33	23.7	95				
900		■	47	23.7	100				
						CL-ML	▨	Reddish brown to strong brown sandy to silty CLAY mottled with olive brown fine sandy SILT, trace fine gravel (moist, hard). Carbon nodules.	
15		■	60	20.8	100				
895		■	73	22.4	99			Increased percentages of sand, locally.	
20		■	61	23.2	99				
890									
25		◤	5/10/17						FC = 60%
885									
30		■	53	20.8	101				
880									
35		◤	4/5/5					At 35'; saturated; becoming very clayey, stiff.	
875									
40		■	9	29.8	92				

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SUBSURFACE LOG

Excavation
Number: IB-3

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
870							▨		
45		◣	3/3/8				▨		
865							▨		
50		☒	33	--	--		▨		
860								Total Depth 51' Groundwater Encountered at 35'. Measured at 26'. No Caving Observed On 7/14 at 6:55AM - water at 20.9' Backfilled with boring cuttings, topped with AC patch.	
55									
855									
60									
850									
65									
845									
70									
840									
75									
835									
80									
830									
85									

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SUBSURFACE LOG

Excavation Number: IB-4

Page Number: 1

Date(s) Excavated 7/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 911±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE 4" OVER 4" BASE	
910	■	■	14	25.8	95	CL	▨	FILL: Very dark grayish brown silty CLAY, trace fine to coarse Sand (moist, medium stiff).	
5		■	30	23.0	97	ML	▥	OLDER ALLUVIUM: Olive brown to yellowish brown clayey SILT, trace fine to coarse Sand, fine gravel (moist, stiff).	
905		■	59	22.0	99	ML	▥	Strong brown clayey SILT, trace fine Sand, trace fine to coarse gravel (moist, hard). Carbon nodules.	
10 / 900								Total Depth 11' No Groundwater Encountered No Caving Observed Boring converted into infiltration test well -11' to -10'; medium bentonite chips -10' to -5'; 2" Slotted (.020) Schedule 40 PVC -5' to +2"; 2" Blank PVC ANNULAR SPACE 10' to -5'; #3 Sand -5' to -4'; medium bentonite chips Hydrated	
15 / 895									
20 / 890									
25 / 885									
30 / 880									
35 / 875									
40 / 870									

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SUBSURFACE LOG

Excavation
Number: IB-5

Page Number: 1

Date(s) Excavated 7/13/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 928±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							▇	ASPHALT CONCRETE 4" OVER 4" BASE	
	■					ML	▥	FILL: Olive brown to yellowish brown sandy to silty CLAY, trace fine gravel (moist, medium stiff).	FC = 68%
		■	14	25.8	95		▥		
925						CL	▨	Vary dark grayish brown silty CLAY, trace fine to coarse SAND (moist, stiff).	
		■	30	23.0	97		▨		
5							▨		
920		■	59	22.0	99		▨		
						ML	▥	OLDER ALLUVIUM: Light olive brown sandy SILT, trace fine to coarse gravel, trace CLAY (damp to moist, hard). Iron oxide staining.	
10							▥		
								Total Depth 11' No Groundwater Encountered No Caving Observed Boring converted into infiltration test well -11' to -10'; medium bentonite chips -10' to -5'; 2" Slotted(.020) Schedule 40 PVC -5' to +3/4"; 2" Blank PVC ANNULAR SPACE 10' to -5'; #3 Sand -5' to -4'; medium bentonite chips Hydrated	
915									
15									
910									
20									
905									
25									
900									
30									
895									
35									
890									
40									

GORIAN & ASSOCIATES, INC.

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SUBSURFACE LOG

Excavation
Number: IB-6

Page Number: 1

Date(s) Excavated 7/14/21	Logged By DM	Excavation Location See Map	Approximate Surface Elevation 929±
Excavation Dimension 8" Dia.	Equipment Contractor 2R Drilling, Inc.	Equipment Type Hollow Stem Auger	Hammer Data 140#, 30" Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0							■	ASPHALT CONCRETE: 2½" OVER 2" BASE	
	■					CL	▨	FILL: Dark yellowish to dark brown silty CLAY with fine to coarse Sand, fine gravel (moist, stiff).	
		■	21	22.7	98				
925						CL	▨	Dark yellowish brown to dark brown silty CLAY with fine to coarse Sand, fine to coarse gravel (moist, stiff).	
5		☒	39	--	--				
						CL	▨	Very dark grayish brown silty CLAY with fine to coarse gravel, trace fine Sand (moist, stiff). carbonate veinlets.	
		☒	31						
		■	23	22.3	99				
920						CL	▨	Dark grayish brown silty CLAY with fine to coarse gravel (moist, stiff).	
10		■	73			CL	▨	Very dark grayish brown silty CLAY with fine to coarse gravel (moist, stiff).	
				18.9	103	ML/ CL	▩	OLDER ALLUVIUM: Light olive brown fine sandy to clayey SILT mottled with dark grayish brown silty CLAY, with fine to coarse gravel (moist, hard).	
		■	90	19.4	100				
						ML	▥	Yellowish brown to light strong brown clayey SILT with fine gravel (moist, hard).	
915						CL	▨	Strong brown silty CLAY with fine to coarse sand, trace fine gravel (moist, hard).	
15		■	43	21.8	99				
						CL-ML	▩	Strong brown very silty CLAY to clayey SILT with fine to coarse sand, fine to coarse gravel (moist, hard). Carbon nodules.	
		■	76	14.7	105				
910									
20		■	69	25.9	92				
905									
25		◩	6/16/ 28						
						CL-ML	▩	Strong brown sandy to clayey SILT mottled with dark grayish brown silty CLAY, trace fine gravel (moist, hard). Carbon nodules.	
900									
30		■	80	22.7	99				
895									
35		◩	10/ 15/ 29						
						CL	▨	Strong brown to reddish brown silty CLAY with fine to coarse Sand, fine gravel (moist, hard).	
890									
40		■	64	30.7	92				

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Work Order: 3196-0-0-100

SUBSURFACE LOG

Excavation Number: IB-6

Page Number: 2

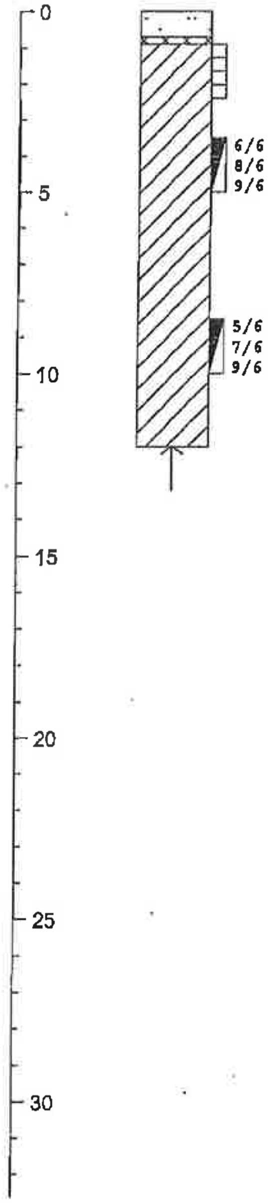
Elevation / Depth (ft.)		Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
885								▨		
	45		◩	15/ 26/ 28				▨	At 45'; saturated; fine to coarse gravel, calcium carbonate veinlets.	
880							ML	▥	Strong brown to dark yellowish brown fine sandy to very clayey SILT, trace medium to coarse sand, trace fine to coarse gravel (saturated, hard).	
	50		■	80/ 11"	19.7	109		▥		
									Total Depth 51½' Groundwater Encountered at 49'5" No Caving Observed Backfilled to 46' with medium bentonite chips Boring converted into infiltration test well -46' to -22'; 2" Slotted(.020) Schedule 40 PVC -22' to '21'; medium bentonite chips -21' to -1; 2" Blank PVC ANNULAR SPACE -46' to -22'; #3 Sand Hydrated	
875										
	55									
870										
	60									
865										
	65									
860										
	70									
855										
	75									
850										
	80									
845										
	85									

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project:	Home Depot Remodel	**Project Number:** TL	D050A3.01
Location:	Thousand Oaks, CA	**Date:**	06/21/04
Logged By:	J. Thatch	**Elevation:**	N/A
Drilled By:	Pacific Drilling	**Depth to Groundwater:**	N/E
Drill Type:	Beaver Tri-Pod	**Cased to Depth:**	N/A
Auger Type:	6" O.D. Solid Flight Auger	**Hammer Type:**	140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		PCC	Portland Cement Concrete = 7.5 inches			
		FILL	At 0.7 Inches - Poorly Graded Sand, damp, fine to coarse, brown, with little gravel and trace silt		--	
		CL	At 0.9 Inches - LEAN CLAY; hard, moist, low plasticity, brown to yellowish orange, with trace gravel		17	
			AT 5 Feet - Very stiff, olive gray to dark gray, with trace silt and sand		16	
			At 8.5 Feet - Dark brown, with gravel and cobbles			
			Drill refusal at 12 Feet			

Notes:

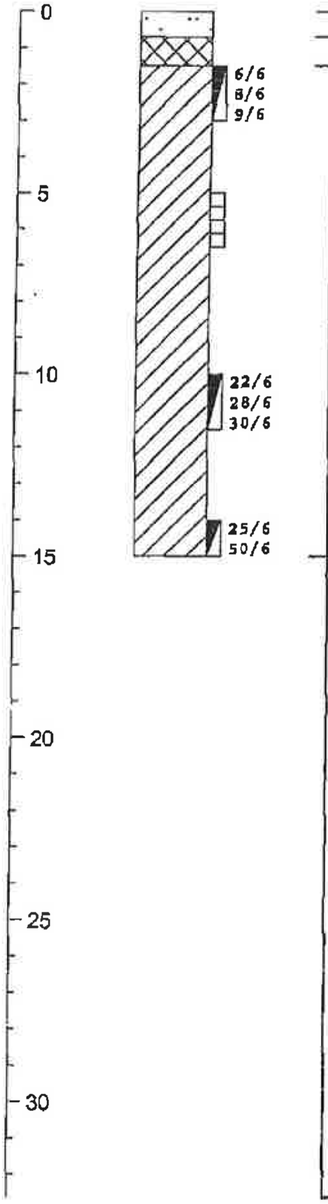
Figure Number B-1

THE TWINING LABORATORIES, INC.
EST. 1893

SOIL TEST BORING SYMBOLIC LOG

BORING B-2

Project: Home Depot Remodel
Project Number: TL D050A3.01
Location: Thousand Oaks, CA
Date: 06/21/04
Logged By: J. Thatch
Elevation: N/A
Drilled By: Pacific Drilling
Depth to Groundwater: N/E
Drill Type: Beaver Tri-Pod
Cased to Depth: N/A
Auger Type: 6" O.D. Solid Flight Auger
Hammer Type: 140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		PCC	Portland Cement Concrete = 6.5 inches			
		FILL	At 0.7 Inches - Poorly Graded Sand, damp, brown, with gravel and silt			
	6/6 8/6 9/6	CL	At 1.5 Feet - LEAN CLAY; very stiff, moist, low plasticity, brown to yellowish orange, with fine gravel		17	
5			At 3 Feet - Dark brown		-- --	
			At 6.5 Feet - Hard, light brown to red			
10	22/6 28/6 30/6				58	
15	25/6 50/6		Cemented layer		>100	
			Bottom of Boring at 15 Feet			
20						
25						
30						

Notes:

Figure Number B-2

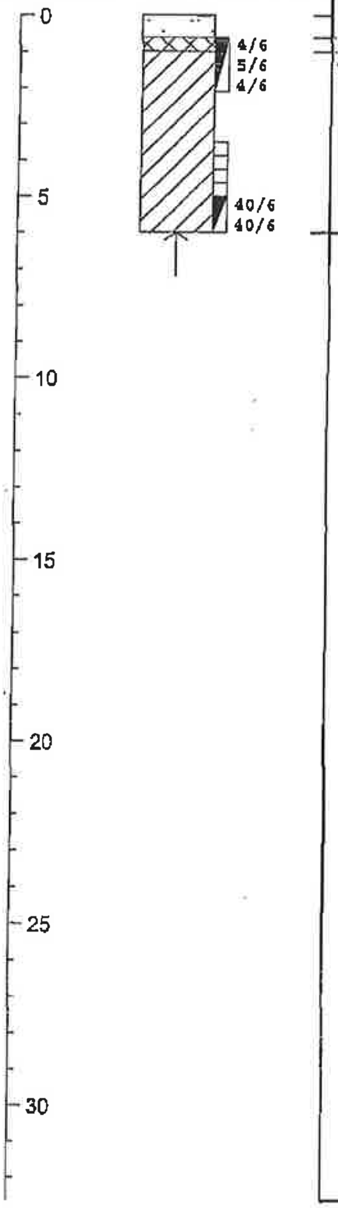
THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-3

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: J. Thatch
Drilled By: Pacific Drilling
Drill Type: Beaver Tri-Pod
Auger Type: 6" O.D. Solid Flight Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: 140 Pound Donut



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		PCC	Portland Cement Concrete = 6.5 inches			
		FILL	At 0.6 Inches - Poorly graded sand and gravel, no vapor barrier		9	
		CL	At 1 Foot - LEAN CLAY; stiff, damp, low plasticity, brown to reddish brown, with sand and gravel		--	
		CL	At 3.5 Feet - Hard, with fine gravel		>100	
			Bottom of Boring at 6 Feet			

Notes:

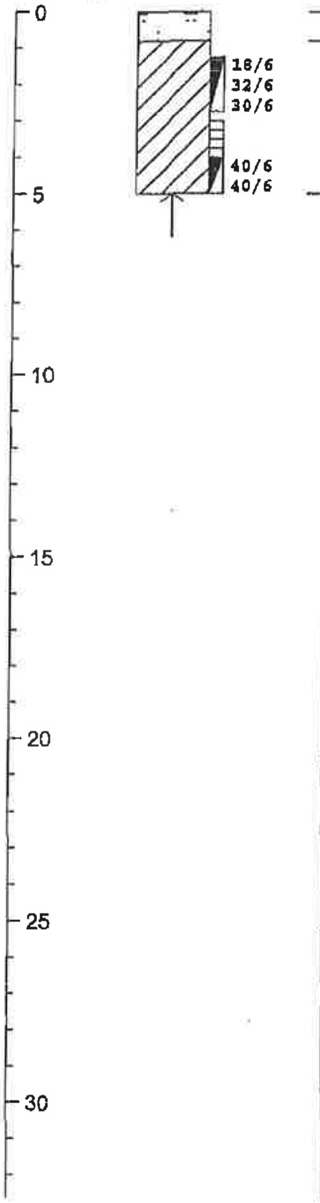
Figure Number B-3

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-4

Project: Home Depot Remodel	**Project Number: TL** D050A3.01
Location: Thousand Oaks, CA	**Date:** 06/21/04
Logged By: J. Thatch	**Elevation:** N/A
Drilled By: Pacific Drilling	**Depth to Groundwater:** N/E
Drill Type: Beaver Tri-Pod	**Cased to Depth:** N/A
Auger Type: 6" O.D. Solid Flight Auger	**Hammer Type:** 140 Pound Donut

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		PCC	Portland Cement Concrete = 6 inches			
		CL	At 0.5 Inches - Poorly Graded Sand, damp, fine to coarse, brown with little gravel		62 —	
			At 0.8 Inches - LEAN CLAY; hard, damp, low plasticity, brown to yellowish orange, trace fine gravels		>100	
			Bottom of Boring at 5 Feet			

Notes:

Figure Number B-4

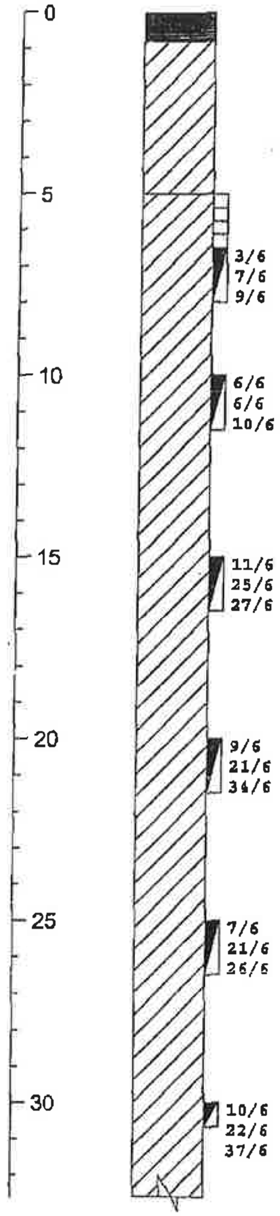
THE TWINING LABORATORIES, INC.
EST. 1899

SOIL TEST BORING SYMBOLIC LOG

BORING B-5

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		AC	Asphaltic Concrete = 3 inches			
		CL	Aggregate Base = 6 inches			
			LEAN CLAY, Sandy; very stiff, moist, low plasticity, black, with fine to medium rounded to subrounded cobbles and gravel			
5		CL			--	
	3/6 7/6 9/6				16	
			Dark brown, with fine subangular gravel			
10	6/6 6/6 10/6				16	
15	11/6 25/6 27/6		Hard, dry, light gray mottled with light reddish brown, increase in percent sand, no cobbles, with silt		52	
20	9/6 21/6 34/6		Light brown, fine subangular gravel, little to no silt		55	
25	7/6 21/6 26/6		Brown mottled with gray, no gravel		47	
30	10/6 22/6 37/6		Trace fine subangular gravel		59	

Notes:

Figure Number B-5

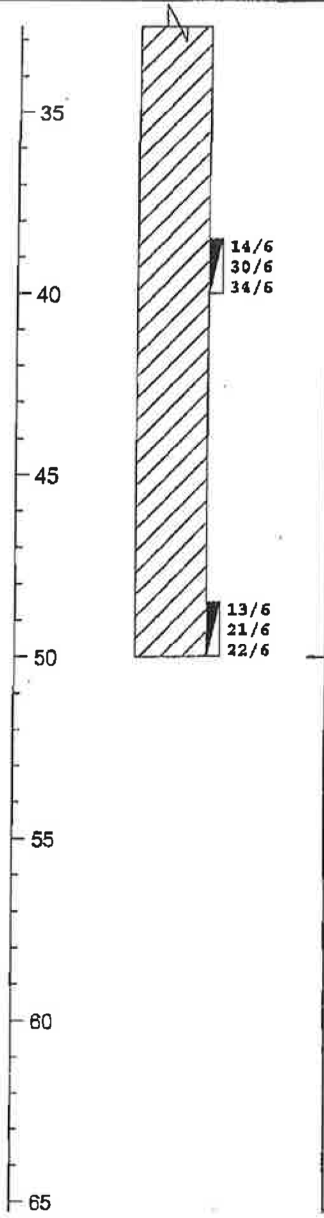
THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-5

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
	14/6 30/6 34/6		Grayish brown to brown		64	
	13/6 21/6 22/6		Damp, olive brown, trace coarse sand, trace fine gravels		43	
			Bottom of Boring at 50 Feet			

Notes:

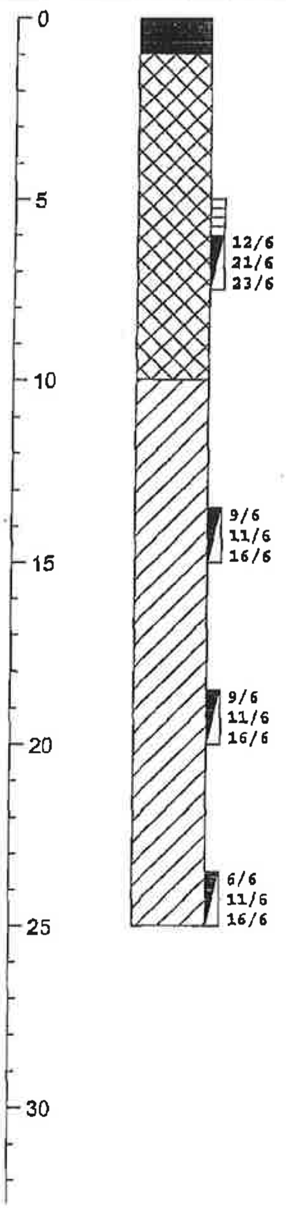
Figure Number B-5

SOIL TEST BORING SYMBOLIC LOG

BORING B-6

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		AC	Asphaltic Concrete = 4 inches			
		FILL	Aggregate Base = 7.5 inches SANDY, Silty; very dense, damp, fine to medium subrounded, trace brick debris Dense, with clay, trace fine to medium subangular gravel		-- 44	
		CL	LEAN CLAY, Sandy; hard, damp, low plasticity, light brown			
			Increase in percent sand, with fine to medium subangular gravel		27	
			Very stiff, moist, reddish brown		27	
			Olive brown with reddish brown		27	
			Bottom of Boring at 25 Feet			

Notes:

Figure Number B-6

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-7

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		AC	Asphaltic Concrete - 3 Inches			
		CL	Aggregate Base = 3 inches		–	
	12/6 20/6 22/6		LEAN CLAY, Sandy; hard, damp, low plasticity, reddish brown At 2 Feet - Trace fine subangular gravel		42	
5	11/6 19/6 26/6		Brown mottled with gray		45	
10	11/6 21/6 29/6				50	
15	14/6 50/3		Increase in fine to medium subrounded to subangular gravel		>100	
20	10/6 34/6 32/6		Gray mottled with reddish brown, weak to moderate cementation, decrease in gravel		66	
25	25/6 50/2	ROCK	Siltstone to sandstone, moderately weathered, light gray with light reddish brown		>100	
			Bottom of Boring at 25.8 Feet			
30						

Notes:

Figure Number B-7

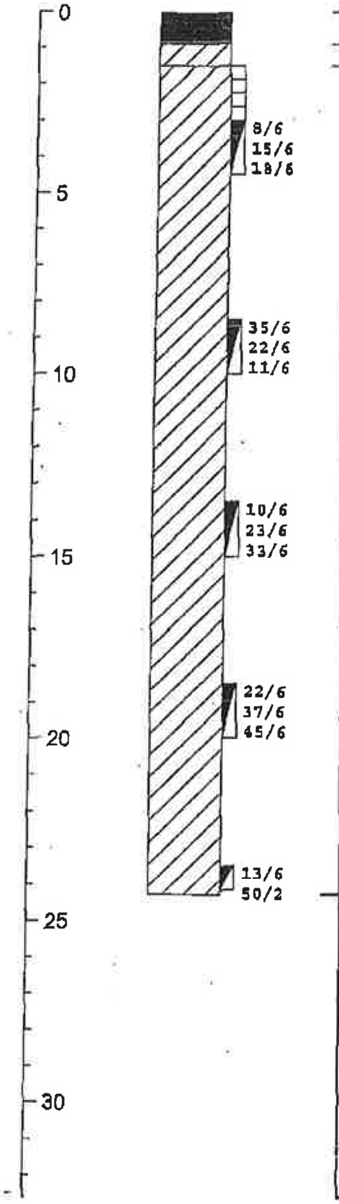
THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-8

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
		AC	Asphaltic Concrete = 4.5 inches			
		CL	Aggregate Base = 4.5 inches			
		CL	LEAN CLAY, Sandy; hard, damp, low plasticity, brown; with fine to medium subangular gravels with coarse sands		–	
	8/6 15/6 18/6				33	
			Drilling encountered dense coarse gravels			
	35/6 22/6 11/6				33	
	10/6 23/6 33/6		Reddish brown, trace coarse sands		56	
	22/6 37/6 45/6		Increase in fine to coarse angular to subangular gravel		82	
	13/6 50/2				>100	
			Bottom of Boring at 24.3 Feet			

Notes:

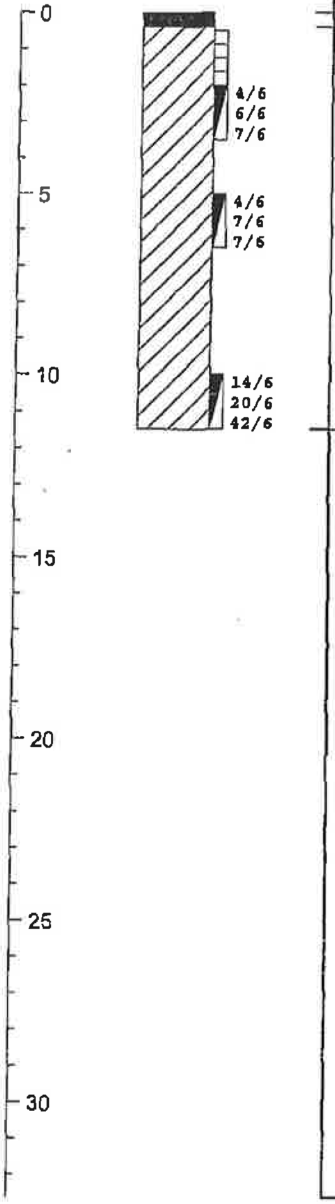
THE TWINING LABORATORIES, INC. EST. 1895

SOIL TEST BORING SYMBOLIC LOG

BORING B-9

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip



USCS	Soil Description	Remarks	N-value	Moisture Content %
AC	Asphaltic Concrete = 2 inches			
CL	Aggregate Base = 2.5 inches		–	
	LEAN CLAY, Sandy; stiff, moist, low plasticity, brown, trace fine subangular gravel		13	
	At 1.5 Feet - Dark grayish green			
	At 2.5 Feet - Dark brown to black, trace fine sands, no gravel		14	
	At 4 Feet - Trace fine subangular gravel			
	Hard, damp, grayish brown to light brown, increase in fine to medium angular gravel, increase in percent sand		62	
	Bottom of Boring at 11.5 Feet			

Notes:

Figure Number B-9

THE TWINING LABORATORIES, INC. EST. 1898

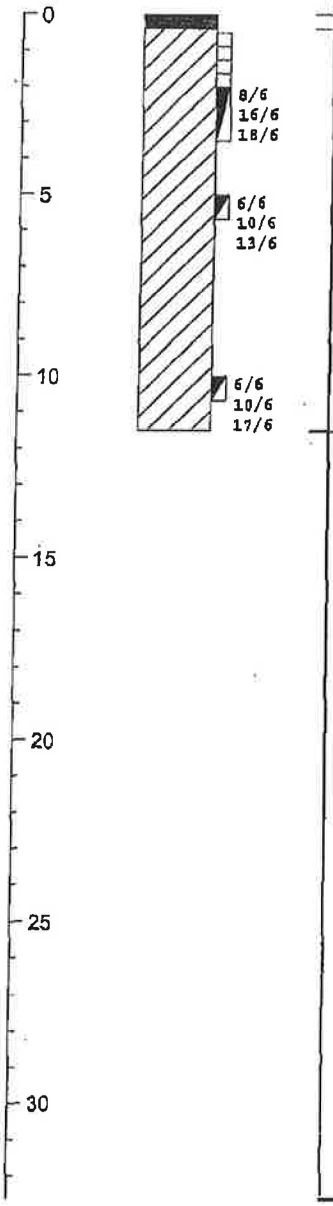
SOIL TEST BORING SYMBOLIC LOG

BORING B-10

Project: Home Depot Remodel
Location: Thousand Oaks, CA
Logged By: D. Ledgerwood
Drilled By: T. Conley
Drill Type: CME 75
Auger Type: 6 5/8" O.D. Hollow Stem Auger

Project Number: TL D050A3.01
Date: 06/21/04
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: Trip

ELEVATION/ DEPTH (feet) | **SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA**



USCS	Soil Description	Remarks	N-value	Moisture Content %
AC	Asphaltic Concrete = 4.5 inches			
CL	LEAN CLAY, hard, moist, low plasticity, brown, trace coarse sand		--	
	At 3 Feet - Light brown, with subangular fine to medium gravel		34	
	At 4 Feet - Very stiff, damp, brown to grayish brown, no gravel		23	
	Sandy, brown, trace fine to medium subangular gravel		27	
	Bottom of Boring at 11.5 Feet			

Notes:

Figure Number B-10

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
△	Portland Cement Concrete
▩	FILL
▨	LEAN CLAY (CL)
■	Asphaltic Concrete
▧	Basalt (or generic rock)
Misc. Symbols	
↑	Drill rejection

Symbol	Description
⌇	Boring continues
Soil Samplers	
▤	California Modified split barrel ring sampler
◩	Standard penetration test

Notes:

1. Test borings B-1 through B-4 were drilled on 06/21/04 using a Beaver Tri-Pod equipped with 140 Pound Hammer. Test borings B-5 through B-10 were drilled on 06/21/04 using a CME 75 equipped with Hollow Stem Auger.

2. Groundwater was not encountered during excavation of the test borings.

3. Test boring locations were located by measuring wheel with reference to the existing site features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

DD = Natural dry density
UC = Unconfined compression (psf)
-4 = Percent passing #4 sieve (%)
-200 = Percent passing #200 sieve (%)
SR = Soil resistivity (ohm-cm)
c = Cohesion (psf)
TS = Field Torvane Shear Strength test (tsf)

LL = Liquid limit (%)
PI = Plasticity index (%)
pH = Soil pH
SS = Soluble sulfates (%)
Cl = Soluble chlorides (%)
ø = Angle of internal friction (degrees)
N/A = Not applicable
N/E = None encountered

FIGURE 1.

GORIAN AND ASSOCIATES, Inc.

SUB-SURFACE DATA

Log No. B-1

PROJECT: Mr. Wade Lewis, Foothill Road

Method of Drilling: 24" Diameter Bucket Auger Logged by JQ Job No. 1607-1-10

Ground Elevation: 962'± Location: See Location Map Date Observed: 7-3-87

Depth in Feet	Classification Unified Soil System	Symbol	Undisturbed Sample	Bulk Sample	Moisture Content %	In Place Dry Density lbs./cu. ft.	DESCRIPTION	
0	CL	▨	■		19.9	80	Artificial Fill- Brown, sandy silty clay with scattered granule to gravel size clasts. Roots scattered (moist, moderately stiff).	
5	CL	▨	■ ■	■ ■	13.2 18.8	75 93	Older Alluvium- Brown to dark brown sandy silty clay with scattered granules. Rootlets and roothairs present, but mostly decayed. Very porous to 1/16" diameter to depth of 5 feet (damp to moist, hard).	
10 15		≡	■ ■	■	19.8 16.6	95 98	Modelo Formation-Sequence of greenish grey to tan, laminated to thin-bedded, fissile, silty claystone (shale) and clayey siltstone with occasional thin interbeds of orange-tan to grey fine to medium sandstone.	Attitude of Bedding N82°W/5°NE @11'
20 25 30 35							TOTAL DEPTH 16' No Caving No Groundwater	

A-6045

Figure 2.

GORIAN AND ASSOCIATES, Inc.

SUB-SURFACE DATA

Log No. B-2

PROJECT: Mr. Wade Lewis, Foothill Road

Method of Drilling: 24" Diameter Bucket Auger Logged by JQ Job No. 1607-1-10

Ground Elevation: 967'± Location: See Location Map Date Observed: 7-3-87

Depth in Feet	Classification Unified Soil System	Symbol	Undisturbed Sample	Bulk Sample	Moisture Content %	In Place Dry Density lbs./cu. ft.	DESCRIPTION	
0				■			Older Alluvium- Dark brown sandy silty clay with abundant granule to gravel size shale fragments. Rootlets and roothairs frequent. Very porous to 1/16" diameter (moist, hard).	
			■		13.9	76		
			■		17.0	82		
				■				
5	CL		■		16.8	77		
10	CL		■		16.1	80	Light brown to tan sandy silty clay (moist, hard).	
				■				
				■			Modelo Formation- Sequence of greenish grey to tan, laminated to thin bedded, fissile, silty claystone (shale) and clayey siltstone with thin to medium thick interbeds of grey to orange-tan fine to medium sandstone. Contact with older alluvium is near horizontal.	Attitude of Bedding N15°E/13°NW @ 16' N58°E/7°NW @ 17' N88°E/12°NW @17½'
15			■		18.6	92		
20			■		26.6	90		
							TOTAL DEPTH 21' No Caving No Groundwater	
25								
30								
35								

A-6045

FIGURE 3.

GORIAN AND ASSOCIATES, Inc.

SUB-SURFACE DATA

Log No. B-3

PROJECT: Mr. Wade Lewis, Foothill Road

Method of Drilling: 24" Diameter Bucket Auger Logged by JQ Job No. 1607-1-10

Ground Elevation: 969'± Location: See Location Map Date Observed: 7-3-87

Depth in Feet	Classification Unified Soil System	Symbol	Undisturbed Sample	Bulk Sample	Moisture Content %	In Place Dry Density lbs./cu. ft.	DESCRIPTION	
0		▨		■			Artificial Fill- Abundant shale fragments in tan, sandy silty clay matrix. Rootlets common. Near horizontal carbonized mat of vegetation at base, (moist, loose).	
	CL	▨	■		16.6	72		
	CL	▨	■		20.0	87		
				■			Topsoil- Dark brown sandy silty clay with tan to orange, granule to gravel size shale and sandstone fragments. Roothairs frequent; porous to 1/16" diameter (moist, hard).	
5			■		26.9	90		
							Modelo Formation- Weathered with topsoil fracture fillings to depth of 5', fresh below. Sequence of greenish grey, laminated to thin bedded fissile silty claystone (shale) and clayey siltstone, with thin to medium-thick interbeds of grey to orange-tan, fine to medium sandstone.	Attitude of Bedding N85°E/7°NW @10' N89°W/7°NE @10½'
10			■		20.2	98		
				■				
15			■		16.0	97		
							TOTAL DEPTH 16' No Caving No Groundwater	
20								
25								
30								
35								

A-6045

APPENDIX B

LABORATORY TESTING

General

Laboratory test results on selected samples are presented below. Test were performed to evaluate the physical and engineering properties of the encountered earth materials, including in-situ moisture content and dry density, maximum density-optimum moisture content relationships, expansion potential, consolidation characteristics, shear strength parameters, and grain size distribution. Soil corrosivity testing was performed under subcontract by a corrosion engineer.

Field Density and Moisture Tests

In-situ dry density and moisture content were determined from the relatively undisturbed drive samples obtained during exploratory operations. The test results and a detailed description of the earth materials encountered are shown on the attached Logs of Subsurface Data, Appendix A.

Maximum Density-Optimum Moisture

Maximum density/optimum moisture test (compaction characteristics) were performed on selected samples of the encountered materials. The tests were performed in general accordance with ASTM D 1557 test method. The results are as follows:

Sample	Visual Soil Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-3 @ 4’	Yellow brown sandy silt	101.8	19.0
B-4 @ 5’	Yellow brown sandy to clayey silt	103.9	18.2

Soil Expansion Test

Expansion Index tests were performed on selected bulk samples of the encountered materials. The results are as follows:

Sample	Expansion Index	Expansion Index Range	Expansion Potential
B-3 @ 1’	83	51 - 90	Medium
IB-6 @ 1’	65	51 - 90	Medium

Direct Shear Tests

Strain controlled direct shear testing was performed on five relatively undisturbed samples and two remolded samples. The sample sets were saturated prior to shearing under axial loads ranging from 920 to 3,680 psf. The shear strength results are presented as graphic summaries.

Load Consolidation Tests

Load consolidation tests were performed on three relatively undisturbed sample. Test loads were added in increments to a maximum of 8,000 psf. Water was added at a load approximating existing overburden stresses to study the effect of moisture infiltration on potential consolidation behavior. The results are presented as graphic summaries.

Grain Size Distribution

Grain size distribution analyses were performed on several bulk samples. The grain size was evaluated by hydrometer analysis. Hydrometer analyses were performed using an approximately 50-gram sample. The results are presented on the attached Logs of Subsurface Data, Appendix A.

Soil Corrosivity

The results of the analytical laboratory testing to evaluate the potential for soil corrosion are presented in this Appendix. The testing was performed on two soil samples considered to represent the site soils. From ACI Table 19.3.1.1 the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1, below. The site soils are considered severely corrosive to metals as determined from Table 1. For specific recommendations a corrosion engineer should be consulted.

ACI Table 19.3.1.1 – Exposure Categories and Classes

Category	Class	Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass	Dissolved sulfate (SO_4^{2-}) in water, ppm[1]
Sulfate (S)	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} < 2.00$	$1500 \leq SO_4^{2-} < 10,000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10,000$

1 ppm (parts per million) = milligrams per kilogram mg/kg of dry soil weight

ACI Table 19.3.2.1 – Requirements for Concrete by Exposure Class

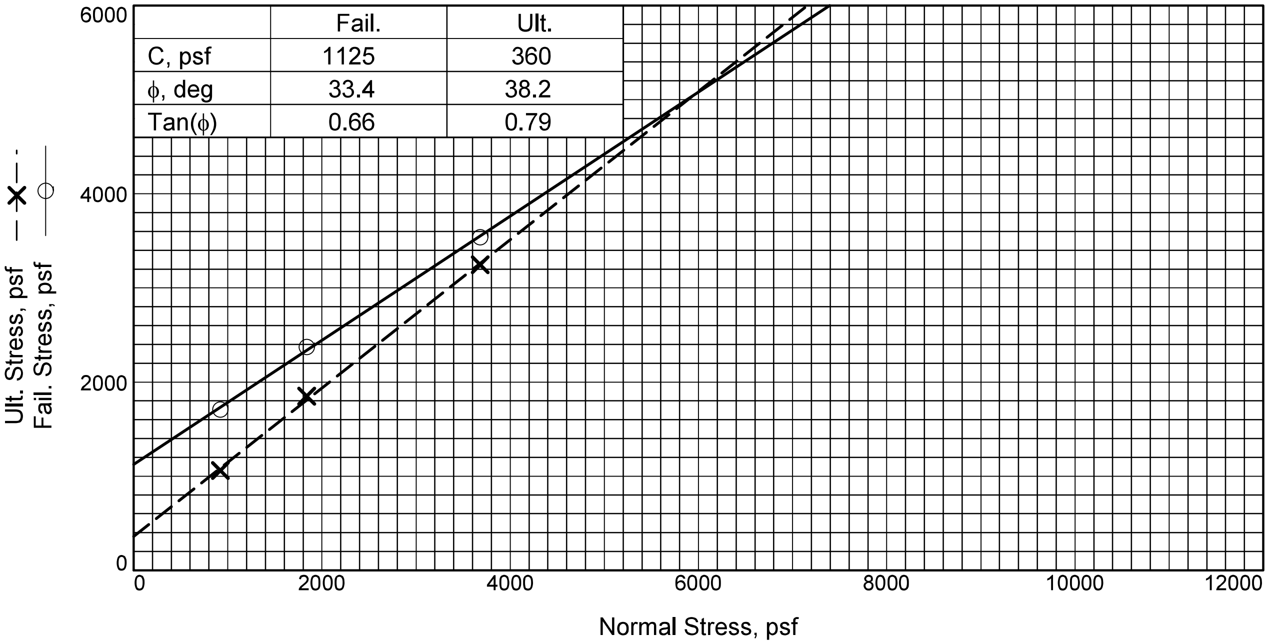
Exposure Class	Maximum *w/cm*	Minimum f_c', psi	Cementitious materials - Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II	Types IP, IS, or IT with (MS) designation	MS	No restriction
S2	0.45	4500	V	Types IP, IS, or IT with (MS) designation	HS	Not permitted
S3	0.45	4500	V plus pozzolan or slag cement	Types IP, IS, or IT with (MS) designation plus pozzolan or slag cement	HS plus pozzolan or slab cement	Not permitted

ACI Tables 19.3.1.1 and 19.3.2.1 - ACI 318-14 Building Code Requirements for Structural Concrete

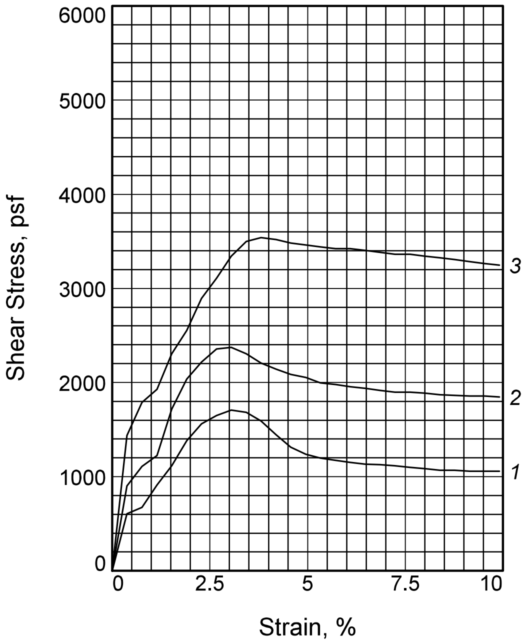
Table 1. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity, ohm-cm	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >10,000	Very mildly corrosive

F. O. Waters, Soil Resistivity Measurements for Corrosion Control, Corrosion. 1952, Vol, No. 12, 1952, p. 407.



	Fail.	Ult.
C, psf	1125	360
ϕ, deg	33.4	38.2
Tan(ϕ)	0.66	0.79



Sample No.		1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	33.2	33.2	33.2
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
Normal Stress, psf		920	1840	3680
Fail. Stress, psf		1708	2373	3540
Strain, %		3.0	3.0	3.8
Ult. Stress, psf		1059	1848	3246
Strain, %		9.9	9.9	9.9
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Relatively Undisturbed
Description: OLDER ALLUVIUM:
Clayey SILT w/ fine to coarse sand

Specific Gravity=
Remarks: 7/29/21

Figure B.1

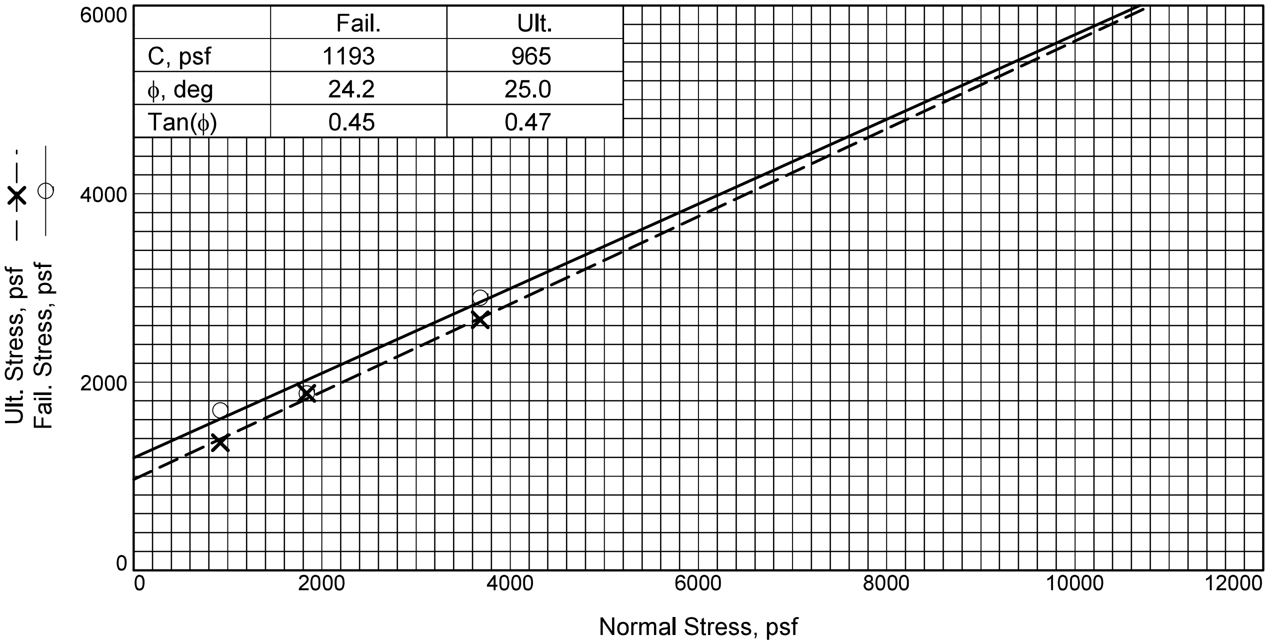
Client: imt Residential

Project: 325 Hampshire Rd

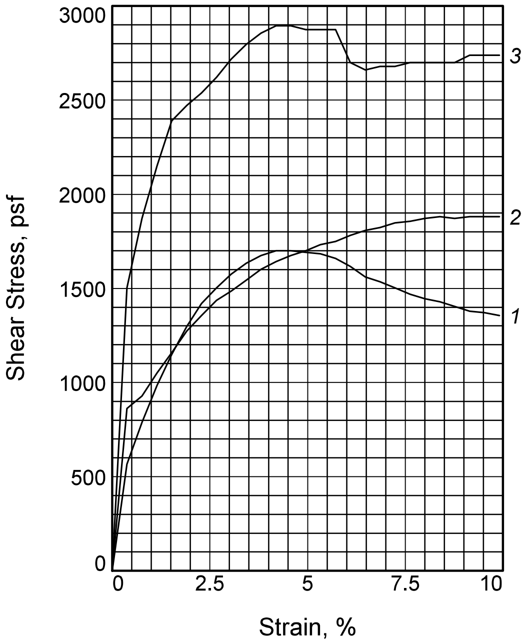
Location: B-1
Depth: 10'
Proj. No.: 3196-0-0 **Date Sampled:** 7/14/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CA



	Fail.	Ult.
C, psf	1193	965
ϕ, deg	24.2	25.0
Tan(ϕ)	0.45	0.47



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	29.2	29.2	29.2
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	1700	1880	2894
	Strain, %	4.6	9.9	4.6
	Ult. Stress, psf	1355	1880	2660
	Strain, %	9.9	9.9	6.5
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Relatively Undisturbed
Description: OLDER ALLUVIUM:
Sandy to silty CLAY

Specific Gravity=
Remarks: 8/23/21

Figure B.2

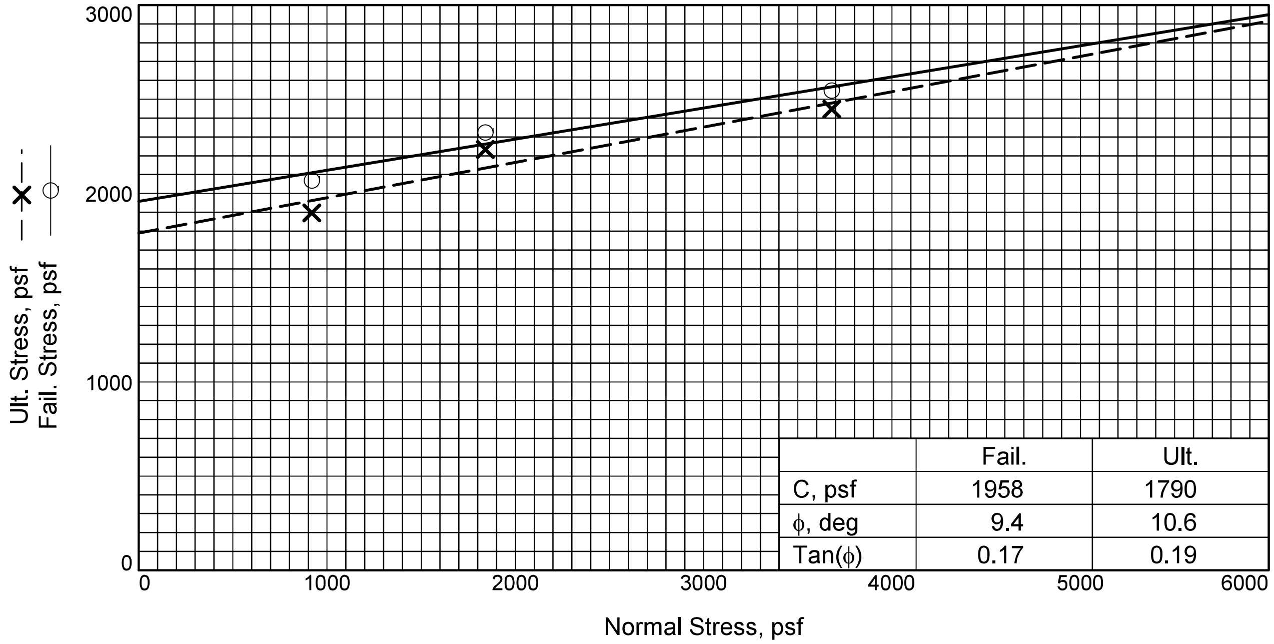
Client: imt Residential

Project: 325 Hampshire Rd

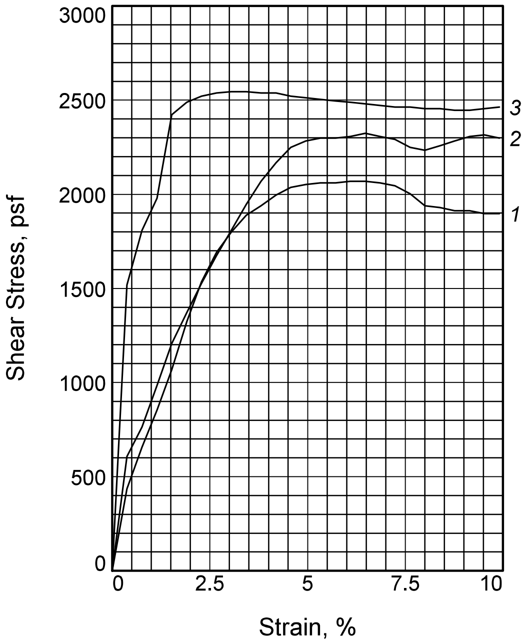
Location: B-3
Depth: 12.5
Proj. No.: 3196-0-0 **Date Sampled:** 8/13/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CD



	Fail.	Ult.
C, psf	1958	1790
ϕ, deg	9.4	10.6
Tan(ϕ)	0.17	0.19



Sample No.		1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	29.5	29.5	29.5
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
Normal Stress, psf		920	1840	3680
Fail. Stress, psf		2069	2324	2545
Strain, %		6.5	6.5	3.4
Ult. Stress, psf		1897	2233	2447
Strain, %		9.9	8.0	9.1
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Relatively Undisturbed
Description: MODELO FORMATION:
Fine sandy SILTSTONE

Specific Gravity=
Remarks: 8/24/21

Figure B.3

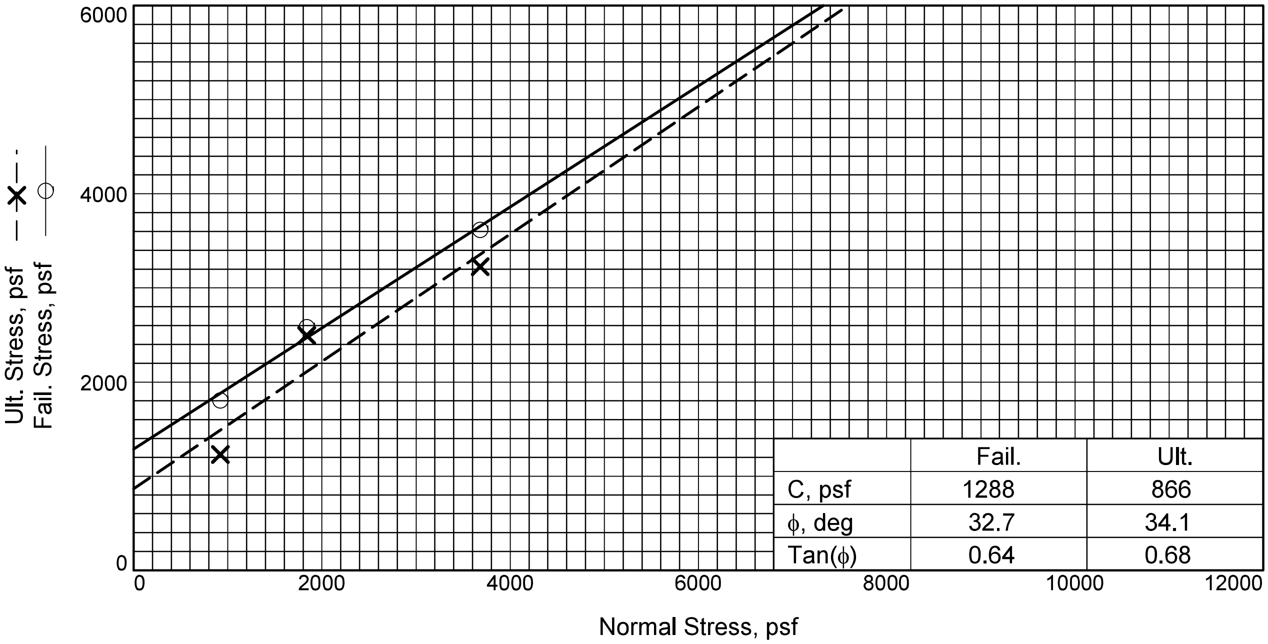
Client: imt Residential

Project: 325 Hampshire Rd

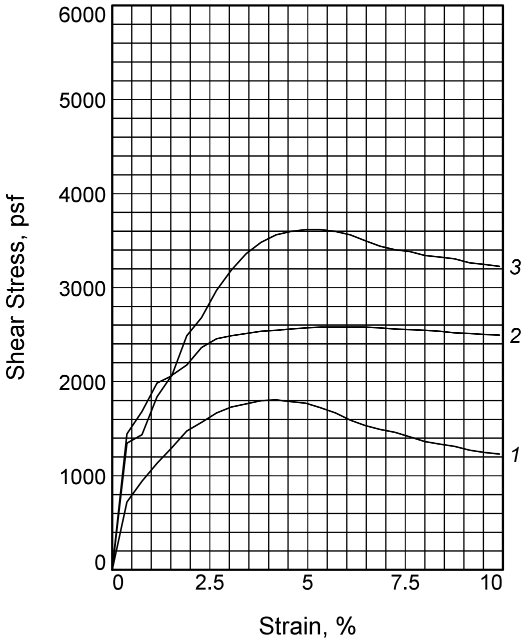
Location: B-5
Depth: 5.0
Proj. No.: 3196-0-0 **Date Sampled:** 8/12/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CD



	Fail.	Ult.
C, psf	1288	866
ϕ, deg	32.7	34.1
Tan(ϕ)	0.64	0.68



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	29.2	29.2	29.2
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	1806	2582	3618
	Strain, %	4.2	6.5	5.3
	Ult. Stress, psf	1232	2496	3227
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Relatively Undisturbed

Description: OLDER ALLUVIUM:
Sandy to clayey SILT

Specific Gravity=

Remarks: 7/29/21

Figure B.4

Client: imt Residential

Project: 325 Hampshire Rd

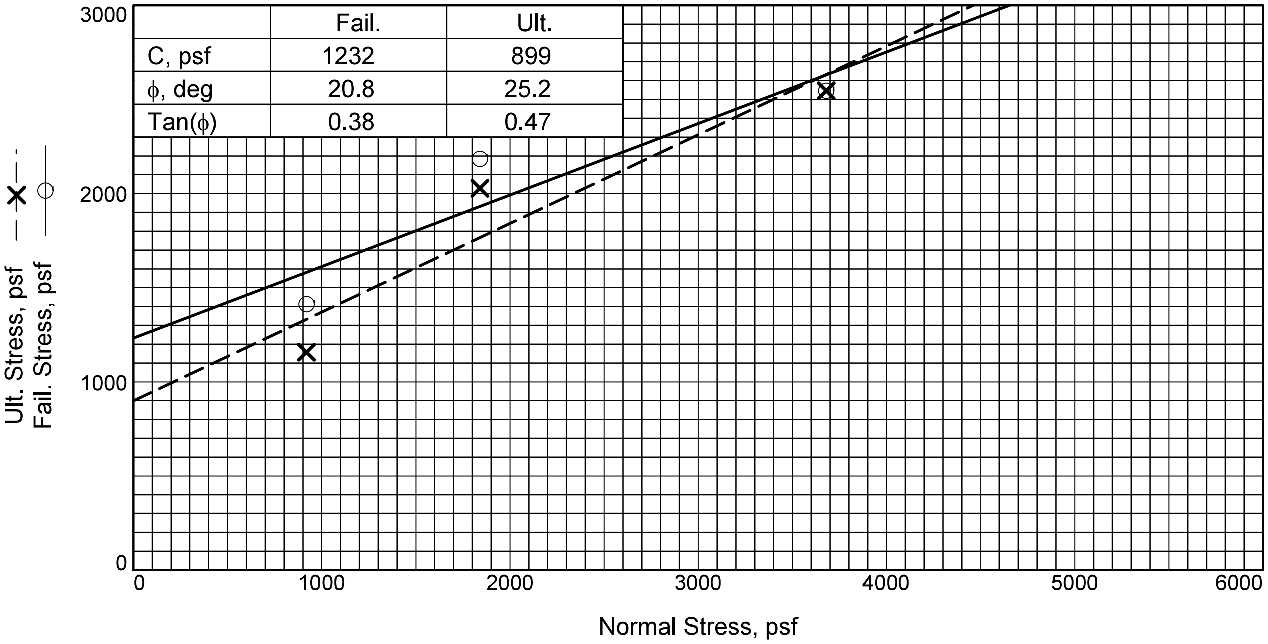
Location: IB-3

Depth: 12.5'

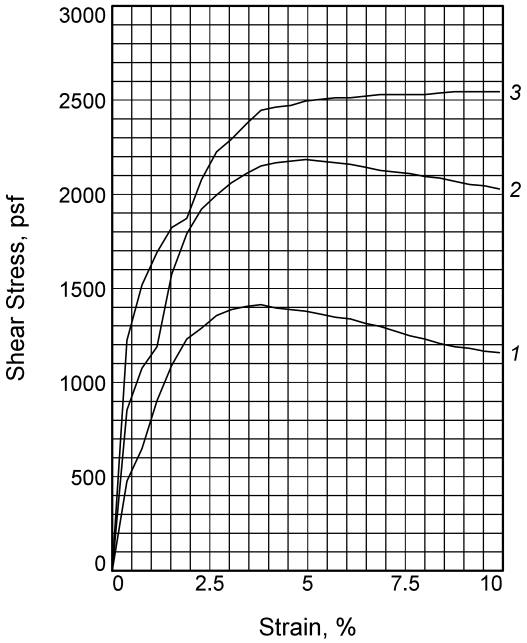
Proj. No.: 3196-0-0 **Date Sampled:** 7/13/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CA



	Fail.	Ult.
C, psf	1232	899
φ, deg	20.8	25.2
Tan(φ)	0.38	0.47



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	31.6	31.6	31.6
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	1412	2184	2545
	Strain, %	3.8	5.0	9.9
	Ult. Stress, psf	1158	2028	2545
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Relatively Undisturbed
Description: FILL:
Silty CLAY

Specific Gravity=
Remarks: 7/29/21

Figure B.5

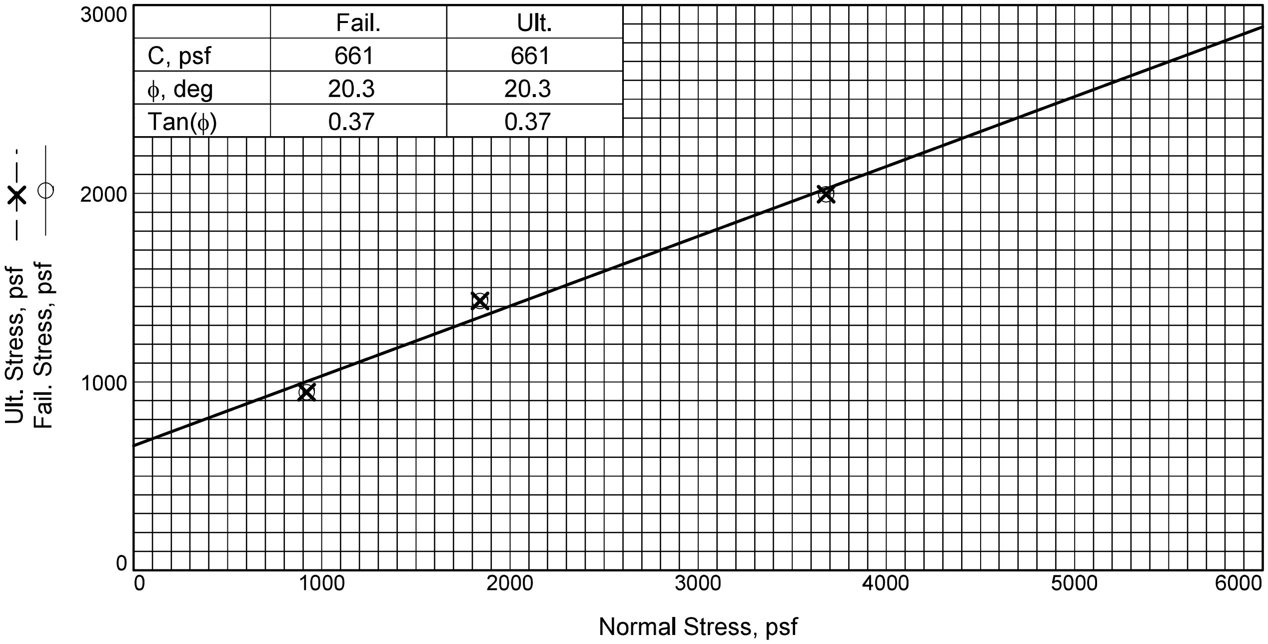
Client: imt Residential

Project: 325 Hampshire Rd

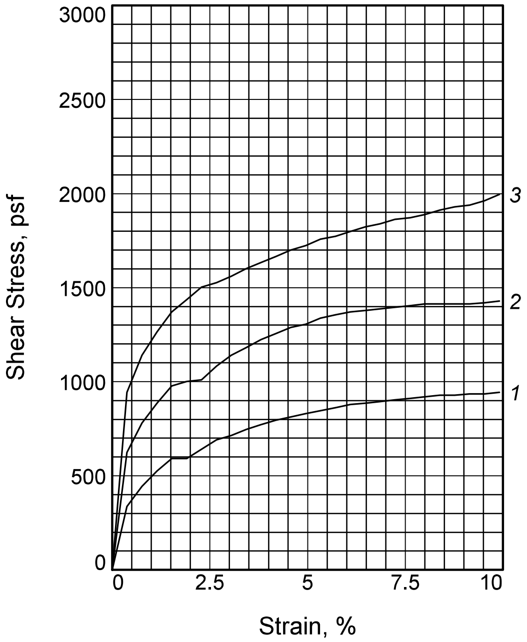
Location: IB-5
Depth: 8'
Proj. No.: 3196-0-0 **Date Sampled:** 7/13/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CA



	Fail.	Ult.
C, psf	661	661
ϕ, deg	20.3	20.3
Tan(ϕ)	0.37	0.37



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	30.7	30.7	30.7
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	944	1429	1995
	Strain, %	9.9	9.9	9.9
	Ult. Stress, psf	944	1429	1995
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Remolded

Description: OLDER ALLUVIUM:
Sandy SILT

Specific Gravity=

Remarks: 8/25/21

Figure B.6

Client: imt Residential

Project: 325 Hampshire Rd

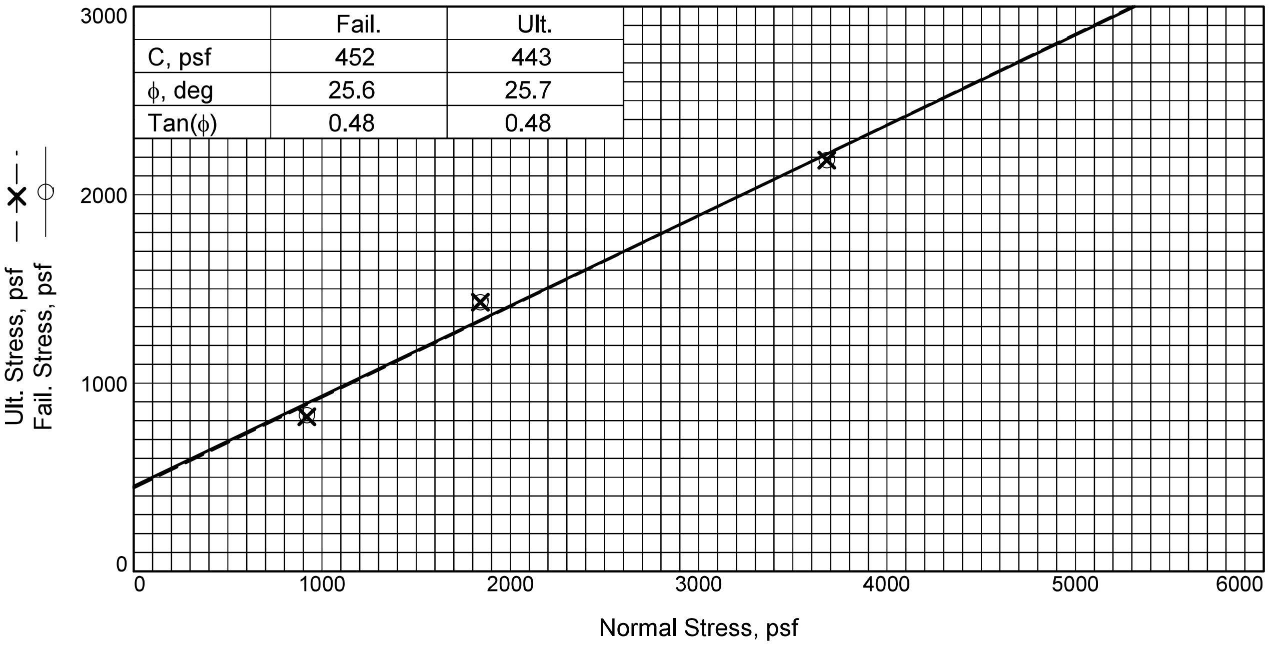
Location: B-3

Depth: 4'

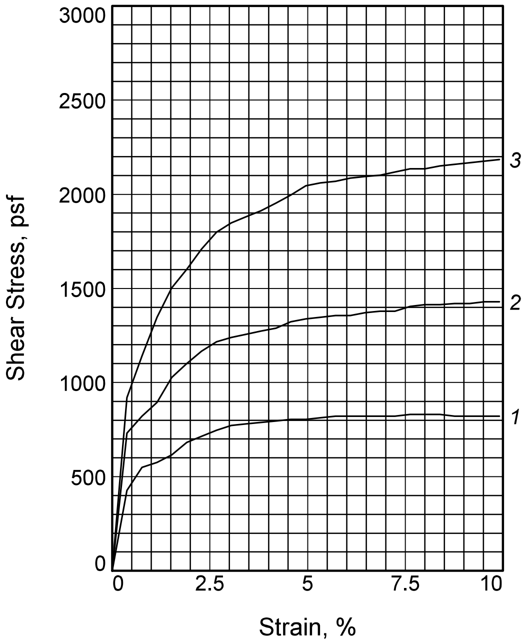
Proj. No.: 3196-0-0 **Date Sampled:** 8/13/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CA



	Fail.	Ult.
C, psf	452	443
ϕ, deg	25.6	25.7
Tan(ϕ)	0.48	0.48



	Sample No.	1	2	3
Initial	Water Content, %	N/A	N/A	N/A
	Dry Density, pcf	N/A	N/A	N/A
	Saturation, %	N/A	N/A	N/A
	Void Ratio	N/A	N/A	N/A
	Diameter, in.	2.63	2.63	2.63
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	30.5	30.5	30.5
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
	Diameter, in.			
	Height, in.			
	Normal Stress, psf	920	1840	3680
	Fail. Stress, psf	829	1429	2184
	Strain, %	8.4	9.9	9.9
	Ult. Stress, psf	821	1429	2184
	Strain, %	9.9	9.9	9.9
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Remolded
Description: OLDER ALLUVIUM:
Sandy to clayey SILT

Specific Gravity=
Remarks: 8/25/21

Figure B.7

Client: imt Residential

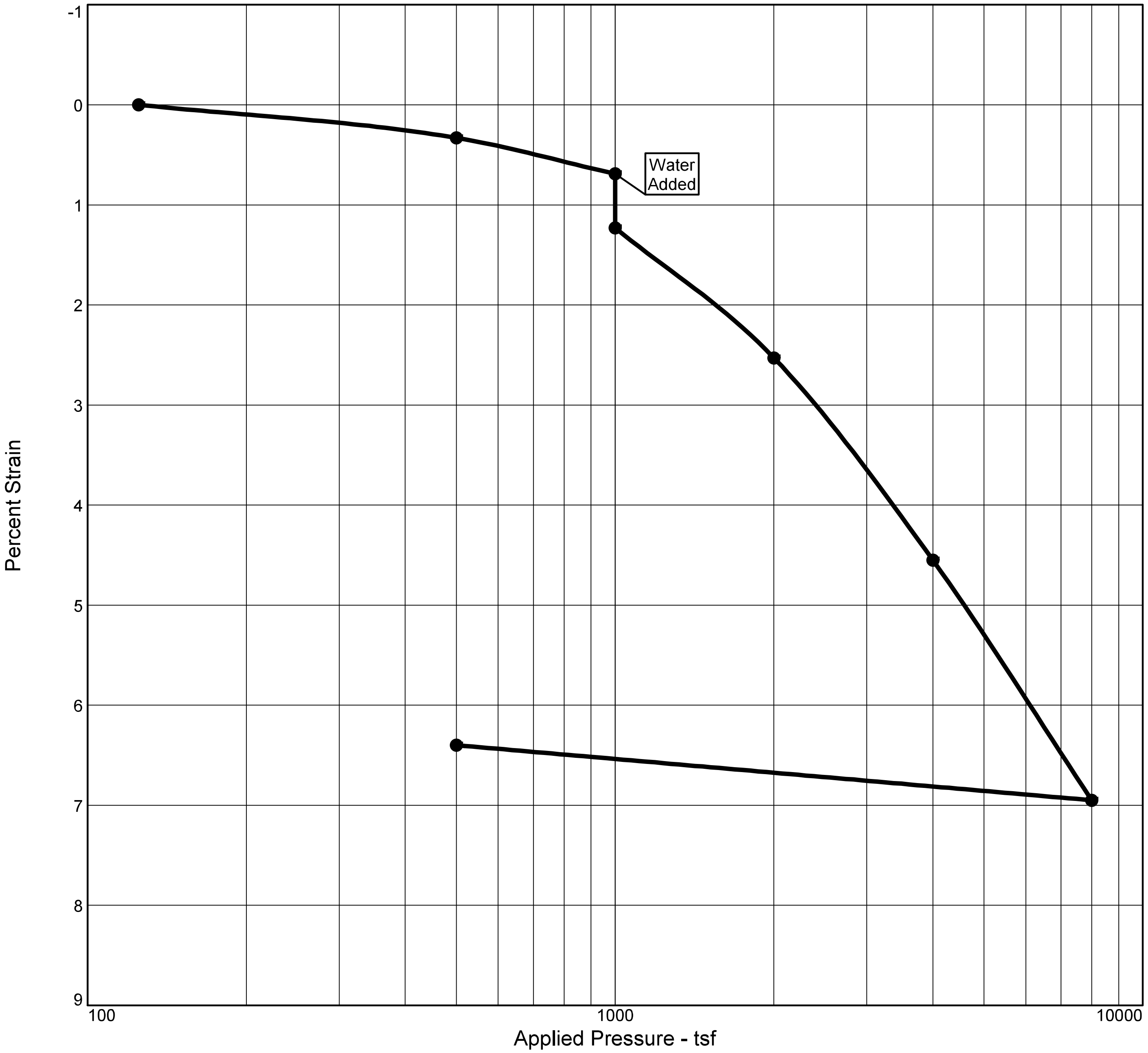
Project: 325 Hampshire Rd

Location: B-4
Depth: 5.0'
Proj. No.: 3196-0-0 **Date Sampled:** 8/12/21

DIRECT SHEAR TEST REPORT
Gorian & Associates
Thousand Oaks, CA

Tested By: CA

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Clpse. %	e_o
Sat.	Moist.											
							2398.0				0.5	

MATERIAL DESCRIPTION	USCS	AASHTO
OLDER ALLUVIUM: Silty CLAY		

Project No. 3196-0-0 **Client:** imt Residential

Project: 325 Hampshire Rd

Location: B-2 **Depth:** 12.5

Remarks:
8/27/21

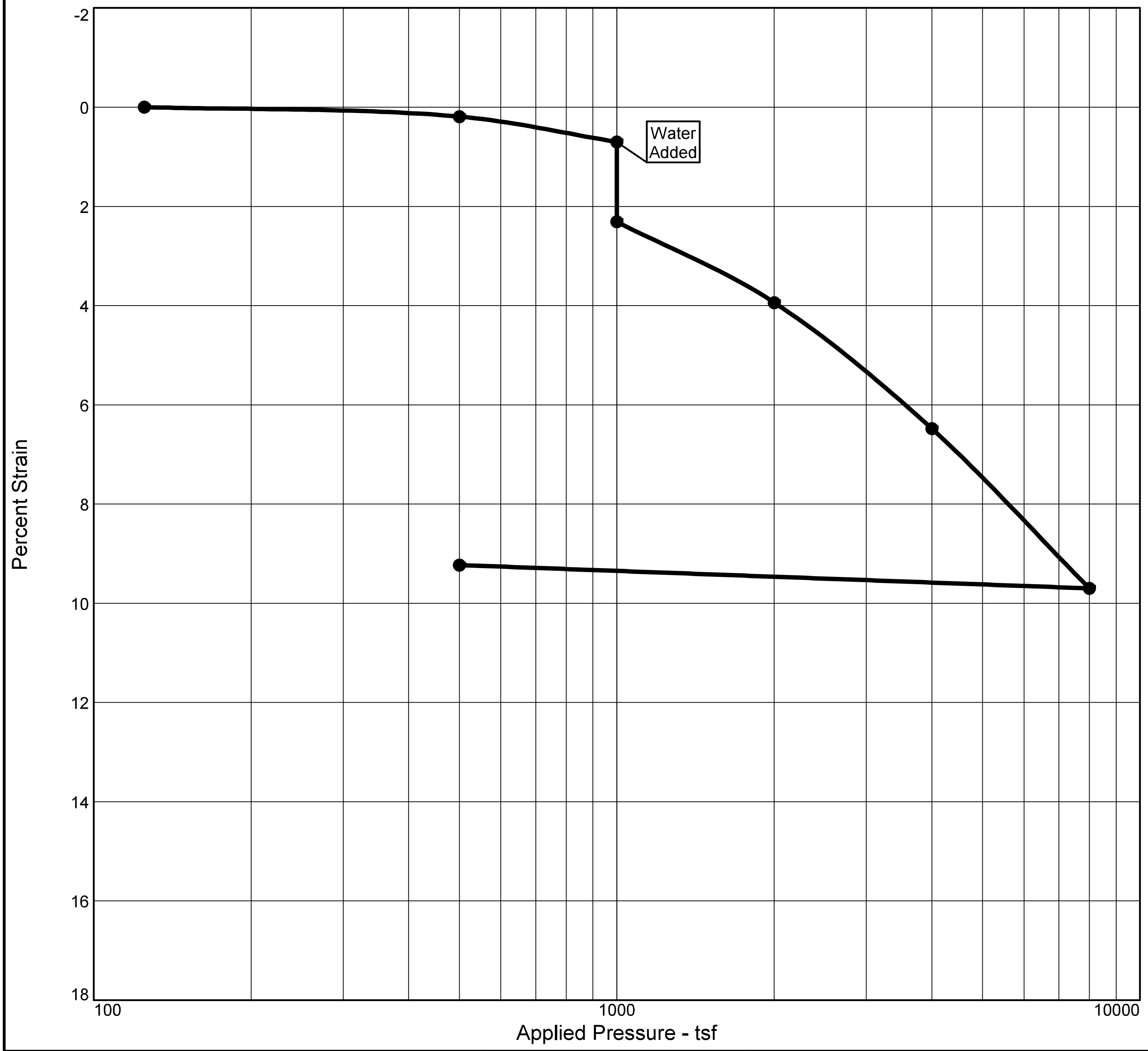
Gorian & Associates

Thousand Oaks, CA

Figure B.8

Tested By: CA

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Clpse. %	e_o
Sat.	Moist.											
							2528.5				1.6	

MATERIAL DESCRIPTION	USCS	AASHTO
MODELO FORMATION: SILTSTONE		

Project No. 3196-0-0 **Client:** imt Residential

Project: 325 Hampshire Rd

Location: B-5 **Depth:** 15'

Remarks:
8/27/21

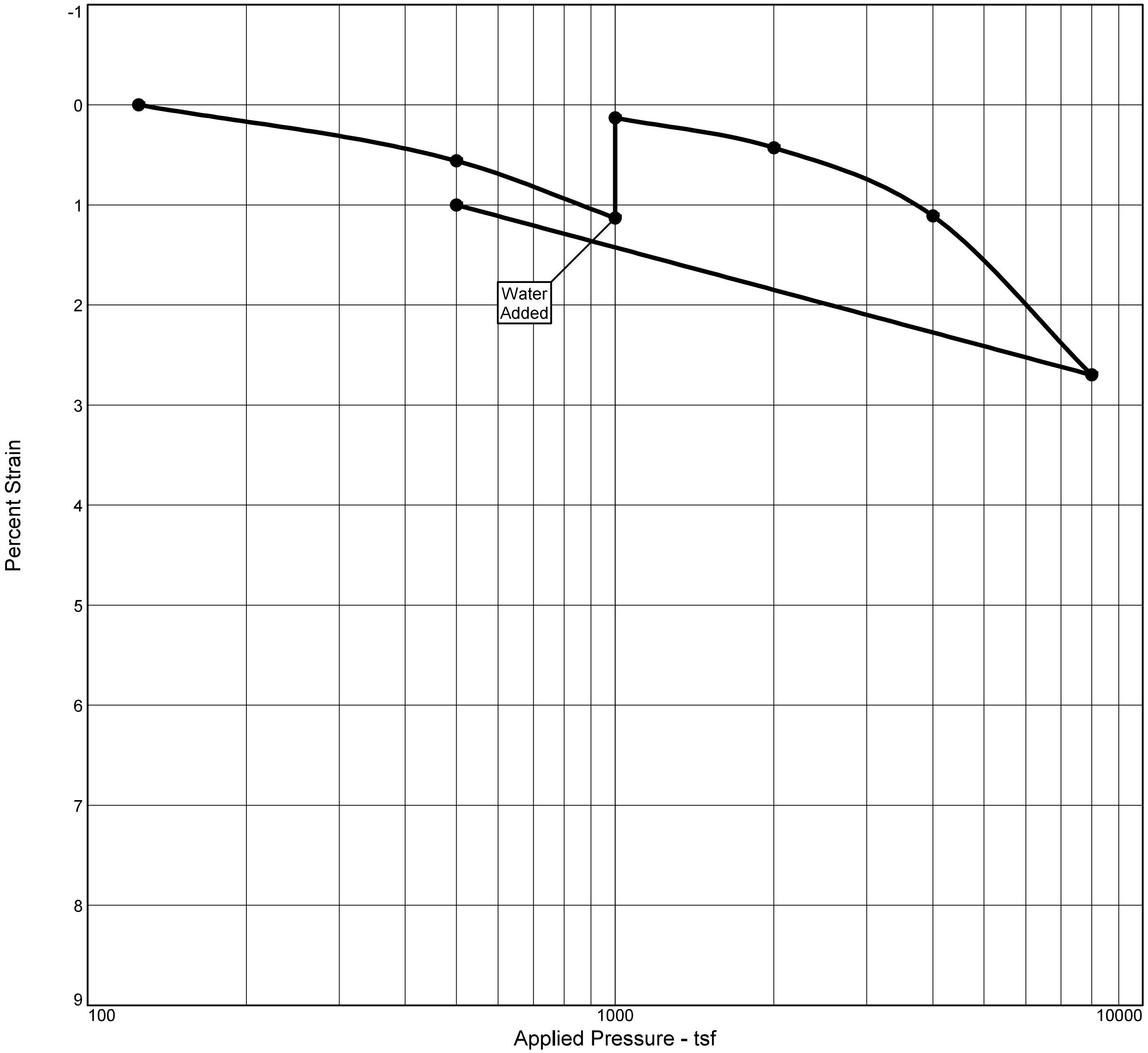
Gorian & Associates

Thousand Oaks, CA

Figure B.9

Tested By: CA

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
							3618.1			4048.5	1.0	

MATERIAL DESCRIPTION	USCS	AASHTO
OLDER ALLUVIUM: Silty CLAY		

Project No. 3196-0-0 **Client:** imt Residential

Project: 325 Hampshire Rd

Location: IB-6 **Depth:** 20'

Gorian & Associates

Thousand Oaks, CA

Remarks:
7/29/21

Figure B.10

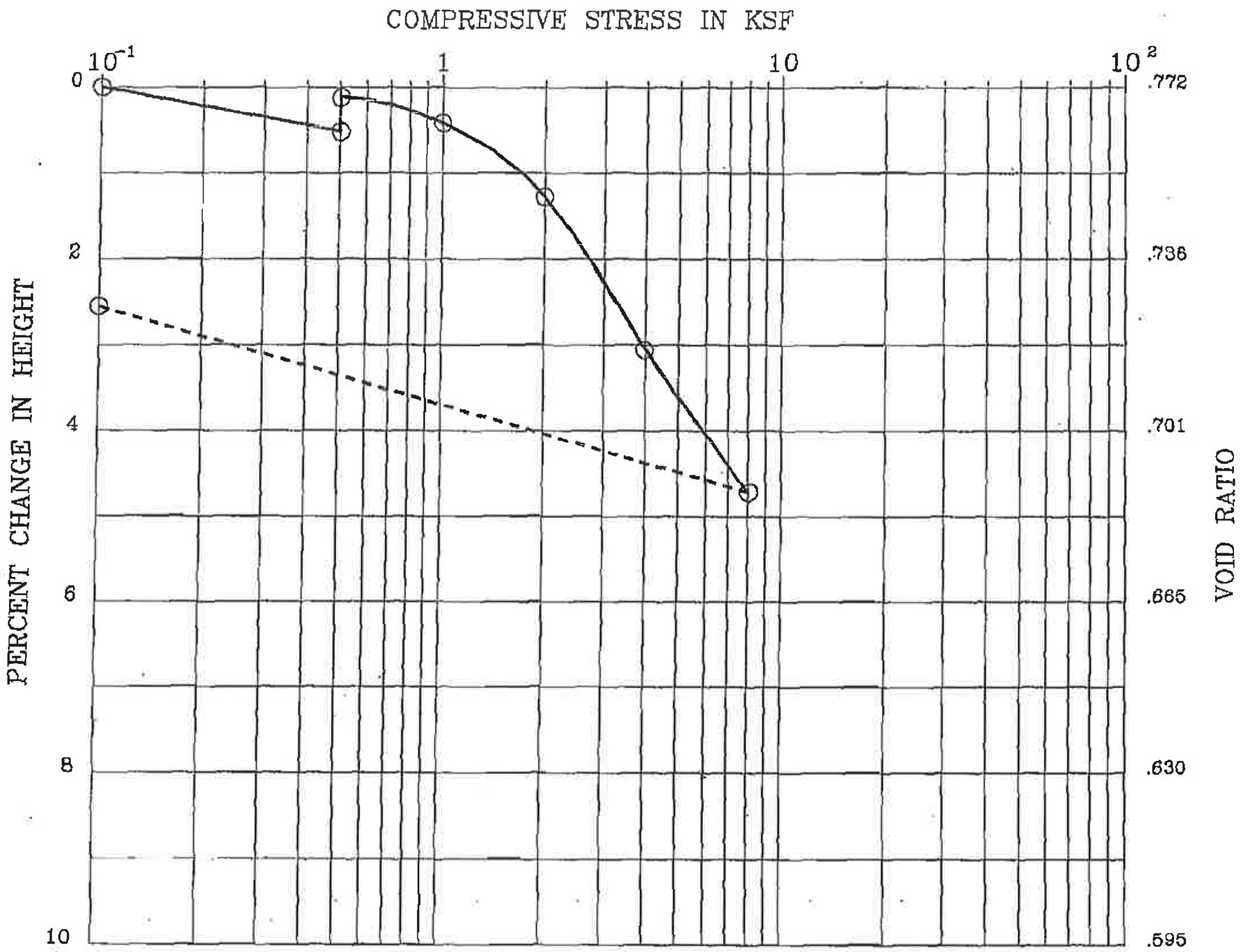
Tested By: CA

Soil Analysis Lab Results

Client: Gorian & Associates, Inc.
Job Name: X
Client Job Number: 3196-0-0-100
Project X Job Number: S210820C
August 23, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	ASTM D4658	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-3	9	65.8	0.0066	64.0	0.0064	2,479	1,072	7.0	113	<0.01	0.1	1.5	0.04	76.0	1.1	51.6	231.1	5.7	4.1
B-1	1	68.3	0.0068	33.3	0.0033	1,273	938	7.5	126	<0.01	32.7	11.8	ND	193.9	0.2	15.2	59.4	2.4	1.9

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)



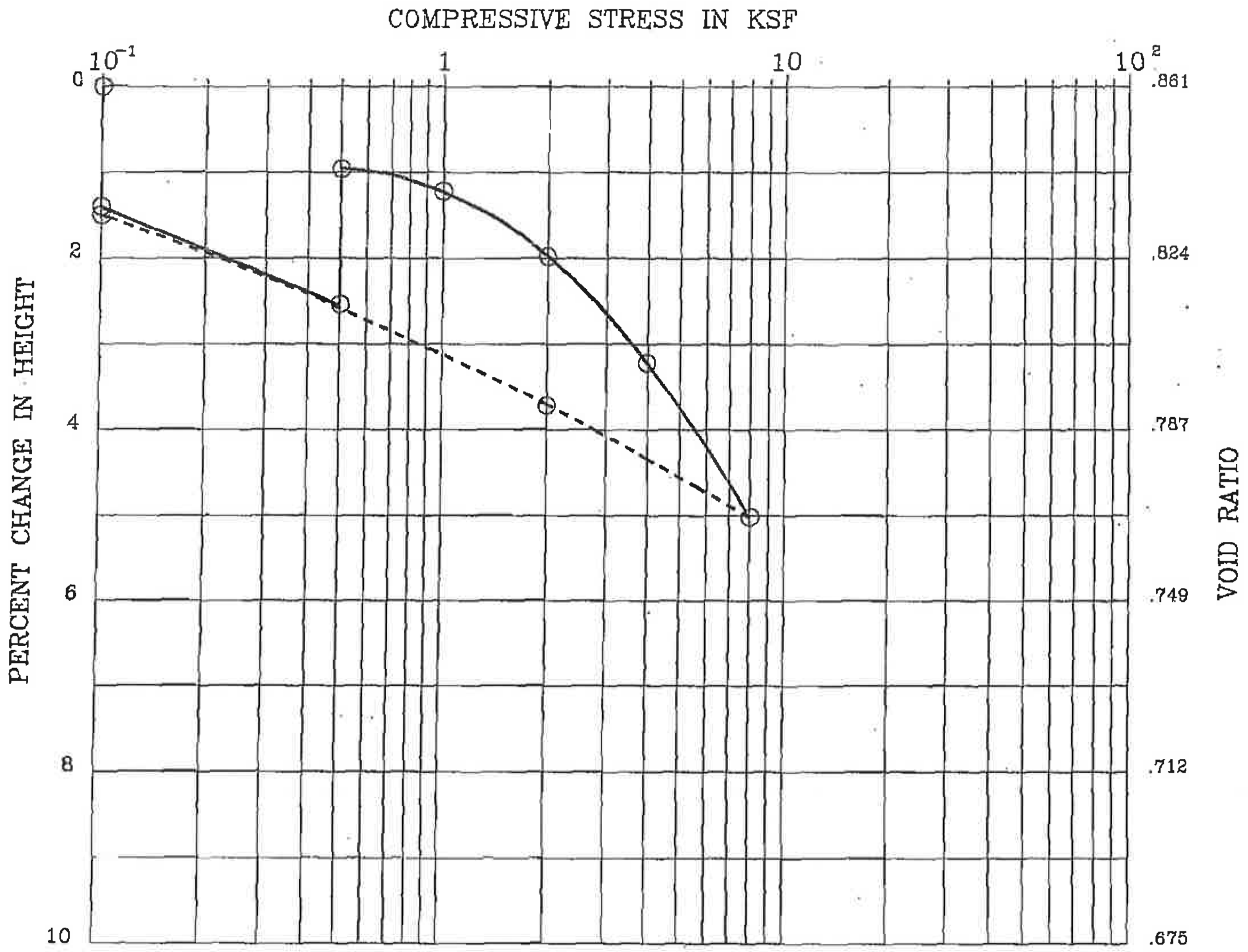
BORING : B-3
DEPTH (ft) : 3.5-5
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	22.0	93.4	76	.772
FINAL	25.5	95.9	94	.726

Remark : Saturated at 0.5 Ksf

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 1



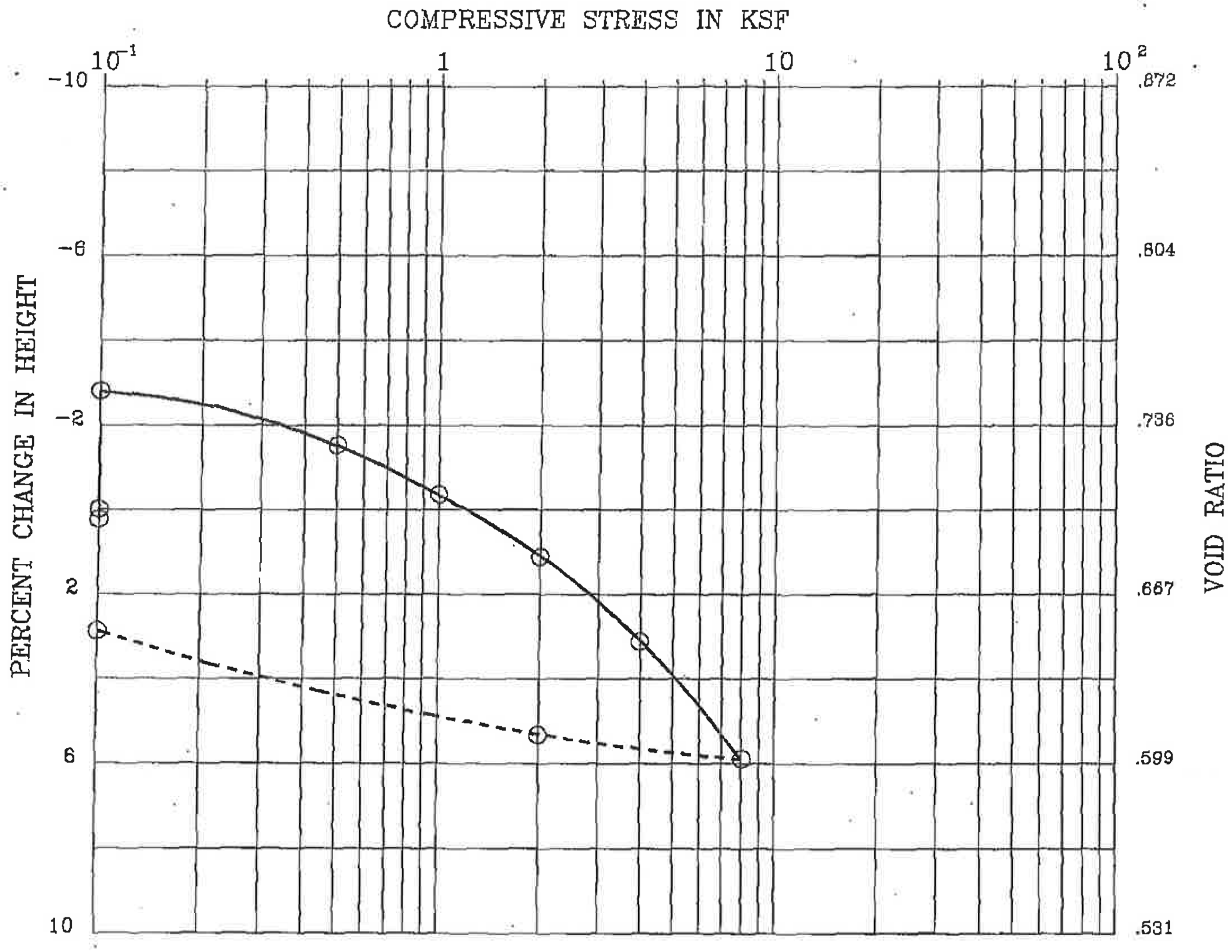
BORING : B-5
DEPTH (ft) : 5-6.5
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	29.2	89.0	90	.861
FINAL	31.3	90.3	100	.833

Remark : Saturated at 0.5ksf

D050A3.01	Home Depot Thousand Oaks
The Twining Labs Inc. Fresno, CA	CONSOLIDATION TEST Figure No. 2



BORING : B-8
DEPTH (ft) : 1.5-3
SPEC. GRAVITY : 2.65

DESCRIPTION :
LIQUID LIMIT :
PLASTIC LIMIT :

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	17.9	97.3	68	.702
FINAL	24.0	100.2	98	.652

Remark : Saturated at 0.1ksf

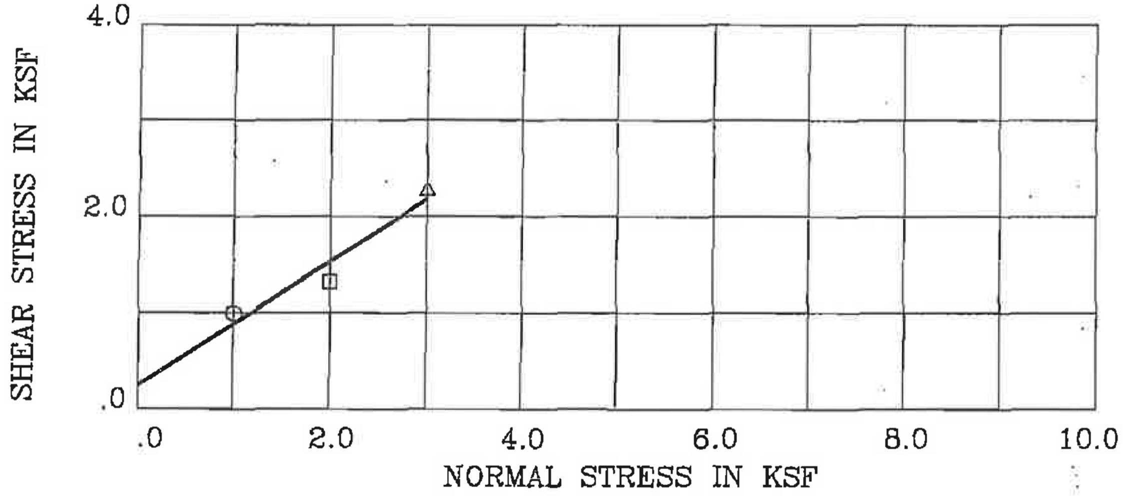
D050A3.01

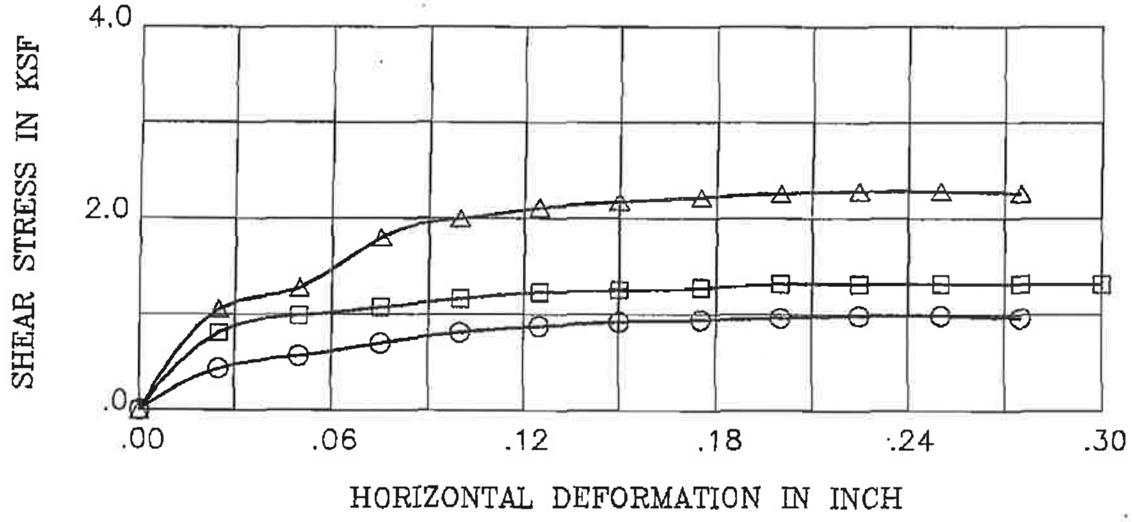
Home Depot Thousand Oaks

The Twining
Labs Inc.
Fresno, CA

CONSOLIDATION TEST

Figure No. 3





BORING/SAMPLE : B-8 DEPTH (ft) : 1.5-3
DESCRIPTION :
STRENGTH INTERCEPT (C) : .238 KSF
FRICTION ANGLE (PHI) : 32.9 DEG (PEAK STRENGTH)

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	VOID RATIO	NORMAL STRESS (ksf)	PEAK SHEAR (ksf)	RESIDUAL SHEAR (ksf)
○	29.3	105.5	.567	1.00	.99	.97
□	28.9	107.0	.546	2.00	1.32	1.32
△	26.9	104.8	.579	3.00	2.29	2.26

Remark :

D050A3.01

Home Depot Thousand Oaks

The Twining
Labs Inc.
Fresno, CA

DIRECT SHEAR TEST Figure No. 4

COMPACTION TEST REPORT

Project No.: D050A3.01 **Date:** 7-6-04
Project: Home Depot Remodel

Location:
Elev./Depth: 0.7 **Sample No.**
Remarks: B-2, 0.7"-1.5'

MATERIAL DESCRIPTION

Description: At 0.7 Inches - Poorly Graded Sand, damp, brown, with gravel and silt

Classifications - **USCS:** FILL **AASHTO:**
Nat. Moist. = **Sp.G. =** 2.65
Liquid Limit = **Plasticity Index =**
% > 3/4 in. = % **% < No.200 =**

TEST RESULTS
Maximum dry density = 99.4 pcf
Optimum moisture = 15.0 %

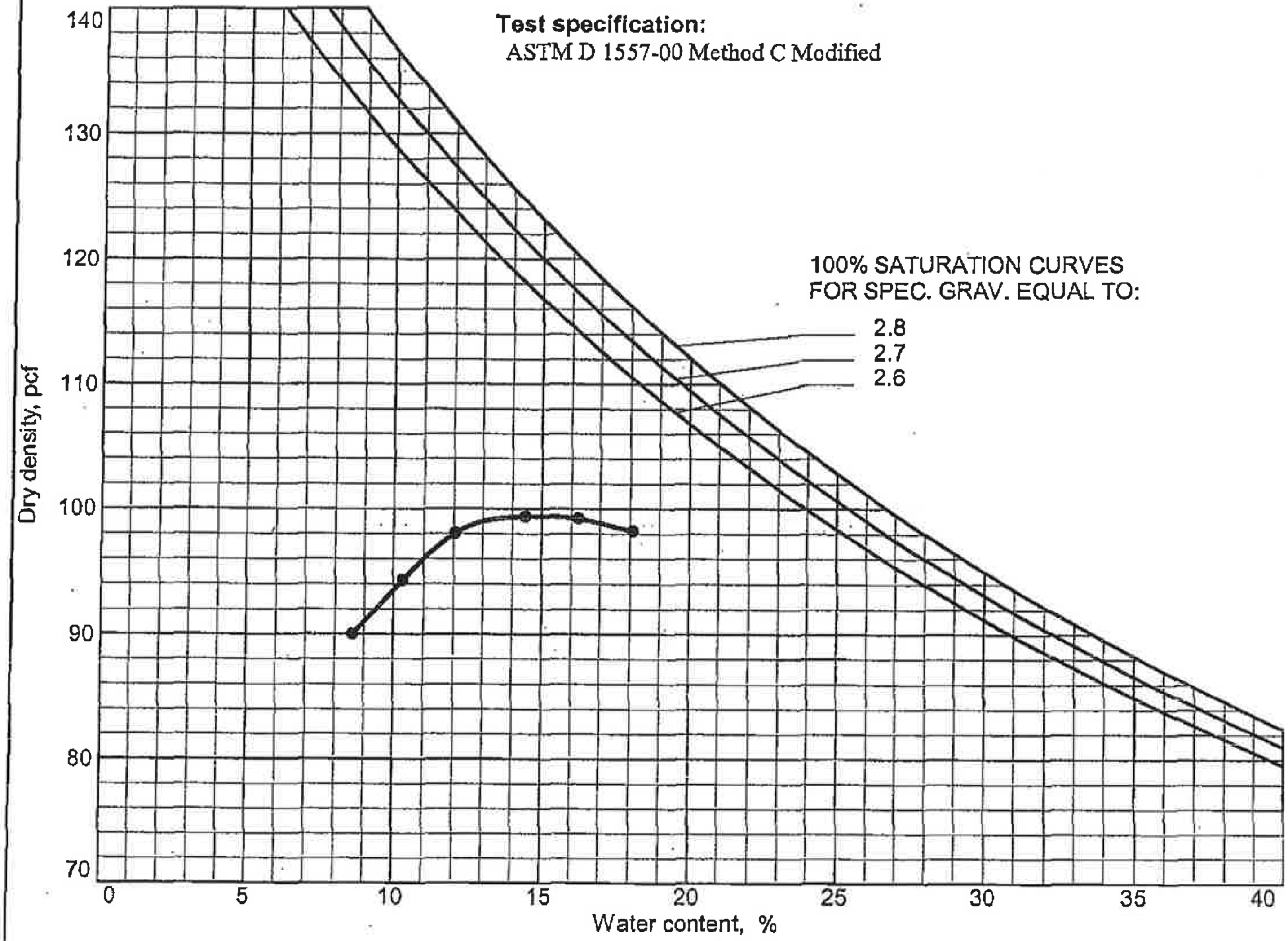


Figure 5

THE TWINING LABORATORIES, INC.

EXPANSION INDEX TEST

Uniform Building Code (UBC) 29-2

Project Number: D050A3.01 Project: Home Depot(Thousand Oaks)

Sample Location: B-8 Depth: 1-3' Date:07-5-04

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
B-8	13.6	28.5	98.6

Initial Thickness: 1.0000 Final Thickness: 1.0670

Expansion Index (EI): 67 Expansion Soil Classification: Medium

TABLE NUMBER 29-C
EXPANSIVE SOIL CLASSIFICATION

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

Figure No. 6

SEC.29.206 (a)

EXPANSION INDEX TEST

Uniform Building Code (UBC) 29-2

Project Number: D050A3.01

Project: Home Depot Thousand Oaks

Sample Location: B-2

Depth: 0.7'-1.5'

Date: 7-4-04

Sampled by:

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
B-2	11.9	21.9	101.9

Initial Thickness: 1.0000

Final Thickness: 1.0238

Expansion Index (EI): 23.8

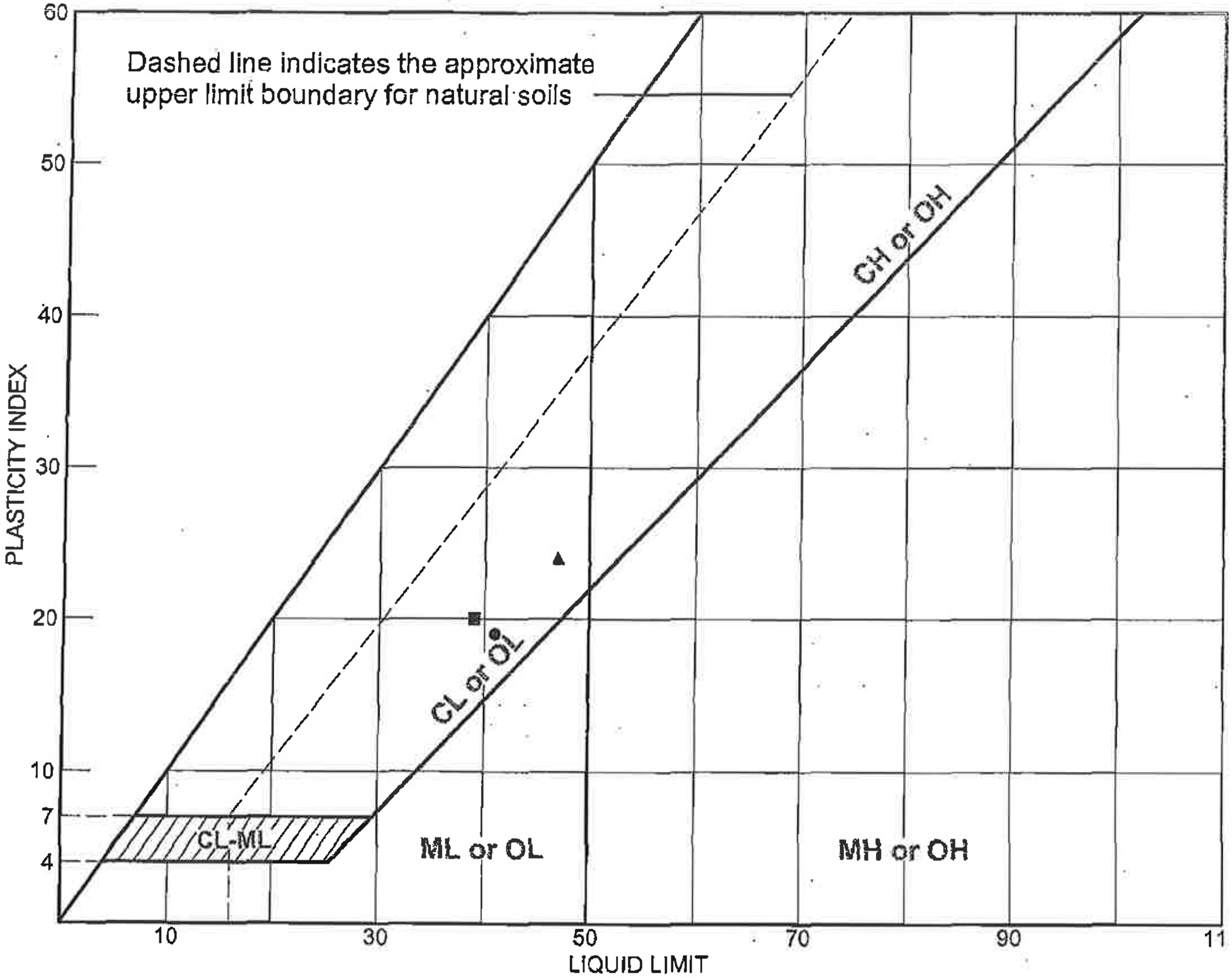
Expansion Soil Classification: Low

TABLE NUMBER 29-C
EXPANSIVE SOIL CLASSIFICATION

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

Figure No. 7

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-3		3.5		22	41	19	CL
■	B-5		5		19	39	20	CL
▲	B-8		1.5		23	47	24	CL

LIQUID AND PLASTIC LIMITS TEST REPORT

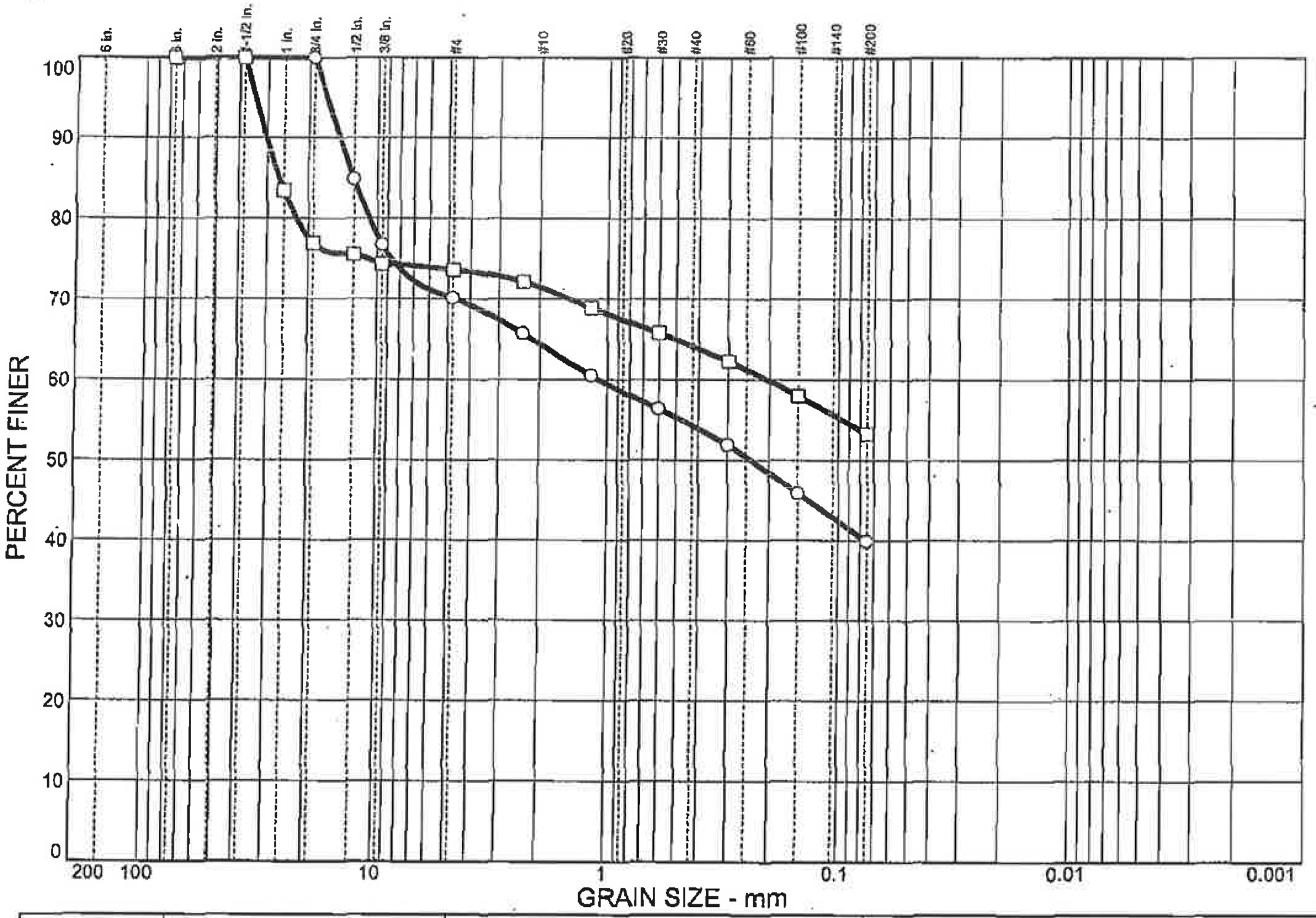
THE TWINING LABORATORIES, INC.

Client:

Project: Home Depot Remodel

Project No.: D050A3.01

Figure No. 8



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	29.9	5.7	10.1	14.4	39.9	
□	0.0	23.1	3.3	2.2	7.3	10.8	53.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○	B-3		3.5	At 3.5 Feet - Hard, with fine gravel	SM
□	B-5		5		CL

THE TWINING LABORATORIES, INC.

Client:

Project: Home Depot Remodel

Project No.: D050A3.01

Figure No. 9

THE TWINING LABORATORIES, INC.
EST. 1898
CALIFORNIA ELAP CERTIFICATE #1371

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

Twining Geotechnical Department 2527 Fresno Street	Project: Home Depot- Thousand Oaks Project Number: D050A3.01	**Reported:**
Fresno CA, 93721	Project Manager: Vasiliy Parfenov	07/06/04

B-7 .5-2

4F29004-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method
Inorganics							
Chloride	**14**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D-4327-84
Chloride	**0.0014**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
Sulfate as SO4	**0.0028**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
pH	**7.7**		pH Units	T4G0102	07/01/04	07/01/04	ATSM D4972-89 Mod
Resistivity	**6200**		ohms/cm	T4G0102	07/01/04	07/01/04	ASTM D1125-82
Sulfate as SO4	**28**	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D4327-84

Twining Geotechnical Department 2527 Fresno Street	Project: Home Depot- Thousand Oaks Project Number: D050A3.01	Reported:
Fresno CA, 93721	Project Manager: Vasiliy Parfenov	07/06/04

B-2 0.7-1.5

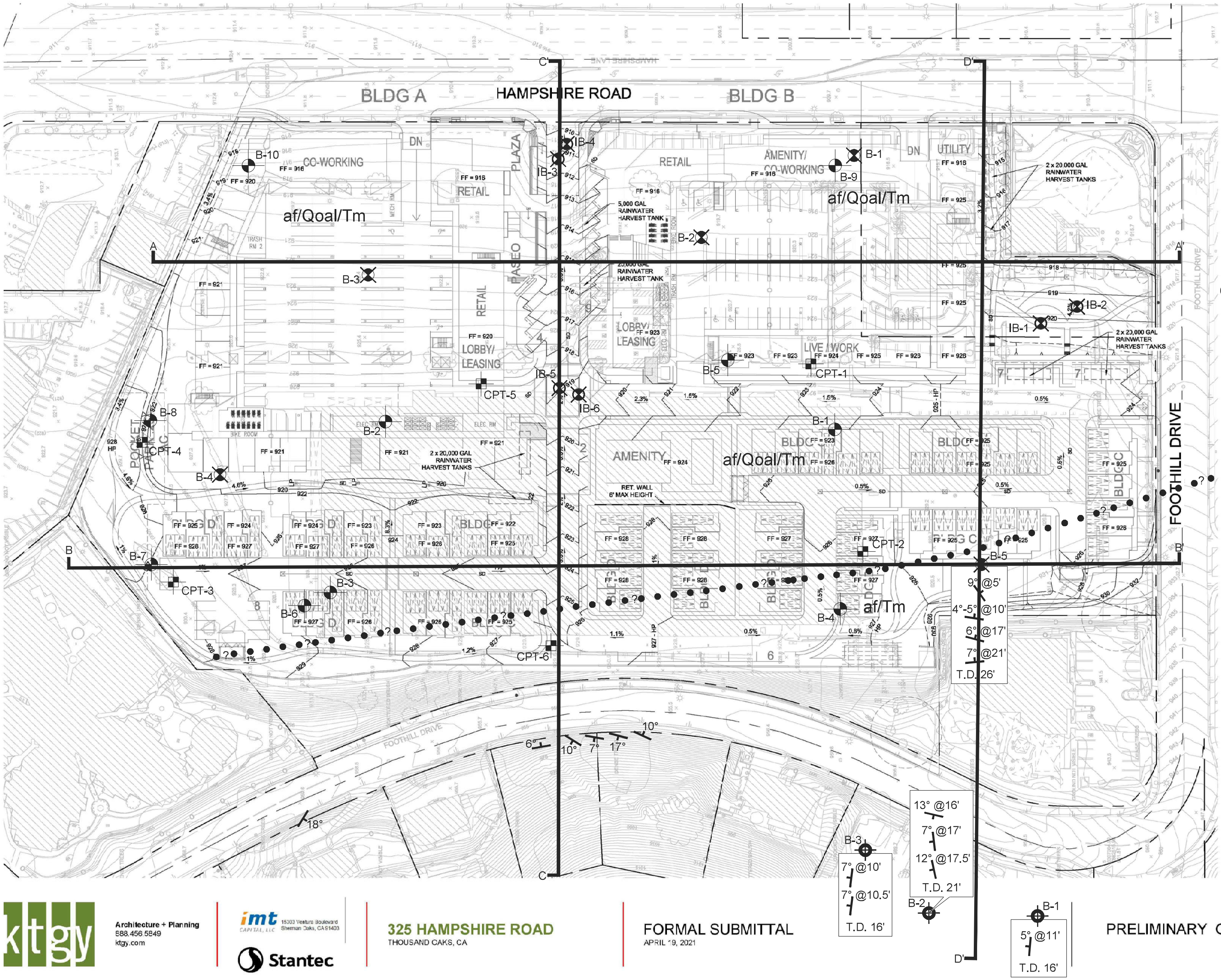
4F29004-02 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method
Inorganics							
Chloride	26	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D-4327-84
Chloride	**0.0026**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
Sulfate as SO4	**0.0079**	0.00060	% by Weight	[CALC]	07/01/04	07/01/04	ASTM D4327-84
pH	**6.9**		pH Units	T4G0102	07/01/04	07/01/04	ATSM D4972-89 Mod
Resistivity	**6200**		ohms/cm	T4G0102	07/01/04	07/01/04	ASTM D1125-82
Sulfate as SO4	79	6.0	mg/kg	T4G0102	07/01/04	07/01/04	ASTM D4327-84

Notes and Definitions

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

Quality Control Data Available Upon Request



EXPLANATION

af - Artificial Fill

Qoal - Older Alluvium

Tm - Modelo Formation

●●?●● - Very Approximate/Conjectural Geologic Contact Location

6° - Strike and Dip Bedding

B-5 ⊗ - Approximate Location of Infiltration Boring (This Report)

IB-6 ⊗ - Approximate Location of Exploratory Boring (This Report)

B-10 - Approximate Location of Exploratory Boring (Twining Laboratories, Inc. 2005)

CPT-6 - Approximate Location of CPT Sounding (Twining Laboratories, Inc. 2005)

B-3 - Approximate Location of Exploratory Boring (Gorian, 1987)

D D' - Geotechnical Cross Section

STORMWATER BMP SUMMARY

PROPOSED STORMWATER BMP:	RAINWATER HARVESTING
ESTIMATED SQDV =	144,000 GAL (19,250 CU-FT)
RAINWATER HARVESTING VOLUME PROVIDED =	145,000 GAL (19,384 CU-FT)

GEOTECHNICAL MAP

325 Hampshire Road
Thousand Oaks, CA

Prepared For
imt Residential
15303 Ventura Boulevard Suite #200
Sherman Oaks, CA

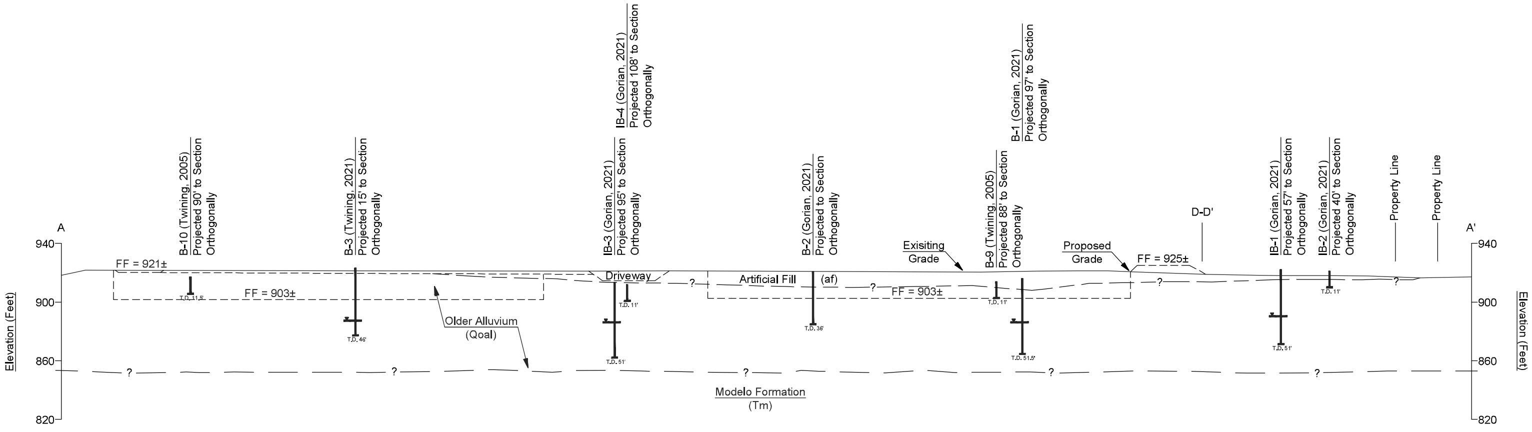
Gorian & Associates, Inc.
Applied Earth Sciences

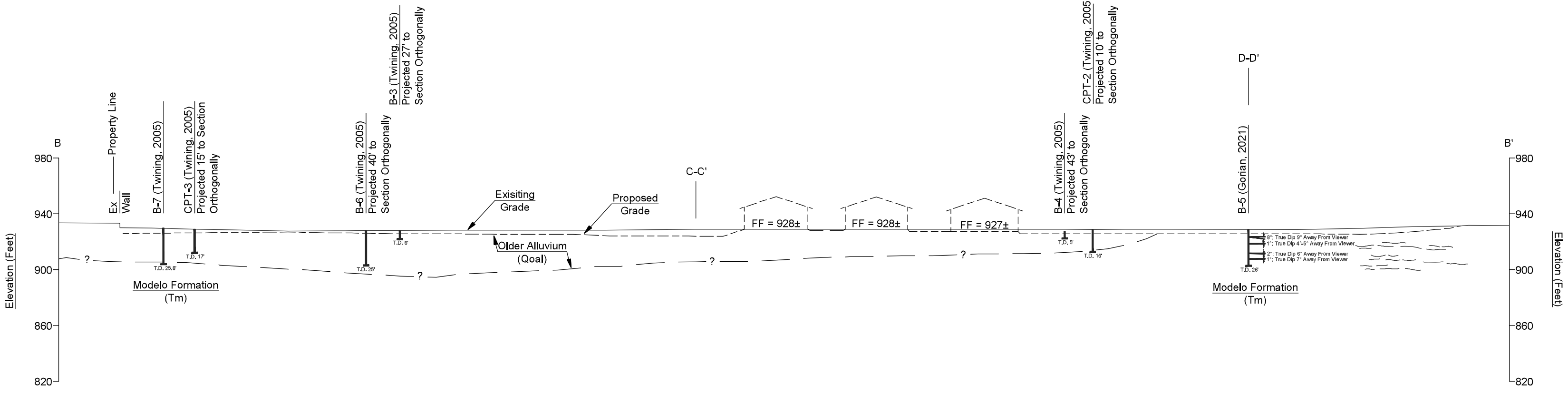
Job No: 3196-0-0-100		Date: October 2021
Scale 1" = 40'	Drawn by: Approved by:	PLATE 1

GRAPHIC SCALE

40 0 20 40 80 160

(IN FEET)
1 inch = 40 ft.



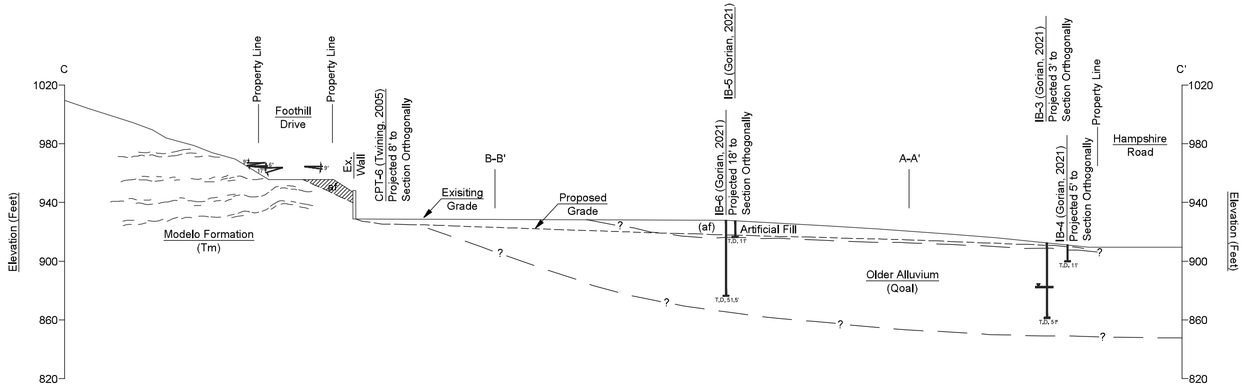


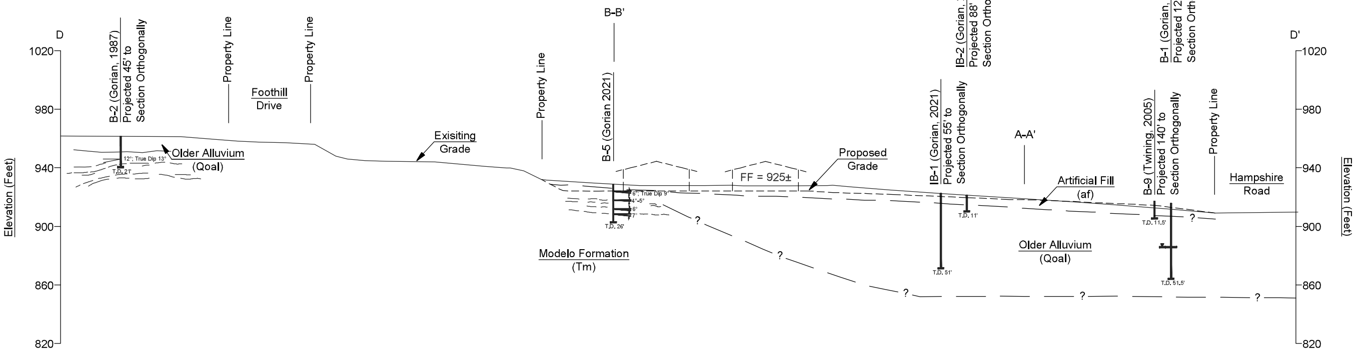
GEOTECHNICAL CROSS SECTIONS
325 Hampshire Road
Thousand Oaks, CA

Prepared For
imt Residential
15303 Ventura Boulevard Suite #200
Sherman Oaks, CA

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3196-0-0-100		Date: October 2021
Scale 1" = 40'	Drawn by: Approved by:	PLATE 2





GEOTECHNICAL CROSS SECTIONS
325 Hampshire Road
Thousand Oaks, CA

Prepared For
imt Residential
15303 Ventura Boulevard Suite #200
Sherman Oaks, CA

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3196-0-0-100		Date: October 2021
Scale 1" = 40'	Drawn by: Approved by:	PLATE 3