Preliminary Geotechnical Evaluation‡

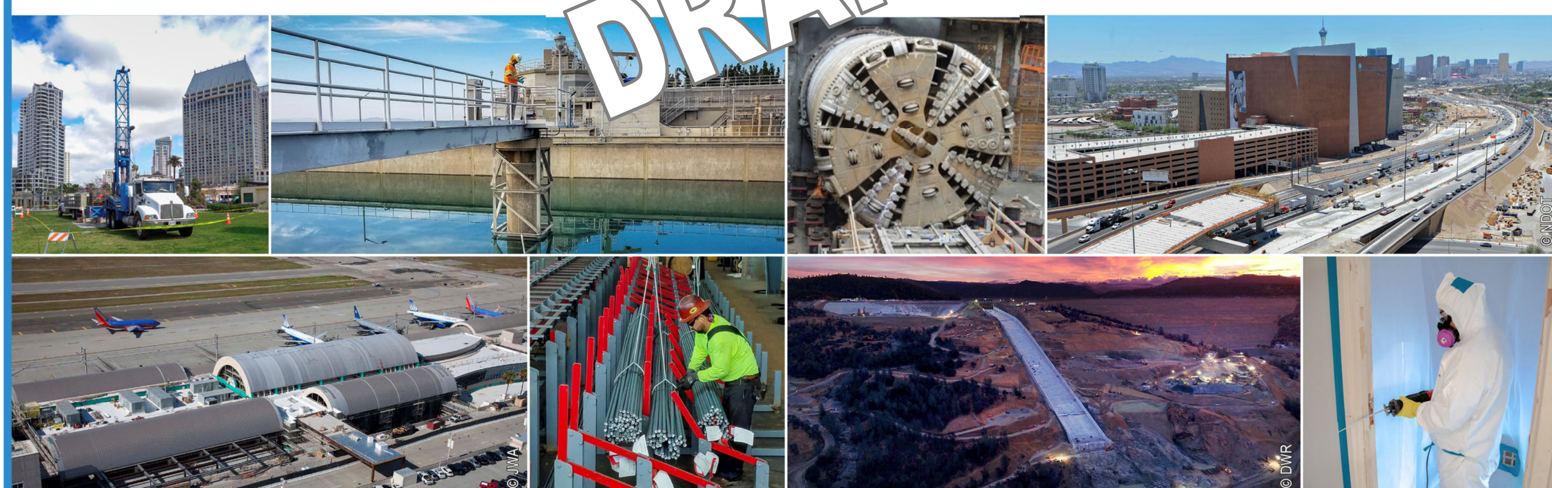
Home Depot Store – Granada Hills

Devonshire Street and Balboa Boulevard
Granada Hills, California♯

Home Depot

4000 West Metropolitan Drive, Suite 100 | Orange, California 92868

July 30, 2021 | Project No. 108824003



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Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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Granada Hills, California

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EXECUTIVE SUMMARY

In accordance with the project authorization from Home Depot U.S.A., Inc., Ninyo & Moore has performed a preliminary geotechnical evaluation for the proposed new Home Depot store located along the south side of Devonshire Street, southeast of its intersection with Balboa Boulevard in Granada Hills, California. The site consists of a developed lot with an existing commercial building and associated parking situated within the existing North Hills Plaza.

The site is currently developed with multi-tenant single, story building, associated asphalt concrete (AC) parking lot, drive aisles, and various underground utility installations. The existing building and associated parking will be partially demolished prior to the construction of the proposed Home Depot Store. An old drainage channel that transected the site from the northwest corner towards the southwest corner of the site has been filled in a part of the site development. Currently, a storm drain easement runs through the southwestern portion of the site. The existing underground utilities include domestic water, private fire service water, electrical and communication conduits, sewer, and storm drain. Existing elevations at the site range from approximately 940 above Mean Sea Level (MSL) at the northeast end of the site to approximately 925 above MSL at the southeast end of the site.

Based on preliminary site information provided to our office, the project will include the construction of a new Home Depot store. The store footprint will occupy 107,891 square feet (sf) with a Garden Center that occupies an area of 28,118 sf, a 2,465 sf tool rental center (TRC) space with a truck loading dock, a lumber canopy, and various other appurtenances. Further site improvements are to include vehicular pavements for drive aisles and parking lots, a pylon sign, surface concrete flatwork, and underground utilities. A portion of the existing storm drain and box culvert will be rerouted prior to construction of the proposed Home Depot store.

Our field evaluation will be performed in three phases, referred to as the Preliminary Evaluation, Base Bid, and Add Alternate phases. This report covers the Preliminary Evaluation phase.

The preliminary subsurface exploration was conducted on June 24, 2021, and included the performance of eight exploratory borings to depths up to approximately 26½ feet. The Base Bid and Add Alternate phases will include the performance of an additional 131 borings. The Base Bid scope will include the further borings to be performed outside the existing building and the Add Alternate scope will include the proposed borings to be performed inside the existing building.

Our preliminary subsurface exploration program indicates that the site is underlain by fill, young alluvium, and very old alluvium. AC pavements were encountered during our subsurface exploration in each of our borings. AC thicknesses ranged from approximately 3 to 8 inches. Where encountered, the aggregate base generally consisted of dark brown, moist, medium dense, poorly graded gravel with sand and poorly graded sand with gravel. The fill materials predominantly consisted of various shades of brown and yellow, moist, firm to stiff, sandy clay, and loose to medium dense, silty and clayey sand.

Groundwater was not encountered during the preliminary evaluation exploration program. Based on our review of groundwater data available on the Geotracker website (2021), historical high groundwater ranges between approximately 170 and 180 feet below the ground surface (CDMG, 1997 & 1998b). Groundwater may be encountered shallower due to the presence of the old drainage. Perched water conditions may occur due to the presence of trench backfill and bedding materials for underground utilities, as these materials tend to act as a conduit for perched water conditions. Fluctuations in the groundwater level and perched conditions may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors.

Based on the results of our preliminary subsurface exploration and our review of project documents, the proposed Home Depot store is underlain by fill and very old alluvium. The encountered fill depths ranged from approximately 3 to 18½ feet. The fill depths may be deeper in some areas, including in the vicinity of a filled in drainage and a storm drain that are present in the southwest portion of the proposed Home Depot store location. Due to the clayey nature of the exsting fill and young allivuim materials associated with the filled in old drainage and elevated moisture content of those materials, we are not recommending the removal and recompaction of the existing fill materials and young alluvium. In order to mitigate the potential for differential settlement, we recommend ground improvement be performed in the southern portion of the building. Additionally, we recommend the remaining portion of the building footprint be overexcavated down to a depth of 5 feet below the finished pad elevation or 2 feet below the bottom of foundations, whichever is deeper. The excavated materials should then be replaced by compacted fill soils to design elevations. Our recommendations may be reevaluated based on the findings in the Base Bid and Add Alternate phase subsurface exploration programs.

Shallow foundations, either spread or continuous, founded on compacted fill prepared in accordance with the ground improvement and remedial grading recommendations provided in Ninyo & Moore's geotechnical evaluation report may be designed using a net allowable bearing capacity of 3,500 pounds per square foot (psf). Concrete foundations placed on compacted fill may be designed using a coefficient of friction of 0.35. We estimate that the proposed structures, designed and constructed as recommended in Ninyo & Moore's geotechnical evaluation report will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 50 feet should be expected.

To provide a modulus of subgrade reaction of 250 pounds per cubic inch (pci), we recommend the slab be underlain by a 4-inch thick layer of aggregate base in accordance with the current Design Criteria Manual (2016).

In areas designated to receive structural pavements we recommend that the existing fill be overexcavated to a depth of 2 feet below planned subgrade elevation and recompacted as engineered fill soil to design elevations. For pavement design we have assumed Traffic Indices of 6.5 (Standard Duty) and 7.5 (Heavy Duty) for site pavements based upon the equivalent axle loading counts from the Design Criteria Manual (2016).

Our laboratory testing indicated that site soils possess a pH between 7.2 and 7.5, and electrical resistivities of approximately 755 and 1,000 ohm-centimeters (ohm-cm). The chloride contents of the tested samples were measured to be approximately 30 parts per million (ppm). The sulfate contents of the tested samples were approximately 0.002 (i.e. 20 ppm). Based on these test results, the site soils would be considered corrosive. We recommend that normal weight concrete in contact with soil possess a compressive strength of 2,500 pounds per square inch (psi) or more. Furthermore, due to the potential for variability of site soils, we also recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

Some onsite soils possess a high potential for expansion. These materials are not considered suitable for reuse as compacted fill within the upper 5 feet below the pad subgrade elevation for the building, within the upper 1 foot below retaining wall footings, or as retaining wall backfill. These materials should be exported or blended with onsite granular material to meet the recommendations presented in the "Materials for Fill" section of this report.

1. INTRODUCTION

In accordance with your request, we have performed a preliminary geotechnical evaluation for the proposed Home Depot Store located along the south side of Devonshire Street, southeast of its intersection with Balboa Boulevard in the North Hills Plaza in Granada Hills, California (Figure 1). Our preliminary evaluation was performed in accordance with our proposal dated March 18, 2020 and the updated Purchase Order No. 9016114794.

Based on our discussions with you, the Home Depot is interested in obtaining preliminary subsurface data prior to performing a full scale geotechnical evaluation in accordance with the Home Depot Design Criteria Manual (HDDCM) dated October 17, 2016. Our field evaluation will be performed in three phases, referred to as the Preliminary Evaluation, Base Bid, and Add Alternate phases. This report covers the Preliminary Evaluation phase. This report will be updated to include the Base Bid and Add Alternate phases at a later date. Since the Preliminary Evaluation is limited in scope, the number of borings performed is not in accordance with the HDDCM. However, the performance of the other two phases of work, the Base Bid and Add Alternate phases, will bring the evaluation up to the standards of the HDDCM. The Base Bid scope will include additional borings to be performed outside the existing building and the Add Alternate scope will include additional borings to be performed inside the existing building, to meet the criteria within the HDDCM.

The intent of this preliminary study was to provide general information regarding the site subsurface soil characteristics to aid in site acquisition. Prior to development, a full scale geotechnical evaluation in accordance with the HDDCM is to be performed. The objectives of this preliminary study were to assess the soil conditions at the site, evaluate the engineering properties of the soils encountered, and provide recommendations relative to the geotechnical aspects of the proposed improvements at the site. This report presents the results of our preliminary field explorations and laboratory testing, as well as our preliminary conclusions regarding the geotechnical conditions at the site and our preliminary recommendations for the design and construction of this project. Our recommendations may be reevaluated based on the findings during the Base Bid and Add Alternate phases.

2. SCOPE OF SERVICES

The scope of services for this preliminary evaluation included the following:

- Reviewing readily available background information including available as-built utility maps, geologic maps and literature, geotechnical reports, as-graded reports, groundwater data, historic aerial photographs, and topographic maps.

- Preparing and submitting for a boring permit from the County of Los Angeles Environmental Health department prior to performing the eight exterior borings for the Preliminary Evaluation.

- Performing an initial site reconnaissance of the site to observe and map existing geotechnical conditions and mark the eight Preliminary Evaluation exterior boring locations.
- Coordinating with site personnel to locate underground utilities near our exploration locations. Additionally, Underground Service Alert (USA) was contacted and a private utility locator was used to mark underground utilities.
- Performing the Preliminary Evaluation subsurface exploration program that includes drilling, sampling, and logging of eight exterior borings using a truck-mounted drill rig in the existing parking lots outside the existing buildings. The borings were drilled to depths up to approximately 26½ feet. Relatively undisturbed and bulk soil samples were collected at selected intervals. The collected samples were transported to our in-house geotechnical laboratory for testing:
- Performing geotechnical laboratory testing on selected samples to evaluate soil characteristics and design parameters. Our testing included an evaluation of in-situ moisture content and dry density, sieve (gradation) analysis, consolidation potential, shear strength, expansion index, soil corrosivity, and R-value.
- Compiling and analyzing the data obtained from our background review, field work, and laboratory testing.
- Preparing this draft preliminary geotechnical evaluation report providing our findings and conclusions relating to the geologic and geotechnical conditions at the site and our conclusions regarding the geotechnical feasibility of the project site.

3. SITE DESCRIPTION

The project site is situated on a developed commercial lot within the North Hills Plaza in Granada Hills, California (Figure 2). The project site fronts on Devonshire Street to the north and is bounded by commercial buildings and associated parking lots to the west, by single-family residential properties and a commercial building to the east, and single-family residential properties to the south. The site is currently occupied by an existing building and associated asphalt concrete (AC) parking lots that will be partially demolished prior to construction of the proposed Home Depot store.

The site is relatively flat, ranging in elevations between approximately 940 feet above mean sea level (MSL) to the northeast and approximately 925 feet above MSL to the southeast. Based on our review of historical topographic maps and aerial photographs, we understand that the site was first developed in the 1950's. As part of development at that time, a southeast-trending drainage was filled in that transected the southwest portion of the site (Figure 2). A sewer easement is roughly located along the center of the filled in drainage.

4. PROJECT DESCRIPTION

Based on the preliminary site plans (Lars Andersen & Associates, 2021), we understand that the proposed project includes the construction of a new 107,891 square foot (sf) Home Depot Store with a 28,118 sf Garden Center, a 2,465 sf tool rental center (TRC) space with a truck loading dock, a lumber canopy, and various associated appurtenances. Other improvements will include vehicular pavements for drive aisles and parking lots, a pylon sign, surface concrete flatwork, and underground utilities. We understand a portion of an existing storm drain and box culvert present in the south portion of the site will be rerouted prior to construction of the proposed Home Depot store.

5. FIELD EXPLORATIONS

Our preliminary subsurface exploration program was conducted on June 24, 2021 included the drilling, logging, and sampling of eight small-diameter borings (B-1 through B-8) to depths up to approximately 26½ feet. The borings were drilled using manual techniques and a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2488 by observing cuttings and drive samples. Bulk and in-place soil samples were obtained from the borings at selected intervals. The samples were then transported to our in-house geotechnical laboratory for testing. The approximate locations of the exploratory borings are shown on Figure 2. The logs of the exploratory borings are presented in Appendix A.

6. LABORATORY TESTING

Geotechnical laboratory testing of representative soil samples encountered during our preliminary subsurface exploration program included the performance of tests to evaluate in-situ moisture content and dry density, gradation (sieve) analysis, consolidation potential, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-situ moisture content and dry density tests are presented on the exploratory boring logs in Appendix A. The results of the other laboratory tests and a description of the test methods used are presented in Appendix B.

7. GEOLOGIC AND SUBSURFACE CONDITIONS

The following sections provide information regarding the geologic conditions relative to the project site.

7.1. Regional Geologic Setting

The project area is situated in the northwestern portion of the Los Angeles Basin, which is included in the Transverse Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 320 miles from the Pacific Ocean at Point Arguello, west of Santa Barbara, to the Joshua Tree National Monument west of Palm Springs. The province is up to 60 miles wide along the Los Angeles-Ventura County line and narrows to about 40 miles at its western end. The Los Angeles Basin has been divided into four structural blocks which are generally bounded by prominent fault systems: the Northwestern Block, the Southwestern Block, the Central Block, and the Northeastern Block (Norris and Webb, 1990The project site is located in the Northwestern Block, which is generally bounded by the Santa Monica and Raymond Faults on the south and the east and northeast by the San Gabriel Mountains. The portion of the province in Los Angeles County that includes the project area (San Fernando Valley) is a deep sedimentary trough.

The Transverse Ranges Province is traversed by a group of sub-parallel faults and fault zones trending approximately east-west and northwest (Jennings and Bryant, 2010). Several of these faults are considered active faults. The Santa Susana, San Gabriel, and San Andreas faults are active fault systems located northeast of the project area and the Northridge and Newport-Inglewood faults are active faults located west and south of the project area. The nearest known active fault is the San Fernando Segment of the Sierra Madre Fault, located approximately 3.3 miles north of the site. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of thrust and right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

7.2. Site Geology

According to the regional geologic map (Figure 3), and based on the results of our geologic reconnaissance and preliminary subsurface evaluation, the project site is generally underlain by fill soils, young alluvium, and very old alluvium. Prior to site development, a southeast-trending drainage was present that transected the southwest portion of the site (Figure 2). Generalized descriptions of the materials encountered during our subsurface exploration are presented below. Additional descriptions are provided on the boring logs presented in Appendix A. The geologic cross section was prepared at the location shown on Figure 2 and are presented on Figures 4A and 4B.

7.2.1. Pavement Sections

AC pavements were encountered during our subsurface exploration in each of our preliminary borings. Where encountered, the aggregate base generally consisted of various shades of gray and brown, moist, dense to very dense, poorly graded gravel and poorly graded sand with gravel. Note, approximately 5 inches of Portland Cement concrete (PCC) was encountered below the aggregate base in boring B-5. Table 1 summarizes the pavement sections as encountered in our borings.

Table 1 – Encountered Pavement Section Thicknesses		
Boring	**Encountered AC Thickness (inches)**	**Encountered Aggregate Base Thickness (inches)**
B-1	8	8
B-2	3	6
B-3	4	6
B-4	3½	4½
B-5*	4	5
B-6	4	6
B-7	3	Not encountered
B-8	4	4

Notes:
AC= asphalt concrete
*5 inches of PCC was encountered below the AC and aggregate base in boring B-5.

7.2.2. Fill

Fill materials were encountered underlying the pavement sections in each of our borings. As encountered, the fill soils extended to depths of approximately 3 to 18½ feet below the existing ground surface. Deeper fill soils were encountered in the area of the filled-in drainage and storm drain easement that crosses the southwestern portion of the site (i.e., near borings B-2 and B-3).

Where encountered, the fill soils generally consisted of various shades of brown, yellow, and gray, moist, loose to dense, clayey sand, silty sand, and poorly graded sand, and firm to hard, sandy lean clay. Various amounts of gravel were encountered in the fill soils. Scattered amounts of construction debris including asphalt and brick fragments were encountered in boring B-2.

7.2.3. Young Alluvium

Young alluvium associated with the old drainage that underlies the site was encountered beneath the fill materials in boring B-3. As encountered, the young alluvium extended to a depth of approximately 14 feet and consisted of brown, moist, loose, clayey sand. These materials are anticipated to underlie other portions of the site in the vicinity of the previously filled old drainage, as shown on Figure 2.

7.2.4. Very Old Alluvium

Materials mapped as very old alluvium were encountered underlying the fill material in borings B-1 through B-4, B-6, and B-8 and extended up to the total depths explored of approximately 26½ feet. As encountered, these materials generally consisted of various shades of brown, red, and yellow, moist, medium dense to very dense, clayey sand, silty sand, and poorly graded sand with silt and clay, and hard, sandy clay. Trace amounts of gravel were encountered within the very old alluvium.

7.3. Groundwater

Groundwater was not encountered during our preliminary evaluation. Based on our review of available data on the Geotracker website (2021), we anticipate that the groundwater table is situated at depths greater than 100 feet below the site. Historical high groundwater measurements range between approximately 170 and 180 feet below the ground surface (CDMG, 1997 & 1998b). Groundwater and seepage is not expected to be a constraint to the construction of the project; however, groundwater levels can fluctuate due to seasonal variations, irrigation, and other factors and may be present as perched groundwater or seepage in some areas.

Additionally, perched water conditions may be present at the site due to the existing drainage in the southwestern portion of the proposed building footprint and the presence of trench backfill and bedding materials for underground utilities, as these materials tend to act as a conduit for perched water conditions.

8. GEOLOGIC HAZARDS

In general, hazards associated with faulting and seismic activity include strong ground motion, ground surface rupture, and liquefaction. These considerations and other potential geologic hazards are discussed in the following sections.

8.1. Faulting and Seismicity

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on our geologic field mapping, the subject site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively). However, like the majority of southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed structures. Figure 5 shows the approximate site location relative to the major faults in the region. The nearest known active fault is the San Fernando Segment of the Sierra Madre Fault, located approximately 3.3 miles north of the site. The site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2007). Table 2 lists selected principal known active faults that may affect the site, including the approximate fault-to-site distances, and the maximum moment magnitudes (M_{max}) as published by the United States Geological Survey (USGS) (USGS, 2021a).

Table 2 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Sierra Madre (San Fernando Segment)	3.3 (5.3)	6.7
Santa Susana	3.6 (5.7)	6.9
Verdugo	4.5 (7.2)	6.9
Northridge	6.0 (9.7)	6.9
San Gabriel	8.9 (14.3)	7.3
Simi-Santa Rosa	11.7 (18.8)	6.9
Sierra Madre	12.0 (19.3)	7.2
Hollywood	13.0 (20.9)	6.7
Elysian Park (Upper)	15.3 (24.6)	6.7
Oak Ridge (Onshore Segment)	16.1 (25.9)	7.2
Puente Hills (Los Angeles Segment)	18.1 (29.1)	7.0
Raymond	18.3 (29.5)	6.8
San Cayetano	19.5 (31.4)	7.2
Palos Verdes	20.0 (32.2)	7.3
Clamshell-Sawpit	29.1 (46.8)	6.7
San Andreas (Northern Mojave Segment)	30.5 (49.1)	7.0
Elsinore (Whittier Segment)	31.5 (50.7)	7.0
Puente Hills (Santa Fe Springs Segment)	31.7 (51.0)	6.7
Santa Ynez (East Segment)	32.3 (52.0)	7.2
Puente Hills (Coyote Hills Segment)	36.2 (58.3)	6.9
Ventura-Pitas Point	37.2 (59.9)	7.0
San Jose	38.6 (62.1)	6.7
Mission Ridge-Arroyo Parida-Santa Ana	40.9 (65.8)	6.9
San Andreas (Bombay Beach Segment)	44.1 (71.0)	7.1
Oak Ridge (Offshore Segment)	44.3 (71.3)	7.0
Garlock (Garlock Western Segment)	44.8 (72.1)	7.3
Cucamonga	45.0 (72.4)	6.7
Red Mountain	46.4 (74.7)	7.4
Channel Islands Thrust	46.6 (75.0)	7.3
Santa Cruz Island	47.5 (76.4)	7.2
San Joaquin Hills	50.5 (81.3)	7.1
Pleito	51.6 (83.0)	7.1
North Channel	53.2 (85.6)	6.8
San Jacinto (San Bernardino Valley Segment)	53.5 (86.1)	7.1
San Andreas (North San Bernardino Segment)	54.5 (87.7)	6.9
Pitas Point (Lower)-Montalvo	56.3 (90.6)	7.3
Newport-Inglewood (Offshore Segment)	56.8 (91.4)	7.0
Cleghorn	59.3 (95.4)	6.8
Elsinore (Glen Ivy Segment)	59.9 (96.4)	6.9
White Wolf	60.5 (97.4)	7.2

Principal seismic hazards evaluated at the site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction related hazards (e.g., dynamic settlement). A brief description of these and other hazards and the potential for their occurrences at the site are discussed below.

8.1.1. Surface Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. The active San Fernando Segment of the Sierra Madre Fault Zone is located approximately 3.3 miles north of the site. Therefore, the probability of damage from surface ground rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

8.1.2. Strong Ground Motion

Based on our preliminary exploratory borings (Appendix A) and in accordance with ASCE 7-16 Sections 20.3 and 20.4, the site is considered Seismic Site Class D.

The 2019 CBC specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the ASCE 7-16 Supplement 1 (ASCE, 2018). We calculated that the S_1 for the site is equal to 0.794g using the 2021 Structural Engineers Association of California [SEAOC]/Office of Statewide Health Planning and Development [OSHPD] seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project site.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16. The average shear wave velocity (V_S) for the upper 100 feet (30 meters) of soil (V_{S30}) is mapped to be 974 fps (297 meters per second [m/s]) (Wald and Allan, 2008) and the depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 35 meters and 3,750 meters, respectively (Southern California Earthquake Center [SCEC] Community Velocity Model Version 4, Iteration 26, Basin Depth). These values were evaluated using the Open Seismic Hazard Analysis software developed by USGS and SCEC (2021). The mapped V_{s30} value was used for our site-specific analysis.

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The Open Seismic Hazard Analysis software developed by USGS and SCEC (2019) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the seismic design tool, hazard curves and deaggregation plots at the site using the USGS Unified Hazard Tool application (USGS and SCEC, 2021). A magnitude 6.5 seismic event on the Santa Susana East fault zone with a rupture distance of 6.3 kilometers from the site was evaluated to be the controlling earthquake. Hence, the deterministic seismic hazard analysis was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 6 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 6 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2019 CBC, are provided in Section 9.2 for the evaluation of seismic loads on buildings and other structures.

The site-specific Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) was calculated as 0.821g.

8.1.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils (with silt and clay contents of less than approximately 35 percent) and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking. According to the State of California Seismic Hazards Zones map (California Geological Survey, 2010), the site is not located in an area mapped as potentially susceptible to liquefaction. Based on the absence of a shallow ground water table and the primarily cohesive materials encountered during our preliminary subsurface evaluation, it is our opinion that the potential for liquefaction to occur at the site is not a design consideration.

8.2. Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. Seiches are similar oscillating waves on inland or enclosed bodies of water. Based on the location and elevation of the site, and the absence of nearby lakes or reservoirs, the potential for a tsunami or seiche to affect the site is not a design consideration.

8.3. Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Mapping Information Platform website (2021), the site is not located within mapped floodplains, flood zones, or active floodways. The site is also not located within a mapped dam inundation area (California Department of Water Resources, 2021). Based on this review and our reconnaissance, the potential for significant flooding and dam inundation at the site are not design considerations.

8.4. Landsliding

Based on our review of referenced geologic maps, literature, topographic maps, and stereoscopic aerial photographs, and on our site reconnaissance, no landslides or indications of deep-seated landsliding were noted underlying the project site. As such, the potential for significant large-scale slope instability at the site is not a design consideration.

9. CONCLUSIONS

Based on our preliminary geotechnical evaluation, it is our opinion that construction of the proposed improvements at the site is feasible from a geotechnical standpoint, provided the following recommendations are incorporated into the design and construction of the project.

- The areas of the proposed improvements are underlain by varying thicknesses of fill soils and young alluvium over very old alluvium.
- As currently planned, the southwestern proposed building will be underlain by a storm drain/old drainage. Fill soils and young alluvium were encountered to depths up to 18½ feet in the southern portion of the building. Recommendations for treatment of the deep fills and young alluvium to mitigate differential settlement are presented herein.
- Fill and young alluvium materials possess elevated moisture contents and additional processing and moisture conditioning should be anticipated prior to potential reuse.
- Fill and young alluvium materials encountered in our subsurface evaluation are not considered suitable for structural support in their current condition. Recommendations are presented herein for remedial grading of this material.
- The onsite are anticipated to be generally excavatable using heavy duty earthmoving equipment in good working condition.
- Granular onsite materials are generally considered suitable for reuse onsite as engineered fill, provided they are processed to meet the recommendations provided herein.
- Based on the results of our geotechnical laboratory testing, onsite soils exhibit a very low to high expansion potential. The clayey onsite materials that are expansive (i.e., an expansion index [EI] greater than 50) are not suitable for reuse as backfill materials within the limits of remedial grading as outlined in this report or behind retaining walls.
- Groundwater was not encountered during our subsurface exploration. However, the contractor should anticipate perched water and/or seepage conditions near the old drainage channel.
- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). However, the closest known major active fault is the San Fernando Segment of the Sierra Madre Fault, which is located approximately 3.3 miles north of the project.
- Based on the results of our limited geotechnical laboratory testing presented in Appendix B, as compared to American Concrete Institute (ACI) 318 (2019), the onsite soils are defined as Exposure Class S0 and C0. Additionally, as compared to the California (Caltrans, 2021) corrosion criteria, the onsite soils would be considered corrosive.

10. RECOMMENDATIONS

Based on our understanding of the project, the following preliminary recommendations are provided for the design and construction of the proposed new building and associated improvements. The proposed site improvements should be constructed in accordance with the requirements of the applicable governing agencies. Our recommendations may be reevaluated based on the findings in the Base Bid and Add Alternate phase subsurface exploration programs.

10.1. Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

10.1.1. Site Preparation

Site preparation should begin with the removal of flatwork, vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Obstructions that extend below the finished grade (such as tree stumps) should be removed to such a depth that organic material is generally not present and the resulting holes filled with compacted soil. Clearing and grubbing should extend outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area.

10.1.2. Excavation Characteristics

The results of our preliminary evaluation field exploration program indicate that the project site, as presently proposed, is underlain by fill soils, young alluvium and very old alluvium. These soils should be generally excavatable with heavy-duty earth moving equipment in good working condition. Both cohesive and granular soils were encountered in our exploratory borings, caving of open excavations should be anticipated, particularly where cohesionless soils are encountered or where excavations are not promptly backfilled.

10.1.3. Temporary Excavations and Shoring

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

Fill, Young Alluvium, and Very Old Alluvium	*Type C*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1½:1 (horizontal to vertical) in fill, young alluvium, and very old alluvium. Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. Onsite safety of personnel is the responsibility of the contractor.

10.1.4. Ground Improvement – Southern Portion of Building

As discussed herein, fill and young alluvium soils were encountered up to approximately 18½ feet deep in the southern portion of the proposed building (i.e., near borings B-2, B-3, and B-4). Due to the clayey nature of the exsting fill and young allivuim materials associated with the filled in old drainage and elevated moisture content of those materials, we are not recommending the removal and recompaction of the existing fill materials and young alluvium. Therefore, we recommend improvement of the subsurface soils to reduce the differential settlement potential.

The ground improvement should be designed to limit settlement to approximately 1 inch or less by improvement of the fill soils and young alluvium. The ground improvement should be performed in the southern portion of the proposed building pad. However, the limits of the old channel, resultant deep fills, and young alluvium are preliminary and may be revised based on the findings during the Base Bid and Add Alternate phases.

Detailed design of the soil improvement, including construction procedures, equipment, and the size and spacing of the improvement should be prepared by a specialty contractor to meet the project objectives. In general, we anticipate that ground improvement by rammed aggregate piers (such as the Geopier Rammed Aggregate Pier System or equivalent) may be appropriate for the site. The soil improvement should be designed to densify soils between the aggregate piers to provide cone tip resistance of 120 tons per square foot (tsf) by the cone penetrometer test (CPT). The system layout should also include support beneath planned foundations to provide an allowable soil bearing capacity of 4,000 pounds per square foot (psf).

10.1.4.1. Rammed Aggregate Piers

Rammed aggregate piers (such as the Geopier™ system) consist of compacted gravel columns that extend through the fill soils. The installation of aggregate piers provides for an increase in soil strength as a result of the compacted gravel columns and increased densification of surrounding soils. Aggregate piers are installed by pushing a probe down to the desired depth and then ramming the hole with 12-inch-thick lifts of mechanically compacted gravel. Since the added compaction increases the shear strength between the soils and aggregate piers, a higher bearing capacity can be realized for design of shallow foundations.

It is our opinion that aggregate piers are a feasible remedial measure for the fill soils and young alluvium conditions present in the southern portion of the proposed building. We recommend that a specialty contractor be retained to design the actual size, spacing, depth, and layout of the aggregate piers. In general, we would anticipate that the aggregate piers, would extend to a depth of approximately 20 feet below the existing ground surface and extend horizontally approximately 15 feet beyond all sides of the building footprint of the southern portion of the building.

The ground improvement should also be planned and coordinated with the site grading and remedial earthwork at the site. Subgrade preparation following ground improvement work may be involved to prepare structure pads.

10.1.5. Remedial Grading – Building Pad

Based on our preliminary borings, the northern portion of the proposed building is underlain by fill and very old alluvium and the southern portion of the proposed building is underlain by fill, young alluvium, and very old alluvium. After the ground improvement operations at the southern portion of the proposed building (i.e., near borings B-2, B-3, and B-4), we anticipate the upper soils within the building footprint will be either disturbed or removed. It has been our experience that this disturbed zone could extend to depths on the order of 5 feet or more. We recommend that the building be supported by foundations bearing on compacted fill and that remedial grading be performed to provide suitable foundation and slab support.

As a means to provide suitable foundation and slab support, we recommend the existing fills within the building pad be removed by overexcavating down to a depth of 5 feet below the finished pad elevation or 2 feet below the bottom of foundations, whichever is deeper. The overexcavation should extend to the horizontal limits of the structural footprint (including foundations for attached overhangs, canopies, elevators, and other building appurtenances) plus a horizontal distance of

5 feet. This overexcavation should extend to the horizontal limits of the building pad as previously defined, where feasible. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing buildings and property lines. Temporary slot cuts may be utilized in areas of deeper fill materials. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field to observe that the existing fills have been removed. Based on our observations, deeper removals may be recommended.

The resultant overexcavation surface should be scarified to a depth of approximately 8 inches, moisture conditioned and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new fill. The resulting excavation should then be backfilled with generally granular soils with a very low to low expansion potential (i.e., an EI of 50 or less). These materials are anticipated to consist of the import soils and/or soils derived from onsite excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

Onsite soils are anticipated to possess a medium to high potential for expansion. These materials are not considered suitable for reuse as compacted fill within the upper 5 feet below the pad subgrade elevation for the building.

10.1.6. Treatment of Existing Storm Drain Beneath Building Pad

We understand that the portion of the existing storm drain beneath the proposed Home Depot store will be rerouted to outside the building. The temporary excavation to remove the existing portions of the storm drain pipe that are no longer to be used should be backfilled in accordance with the recommendations provided in Section 10.1.9 of this report. As an alternative to removal of the storm drain pipe, the pipe may be abandoned and filled in place. If left in place, the pipe should be filled with a controlled low strength material (CLSM) with a compressive strength of 150 pounds per square inch (psi) according to "Greenbook," Section 2018 specifications. CLSM backfill should fill the entire void space within the abandoned pipe.

10.1.7. Remedial Grading – Site and/or Retaining Walls

If site and/or retaining walls not connected to buildings are planned, we recommend that the existing fill soils and alluvium not removed during grading be removed down to a depth of 1 foot below the bottom of footings. Additionally, any clay layers encountered should be removed from the footing excavation. This overexcavation should extend to the horizontal limits of the retaining wall foundations. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing structures and property lines. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

The resulting removal surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new compacted fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with onsite soils that possess a very low to low potential (i.e., an EI less than 50). These compacted fill soils should be placed at a relative compaction of 90 percent as evaluated by ASTM D 1557. These materials are anticipated to consist of import soils and/or soils derived from onsite excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

Onsite soils are anticipated to possess a medium to high potential for expansion. These materials are not considered suitable for reuse as compacted fill within the upper 1 foot below retaining wall footings unless they are blended with onsite granular material to meet the recommendations presented in the "Materials for Fill" section of this report. Additionally, the expansive clayey materials should not be used as wall backfill material.

10.1.8. Remedial Grading - Pavement and Flatwork

In the proposed pavement and flatwork areas, we recommend that the onsite soils be overexcavated to a depth of 2 feet below the subgrade elevation. The proposed overexcavations should extend outward horizontally 2 feet from the horizontal limits of the pavement or flatwork. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 8 inches, moisture conditioned, and recompacted to a relative compaction of 92 percent as evaluated by ASTM D 1557.

The overexcavation should then be filled with engineered fill. The engineered fill should be moisture conditioned to at or slightly above the material's optimum moisture content and compacted to a relative compaction of 92 percent as evaluated by ASTM D 1557. The upper 12 inches of subgrade soils beneath vehicular pavements should be placed at a relative compaction of 95 percent as evaluated by ASTM D 1557.

Onsite soils are anticipated to possess a medium to high potential for expansion. These materials are not considered suitable for reuse as compacted fill within 2 feet below the subgrade elevation for pedestrian concrete paving and/or exterior flatwork areas.

10.1.9. Shrink and Swell Factors

Due to the limited scope of the preliminary phase, the shrink and swell factors will be evaluated during the Base Bid and Add Alternate phases.

10.1.10. Materials for Fill

Onsite soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight), and that possess an EI of less than 50, are suitable for reuse as engineered fill material. In general, fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch.

Imported fill material, as needed based on the contractor's means and methods, should generally be granular soils with a very low to low expansion potential (i.e., an EI of 50 or less). Import fill material should also be non-corrosive in accordance with the California (Caltrans, 2021) corrosion criteria. Non-corrosive soils are soils that possess an electrical resistivity more than 1,500 ohm-centimeter (ohm-cm), a chloride content less than 500 parts per million (ppm), less than 0.15 percent sulfates, and a pH greater than 5.5. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing.

10.1.11. Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve moisture contents generally at or slightly above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify this office and the appropriate governing agency when project areas are ready for observation and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally at or slightly above the laboratory optimum moisture content prior to placement and should be generally consistent within the soil mass. The optimum moisture content will vary with material type and other factors.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally at or slightly above the laboratory optimum, mixed, and then compacted by mechanical methods, to a relative compaction of 92 percent as evaluated by ASTM D 1557. The upper 12 inches of the subgrade materials beneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

10.1.12. Pipe Bedding and Modulus of Soil Reaction (E')

It is our recommendation that new pipelines (pipes), where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or more.

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,800 psi may be used for an excavation depth of up to approximately 5 feet when backfilled soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557.

10.1.13. Pipe Zone Backfill

The pipe zone backfill should be placed on top of the pipe bedding material and extend to 1 foot or more above the top of the pipe in accordance with the recent edition of the Standard Specifications for Public Works Construction ("Greenbook"). Pipe zone backfill should have a Sand Equivalent (SE) of 30 or more and be placed around the sides and top of the pipe. Silts and clays should not be used as pipe zone backfill. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

It has been our experience that the voids within a crushed rock material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric.

10.1.14. Utility Trench Zone Backfill

Based on our subsurface evaluation, the onsite earth materials should be generally suitable for reuse as trench zone backfill provided they are free of organic material, clay lumps, debris, and rocks more than approximately 3 inches in diameter. Trench zone backfill should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench zone backfill should be compacted to relative compaction recommendations presented in "Compacted Fill" section of this report. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

10.1.15. Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the magnitude and distribution of passive lateral earth pressures presented on Figure 7. Thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

10.2. Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 3 presents the seismic design parameters for the site in accordance with the CBC (2019) guidelines and adjusted MCE spectral response acceleration parameters (SEAOC/OSHPD, 2021).

Table 3 – 2019 California Building Code Seismic Design Parameters

Seismic Design Factors	Value
Seismic Design Category	D

Table 3 – 2019 California Building Code Seismic Design Parameters	
Seismic Design Factors	**Value**
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	2.265g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.796g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.812g (per Figure 6)
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.592g (per Figure 6)
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.208g (per Figure 6)
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.061g (per Figure 6)
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration (PGA_M)	0.821g (per Figure 6)

10.3. Foundations

Based on our understanding of the project, the proposed building will be constructed at the site following demolition and/or removal of the existing improvements. Provided the ground improvement measures have been implemented in the southern portion of the building, the building may be founded on shallow foundations with interior concrete slabs-on-grade. Recommendations for the shallow foundations are presented in following sections. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

10.3.1. Shallow Spread or Continuous Footings

Shallow spread or continuous footings bearing on compacted fill prepared as recommended herein may be designed using a net allowable bearing capacity of 3,500 psf. Shallow spread or continuous footings should be founded 18 inches or more below the lowest adjacent grade and should be 18 inches or more in width. The allowable bearing capacity may be increased by 250 psf for each additional foot of foundation depth or width up to a value of 4,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

To provide consistent bearing conditions for the foundations, we recommend that utilities, piping, or duct banks that are to be constructed parallel to foundations, be installed 1 foot or more away from or outside of the zone of influence from the bottom of each foundation. The zone of influence is defined by a 1:1 (horizontal to vertical) downward projection that extends outward from the bottom, outside edge of the foundation.

10.3.2. Lateral Resistance

For resistance of footings bearing on compacted fill to lateral loads, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three multiplied by the height generating the passive pressure, whichever is more. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between compacted fill and concrete. These values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

10.3.3. Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, and founded on compacted fill will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 50 feet should be expected.

10.4. Canopy/Shade Structure and Light Pole Foundations

Canopy/shade structures and light poles may be supported on cast-in-drilled-hole (CIDH) concrete piles. Such structures typically impose relatively light axial loads on foundations. Although we anticipate that foundation dimensions will be generally governed by the lateral load or uplift demand, therefore we recommend that CIDH foundations for canopy/shade structure and/or light poles have a diameter of 18 inches or more and shallow spread footings be 24 inches or more in width. The pile (i.e., diameter and embedment) and/or shallow spread foundation dimensions should be evaluated by the project structural engineer.

For resistance of CIDH piles to lateral loads that are founded in compacted fill or alluvium, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a maximum value of up to 3,500 psf. This value assumes that the structures are designed to tolerate ½ inch of deflection at the surface and that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is more. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The CIDH pile construction should be observed by Ninyo & Moore during construction to evaluate if the piles have been extended to the design depths. The drilled holes should be cleaned of loose soil and gravel. It is the contractor's responsibility to (a) take appropriate measures for maintaining the integrity of the drilled holes, (b) see that the holes are cleaned and straight, and (c) see that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by 1 percent of the depth of the pile. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile.

10.5. Standard Retaining and/or Site Walls

Standard site retaining walls consisting of cast-in-place concrete, masonry, or concrete tilt-up panels that are not a part of or are not connected to the buildings may be supported on continuous footings bearing entirely on compacted fill. The continuous footings should have a width of 24 inches or more and be embedded a depth of 18 inches or more. A net allowable bearing capacity of 3,000 psf may be used for the design of site retaining wall foundations. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces.

For the design of a yielding retaining wall that is not restrained against movement by rigid corners or structural connections, the design lateral earth pressures are presented on Figure 8. Restrained walls (non-yielding) may be designed for the lateral earth pressures presented on Figure 9. These pressures assume low-expansive backfill and free draining conditions. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 10. The drain should be connected to an appropriate outlet.

10.6. Interior Slabs-On-Grade

We recommend that conventional, interior concrete slab-on-grade floors be underlain by compacted fill materials of generally very low to low expansion potential. Interior concrete slabs on grade should be 5 inches thick or greater and be reinforced in accordance with the structural engineer's recommendations. In addition to the slab reinforcement, crack control and expansion joint spacing should be designed by the project structural engineer.

The contact pressure beneath the concrete slab-on-grade should not exceed 3,500 psf. To provide a modulus of subgrade reaction of 250 pounds per cubic inch (pci), we recommend the slab be underlain by a 4-inch thick layer of aggregate base materials in accordance with the current Design Criteria Manual (2016). The base materials should be moisture-conditioned and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. It is our understanding that a significant portion of the concrete slab will not be covered with flooring. As such, a vapor retarder may not be needed. However, we recommend that a concrete surface sealer be considered for areas to receive moisture-sensitive flooring. A coefficient of friction of 0.35 may be used between the concrete slab-on-grade and subgrade soils in design.

The coefficient of subgrade reaction K_b for a slab of a specific width may be evaluated using the following equation:

$$\mathbf{K_b = K_v[(b+1)/2b]^2}$$

where b is the width of the slab in feet.

10.7. Concrete Flatwork

We recommend that exterior concrete flatwork that is underlain by 2 feet of more of compacted fill with a low potential for expansion be at least 4 inches in thickness and be reinforced with No. 3 reinforcing bars placed at 24 inches on-center in both directions. A vapor retarder is not needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the civil engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 6 inches, moisture conditioned to generally at or slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

10.8. Preliminary Flexible Pavement Design

We understand that the project will include the construction of new pavements. Our laboratory testing of a near surface soil samples at the project site indicated R-values 19 and 41. We have used an R-value of 19 for preliminary design of the pavement. We have used an R-value of 19, along with design Traffic Indices (TI) of 6.5 for Standard Duty pavements and 7.5 for Heavy Duty pavements as the basis of our preliminary flexible pavement design. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 4.

Table 4 – Recommended Preliminary Flexible Pavement Sections

Traffic Index	Pavement Type	Design R-Value	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
6.5	Standard Duty	19	4	11½
7.5	Heavy Duty	19	4½	13½

As indicated, these values assume TIs of 7.5 or less for site pavements. If traffic loads are different from those assumed, the pavement design should be re-evaluated. In addition, we recommend that the upper 12 inches of the subgrade soils and the aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557. Additionally, the AC material should be compacted to 95 percent of the its Hveem density.

10.9. Preliminary Rigid Pavement Design

We understand that the project will include the construction of new pavements. Our laboratory testing of a near surface soil samples at the project site indicated R-values 19 and 41. We have used an R-value of 19 for preliminary design of the pavement. We have used an R-value of 19, along with design Traffic Indices (TI) of 6.5 for Standard Duty pavements and 7.5 for Heavy Duty pavements as the basis of our preliminary rigid pavement design. The Standard Duty and Heavy Duty TIs are based on Equivalent Axle Loadings (EALs) of 50,000 and 220,000, respectively. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 5.

Table 5 – Recommended Preliminary Rigid Pavement Sections

Traffic Index	Pavement Type	Portland Cement Concrete Thickness (inches)	Aggregate Base Thickness (inches)
6.5	Standard Duty	6	6
7.5	Heavy Duty	7	6

As indicated, these values assume TIs of 7.5 or less for site pavements. If traffic loads are different from those assumed, the pavement design should be re-evaluated. In addition, we recommend that the upper 12 inches of the subgrade soils and the aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557. PCC pavements should have a flexural strength of 600 psi.

10.10. Corrosivity

Laboratory testing was performed on representative samples of the near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. The chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417.

The pH of the tested samples were 7.2 and 7.5, the electrical resistivity results were 755 and 1,000 ohm-cm, the chloride content was 30 ppm, and the sulfate content was measure to be 0.002 percent (i.e., 20 ppm). Based on a comparison with the California (Caltrans, 2021) corrosion criteria, the onsite soils would be classified as corrosive. Corrosive soils are defined as soil with an electrical resistivity less than 1,500 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

10.11. Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. As noted, the soil samples tested in this evaluation indicated water-soluble sulfate contents of 0.002 percent by weight (i.e., 20 ppm). Based on the ACI 318 (2019) criteria, the site soils would correspond to exposure class S0. Additionally, per ACI 318 (2019), the soils would also be classified as exposure class C0. For these exposure classes, ACI 318 (2019) recommends that normal weight concrete have a compressive strength of 2,500 psi or more.

10.12. Drainage

Proper surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from the new improvements, including structures, retaining walls, and flatwork. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet away from foundations and tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to footings or pavements.

11. PERMANENT INFILTRATION DEVICES

Based on our discussions with the client, we understand that the project is considering the use of biofiltration basins in the design of the project. The National Resources Conservation Service (NRCS) soil survey maps (USDA, 2021) classify the onsite materials of the site as Soil Group B in the northern and central portions of the site and Soil Group C in the southern portion of the site. NRCS describes Soil Groups B and C as materials that have slow and very slow, respectively, infiltration characteristics when thoroughly wet.

As described in this report, the site is underlain by fill soils and alluvium. The encountered materials were generally cohesive in nature and are anticipated to have very low to no infiltration. Accordingly, we recommend that the sides of the biofiltration basins be lined with an impermeable liner. In addition, site design may consider the use of pavement edge drains and cutoff curbs to reduce the potential for lateral migration of irrigation and runoff both into adjacent trench backfill, subgrade soils, or aggregate base materials beneath other improvements. Additionally, we recommend that infiltration devices incorporate an overflow pipe that is connected to an appropriate outlet.

12. PRE-CONSTRUCTION CONFERENCE

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.

13. PLAN REVIEW AND CONSTRUCTION OBSERVATION

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

14. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

15. REFERENCES

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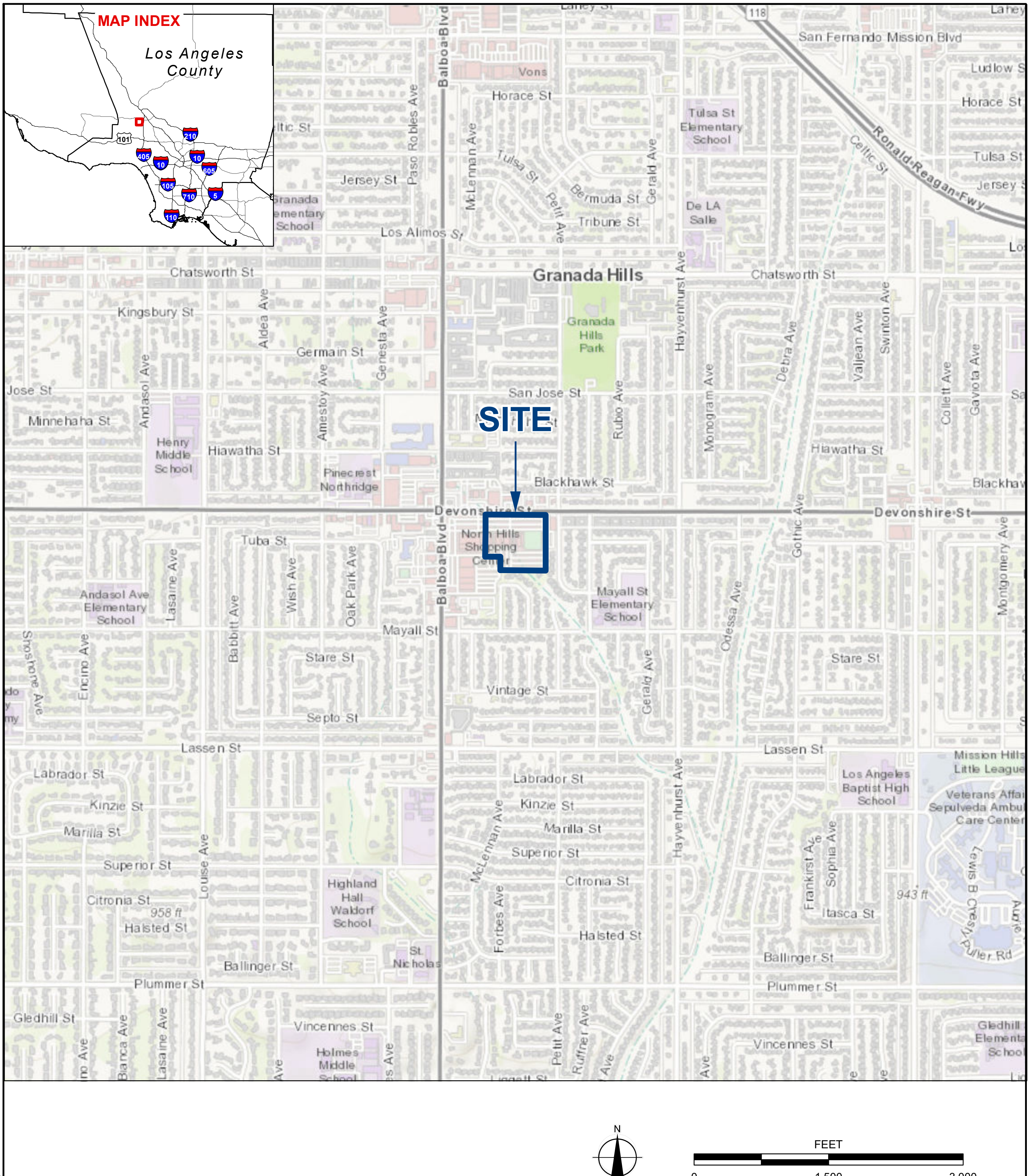
FIGURES



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2020

FIGURE 1

SITE LOCATION

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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LEGEND

SITE BOUNDARY	Qaf FILL
B-8 TD=11.5 BORING TD=TOTAL DEPTH IN FEET	Qya YOUNG ALLUVIUM
LOCATION OF OLD DRAINAGE	Qvoa VERY OLD ALLUVIUM
STORM DRAIN EASEMENT	B B' GEOLOGIC CROSS SECTION
GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN	932 TOPOGRAPHIC CONTOUR (NOAA, 2015)

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

REFERENCE: GOOGLE EARTH, 2021.
LARS ANDERSEN, 2/15/21.
HISTORIC AERIALS, 2021.

N

FEET

0 100 200

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FIGURE 2

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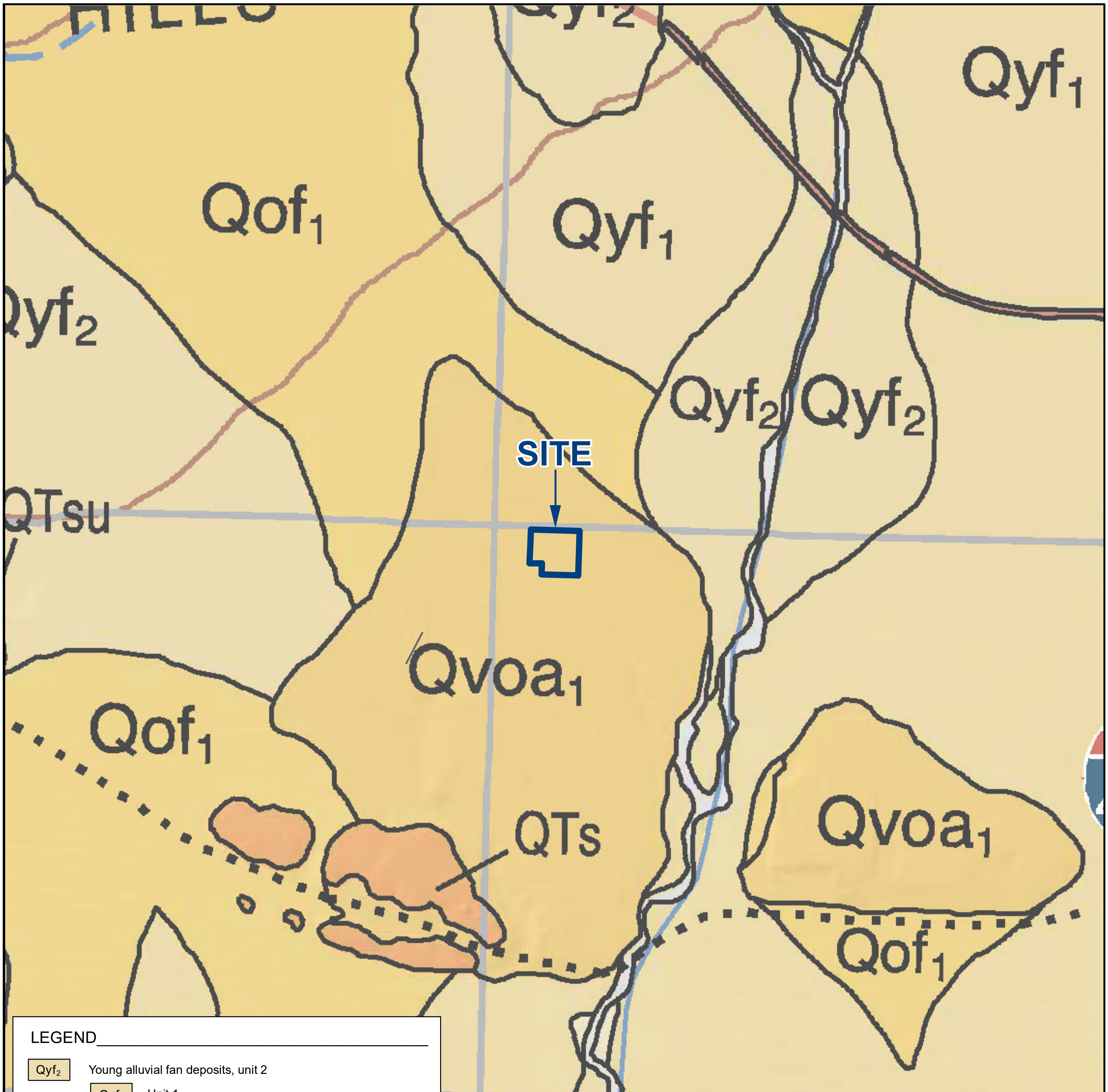
BORING LOCATIONS
HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA
108824003 | 7/21



REFERENCE: CAMPBELL ET AL., 2014, PRELIMINARYGEOLOGIC MAP OF THE LOS ANGELES 30 X 60-MINUTEQUADRANGLE, CALIFORNIA

LEGEND

Qyf_2	Young alluvial fan deposits, unit 2
Qyf_1	Unit 1
Qof_1	Old alluvial fan deposits, unit 1
$Qvoa_1$	Very old alluvium, unit 1
QTs	Saugus Formation, undivided

70 U D — Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

N

FEET

0 2,000 4,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

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GEOLOGY
HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA
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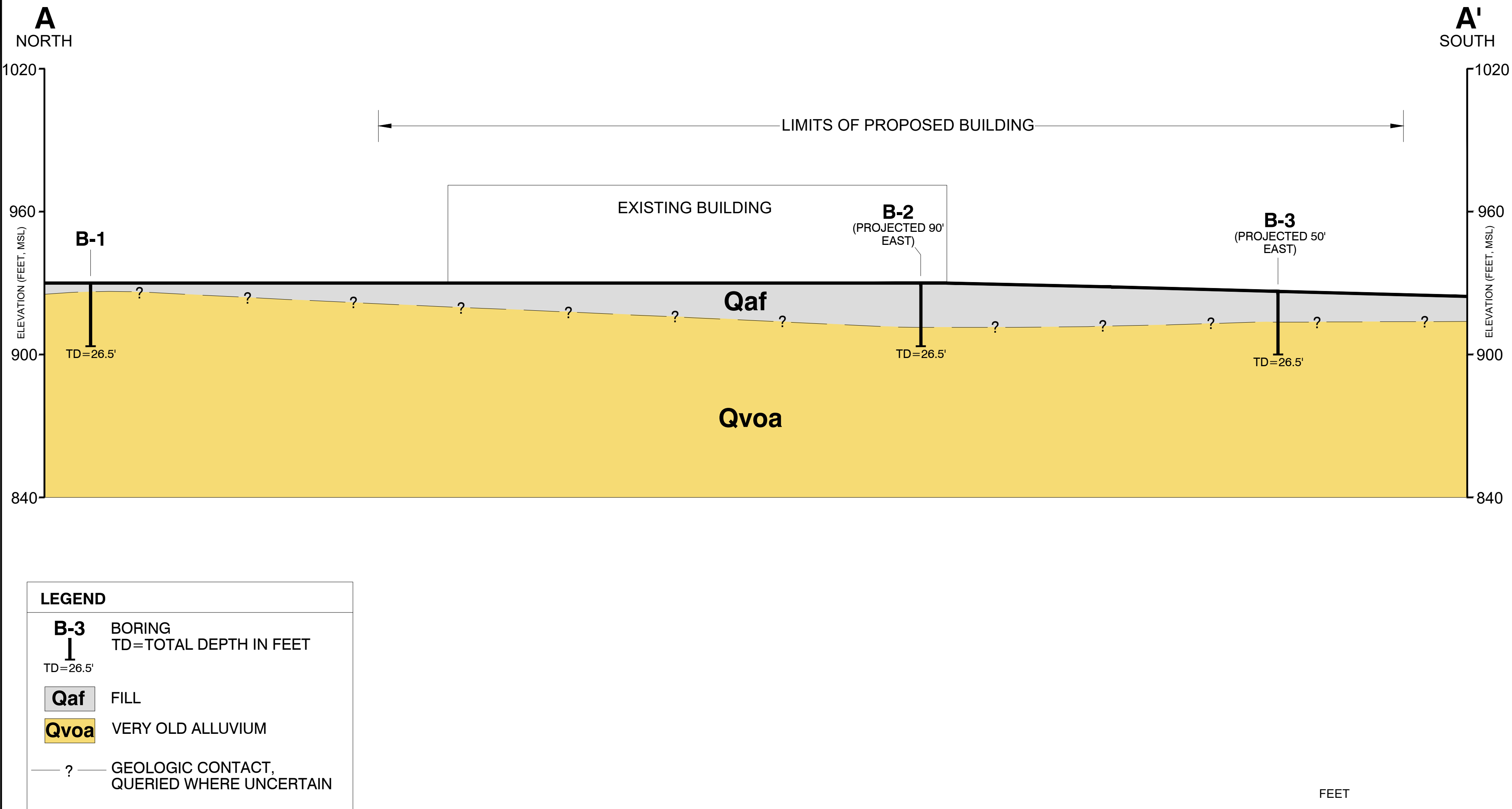

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

4A 108824003 CS-A-A'.DWG AOB

FIGURE 4A

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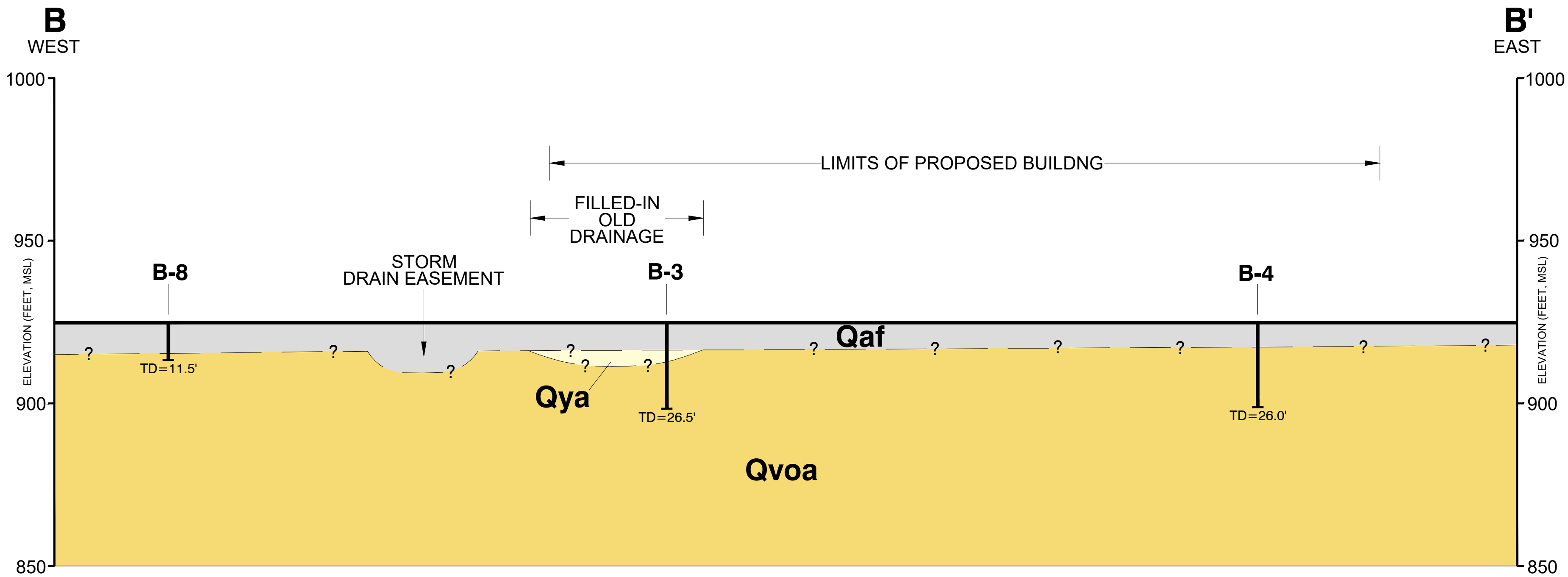
GEOLOGIC CROSS SECTION A-A'

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA
108824003 I 7/21



LEGEND

B-3 TD=26.5'	BORING TD=TOTAL DEPTH IN FEET
Qaf	FILL
Qya	YOUNG ALLUVIUM
Qvoa	VERY OLD ALLUVIUM
—— ? ——	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET
0 50 100

4B 108824003 CS-B-B'.DWG AOB

FIGURE 4B

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GEOLOGIC CROSS SECTION B-B'

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA
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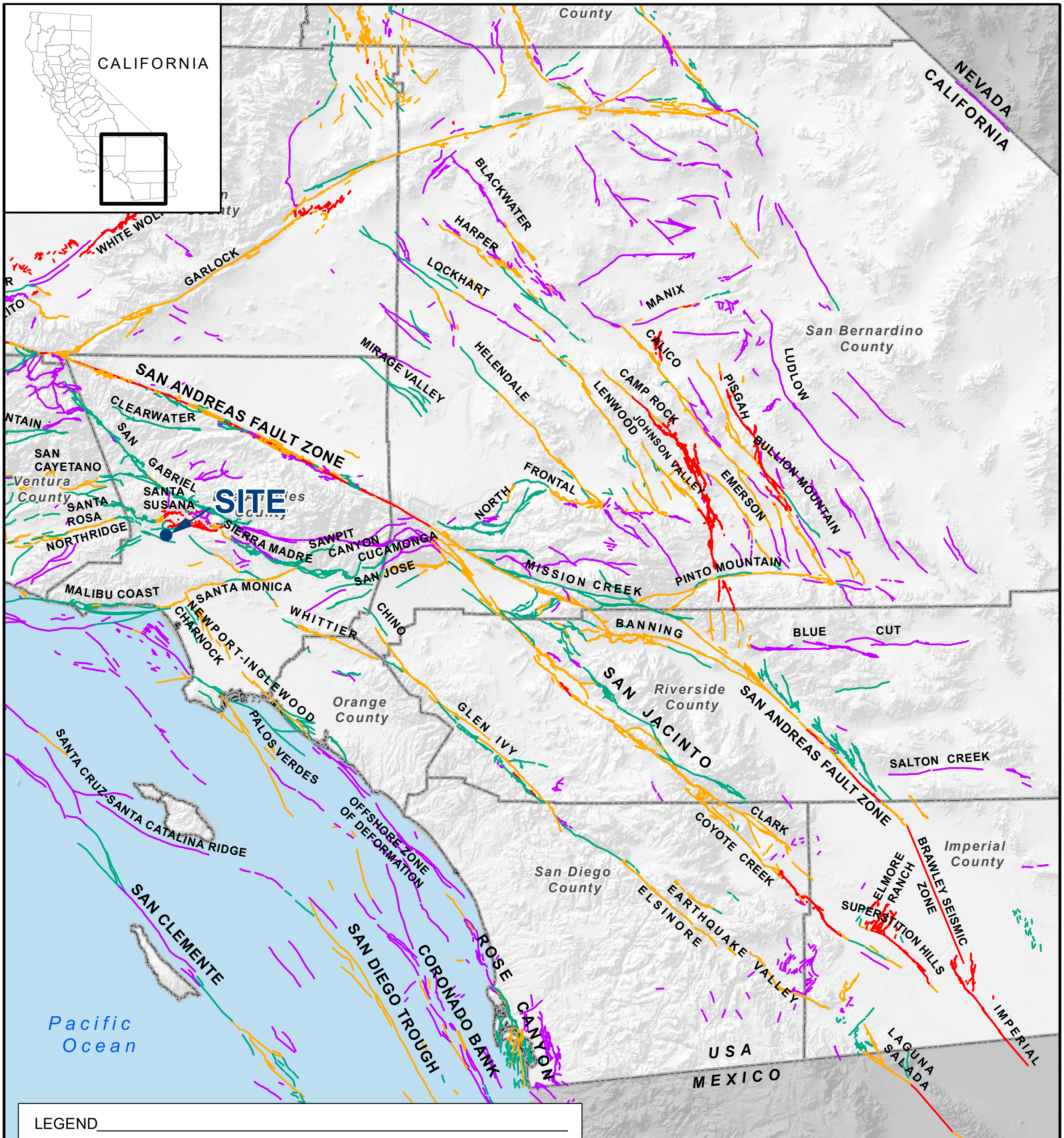

LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

N

MILES

0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 5

FAULT LOCATIONS

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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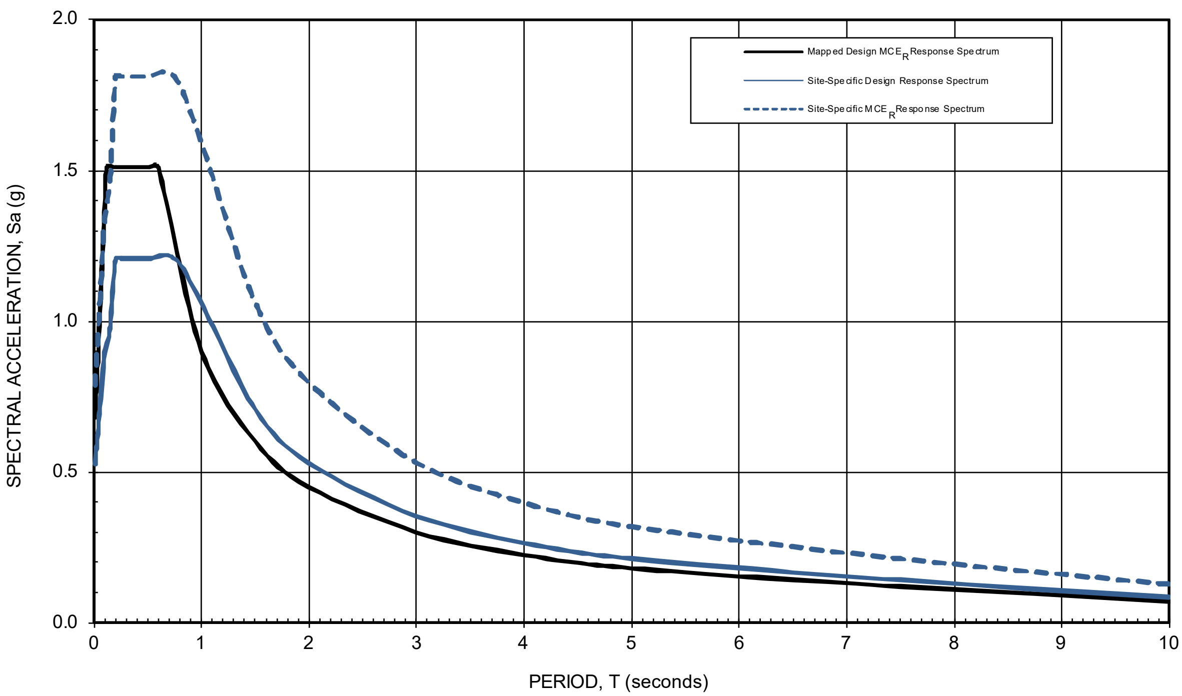
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.787	0.524
0.020	0.849	0.566
0.030	0.910	0.607
0.050	1.034	0.689
0.075	1.189	0.793
0.100	1.344	0.896
0.150	1.464	0.976
0.200	1.812	1.208
0.250	1.812	1.208
0.300	1.812	1.208
0.400	1.812	1.208

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	1.812	1.208
0.750	1.812	1.208
1.000	1.592	1.061
1.500	1.061	0.708
2.000	0.796	0.531
3.000	0.531	0.354
4.000	0.398	0.265
5.000	0.318	0.212
7.500	0.212	0.142
10.000	0.127	0.085

S_{DS} = 1.208 g	S_{D1} = 1.061 g	S_{MS} = 1.812 g	S_{M1} = 1.592 g	PGA_M = 0.821 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (20 attenuation relationships and the risk coefficients.

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering on the Santa Susana East fault zone located 6.30 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

3 The Site-Specific MCE_R Response Spectrum is the lesser of spectral ordinates of deterministic and probabilistic accelerations at each period per ASCE 7-16 Sectio 21.2.3. The Site-Specific Design Response Spectrum conforms with lower bound limit per ASCE 7-16 Section 21.3.

4 The Mapped Design MCE_R Response Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11. It is presented for the sake of comparison.

FIGURE 6

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ACCELERATION RESPONSE SPECTRA

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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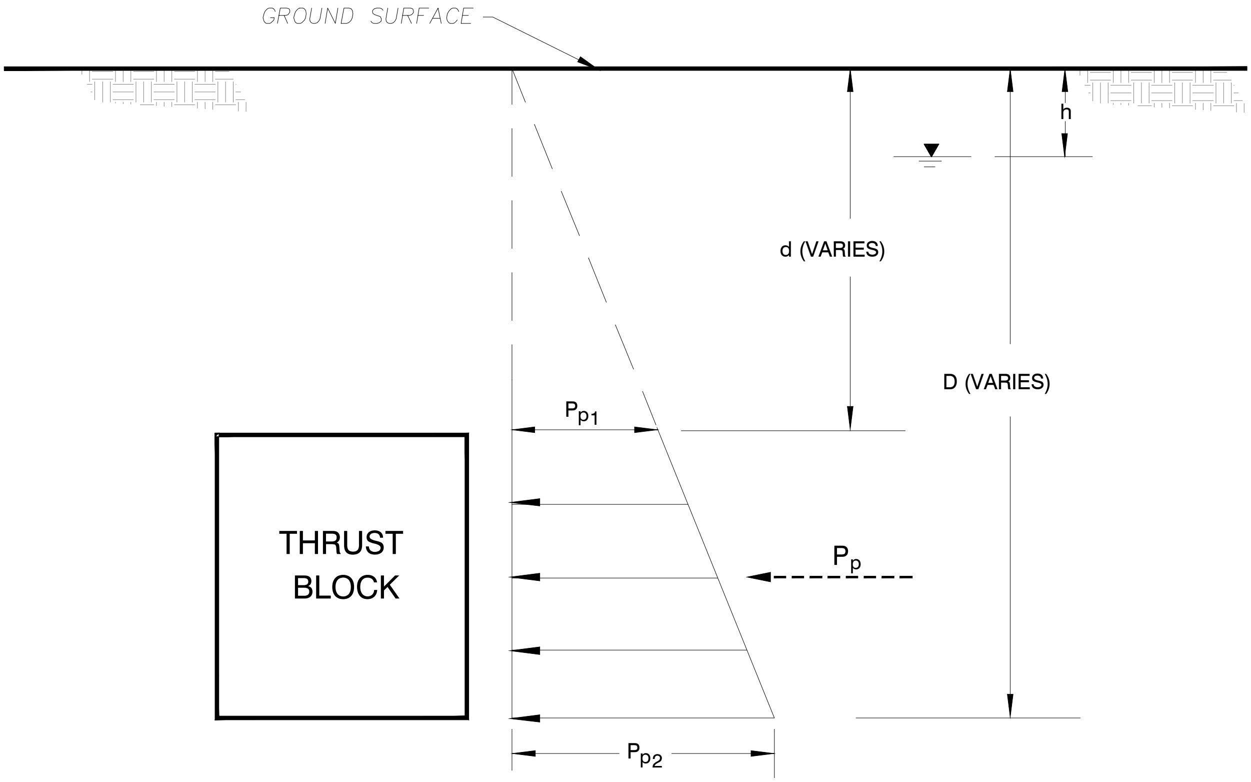

NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 185\,(D^2 - d^2)$ lb/ft

2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.5\,(D - d)[\,124.8h + 62.6\,(D+d)\,]$ lb/ft

3. ASSUMES BACKFILL IS GRANULAR MATERIAL

4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL

5. D, d AND h ARE IN FEET

6. ▼ GROUNDWATER TABLE

108824003 DETAIL FIGS.DWG AOB

NOT TO SCALE

FIGURE 7

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THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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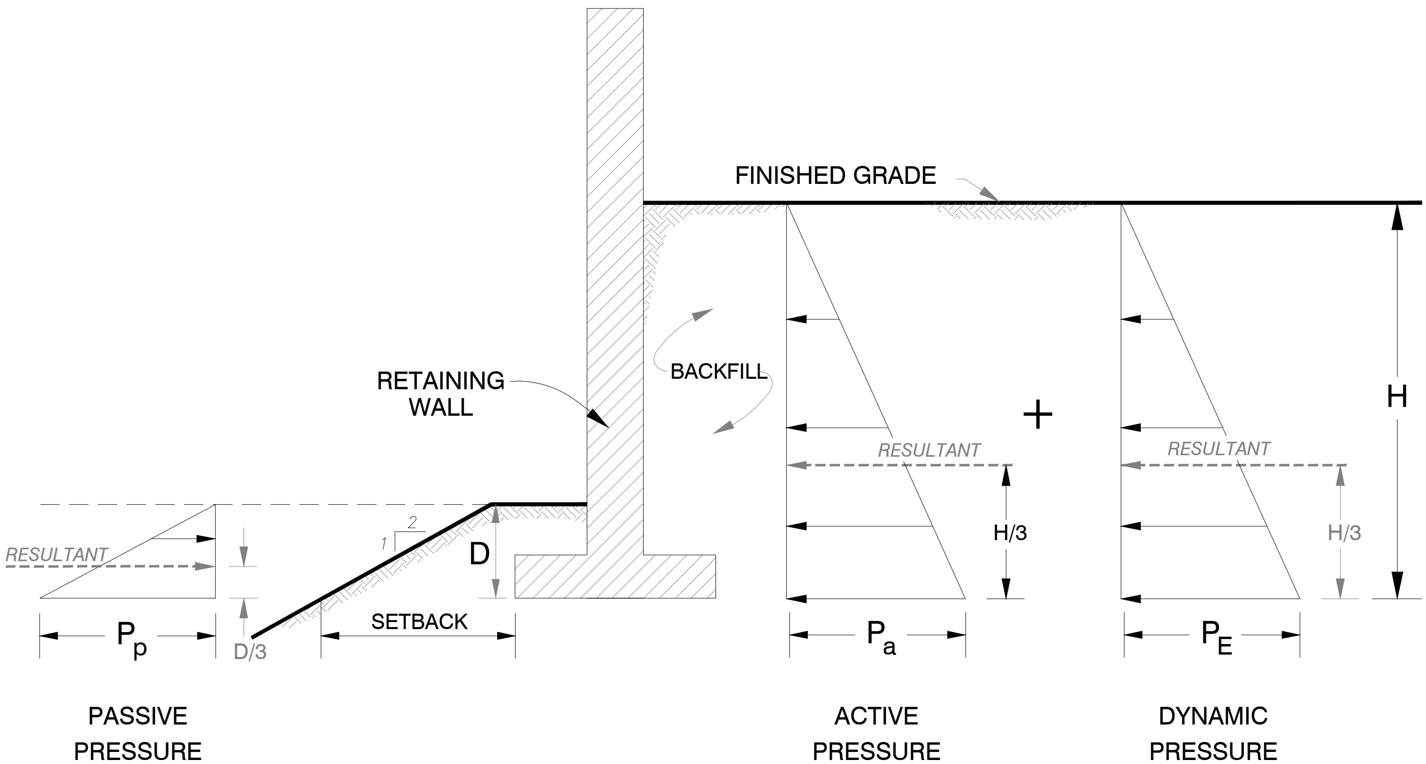

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.82 g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE CBC (2016)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_a	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	41 H	67 H
P_E	47 H	
P_p	Level Ground	2H:1V Descending Ground
	350 D	140 D

NOT TO SCALE

108824003 DETAIL FIGS.DWG AOB

FIGURE 8

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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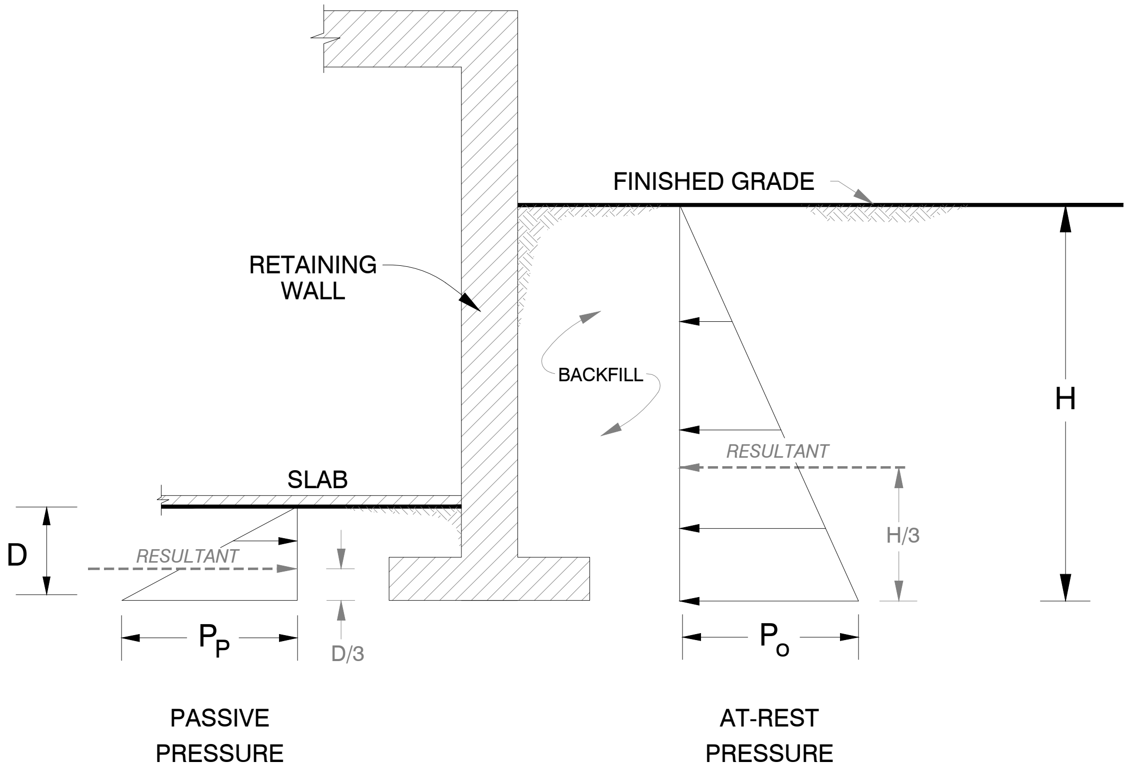

NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_O	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	62 H	90 H
P_P	Level Ground	2H:1V Descending Ground
	350 D	140 D

108824003 DETAIL FIGS.DWG AOB

NOT TO SCALE

FIGURE 9

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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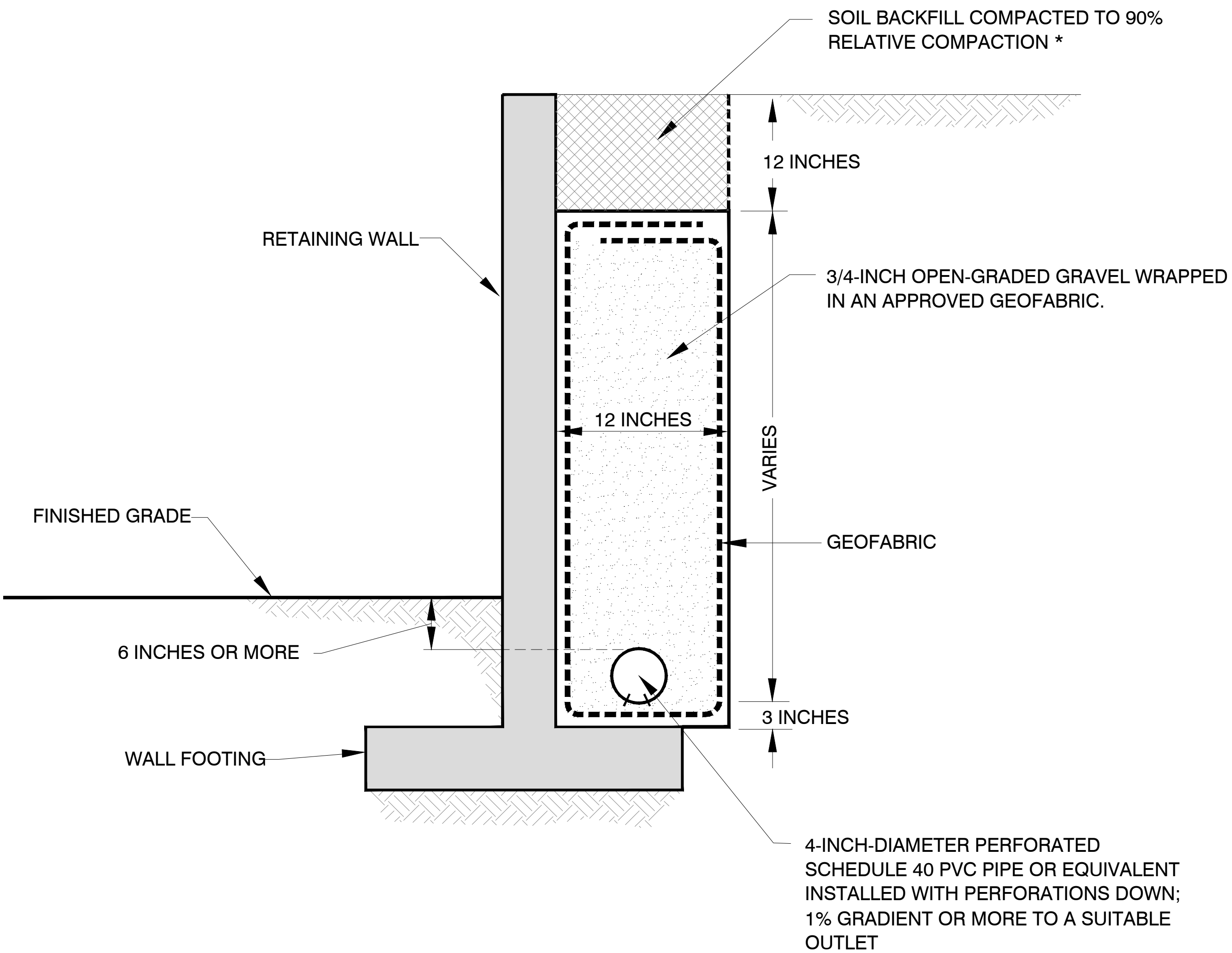

*BASED ON ASTM D1557

NOT TO SCALE

108824003 DETAIL FIGS.DWG AOB

FIGURE 10

RETAINING WALL DRAINAGE DETAIL

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD
GRANADA HILLS, CALIFORNIA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

DATE DRILLED	6/24/21	BORING NO.	B-1	
GROUND ELEVATION	930' ± (MSL)	SHEET 1 OF 1		
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Martini Drilling)			
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"	
SAMPLED BY	KMB	LOGGED BY	KMB	REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 8 inches thick.
	X					CL	AGGREGATE BASE: Brown, moist, dense, poorly graded SAND; little gravel; approximately 8 inches thick.
	X					SC	FILL: Brown, moist, dense, sandy CLAY.
		X	33	7.6	115.9	SM	VERY OLD ALLUVIUM: Brown to reddish yellow, moist, medium dense, clayey SAND; trace gravel. Brown to reddish yellow, moist, medium dense, silty SAND.
10		X	46	3.6	111.2		Dense; trace gravel.
		X	16	14.	94.6		Medium dense; increase in fines content.
20		X	40	10.9	112.8	SC	Brown to reddish yellow, moist, medium dense, clayey SAND. Trace calcium carbonate stringers.
		X	35	4.4	120.7		
30							Total Depth = 26.5 feet. Groundwater was not encountered during drilling. Backfilled with bentonite grout and patched with asphalt shortly after drilling on 6/24/21.
40							

Total Depth = 26.5 feet.
Groundwater was not encountered during drilling.
Backfilled with bentonite grout and patched with asphalt shortly after drilling on 6/24/21.

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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FIGURE A- 1

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

DATE DRILLED	6/24/21	BORING NO.	B-2
GROUND ELEVATION	930' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	KMB	LOGGED BY KMB	REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
							SP	AGGREGATE BASE: Dark brown, moist, dense, poorly graded SAND; few to little gravel; approximately 6 inches thick.
							CL	FILL: Dark brown, moist, stiff, sandy lean CLAY.
			14	19.9	107.5			Very stiff.
10			16	14.2	114.9			Asphalt fragments; mottled.
			18	14.4	110.8			Increase in sand content; trace gravel; brick fragments.
							CL	VERY OLD ALLUVIUM: Brown, moist, hard, sandy CLAY.
20			28	20.1	101.6			
							SP-SM	Brown to reddish yellow, moist, medium dense, poorly graded SAND with silt; trace gravel; interbeds of sandy clay.
			35	15.4	112.4			
30								Total Depth = 26.5 feet. Groundwater was not encountered during drilling. Backfilled with bentonite grout and patched with rapid set concrete dyed black shortly after drilling on 6/24/21. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED 6/24/21 BORING NO. B-3

GROUND ELEVATION 925' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							SP	AGGREGATE BASE: Reddish gray, moist, dense, poorly graded SAND with gravel; approximately 6 inches thick.
	X						CL	FILL: Brown, moist, stiff, sandy lean CLAY.
		X	24	15.4	114.4		SC	Reddish gray, moist, medium dense, clayey SAND; iron-oxide staining.
10		X	10	19.9	100.3		SC	YOUNG ALLUVIUM: Brown, moist, loose, clayey SAND.
		X	29	15.9	110.8		CL	VERY OLD ALLUVIUM: Brown to reddish yellow, moist, hard, sandy CLAY; calcium carbonate stringers.
20		X	32	19.0	101.6			
		X	55	19.3	108.8			
30								Total Depth = 26.5 feet. Groundwater was not encountered during drilling. Backfilled with bentonite grout and patched with asphalt shortly after drilling on 6/24/21.
								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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FIGURE A- 3

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

DATE DRILLED 6/24/21 BORING NO. B-4

GROUND ELEVATION 925' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3.5 inches thick.
							SP	AGGREGATE BASE: Dark brown, moist, dense, poorly graded SAND; few to little gravel; approximately 4.5 inches thick.
							CL	FILL: Brown to reddish yellow, moist, very stiff, sandy CLAY.
			29	14.7	114.0			Hard.
							SM	VERY OLD ALLUVIUM: Reddish yellow, moist, medium dense, silty SAND.
10			32	11.6	92.6			
							SC	Light brown to reddish yellow, moist, dense, clayey SAND; trace gravel.
			53	8.6	121.4			
20			50/5"	11.7	111.6			Very dense; calcium carbonate stringers.
			50/6"	11.4	114.7			
								Total Depth = 26 feet. Groundwater was not encountered during drilling. Backfilled with bentonite grout and patched with asphalt shortly after drilling on 6/24/21.
30								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								DATE DRILLED 6/24/21 BORING NO. B-5 GROUND ELEVATION 930' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Martini Drilling) DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30" SAMPLED BY KMB LOGGED BY KMB REVIEWED BY NMM
								ASPHALT CONCRETE: Approximately 4 inches thick.
							SP	FILL: Brown, moist, medium dense, poorly graded SAND; trace gravel; approximately 5 inches thick.
								PORTLAND CEMENT CONCRETE: Approximately 5 inches thick.
	X						CL	FILL: Dark brown to reddish gray, moist, firm, sandy lean CLAY.
		X	12	17.1	109.2		SM	Dark brown to reddish gray, moist, loose, silty SAND.
10		X	8					Reddish brown.
								Total Depth = 11.5 feet. Groundwater was not encountered during drilling. Backfilled and patched with asphalt shortly after drilling on 6/24/21. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 6/24/21 BORING NO. B-6

GROUND ELEVATION 930' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY NMM

DESCRIPTION/INTERPRETATION

DEPTH (feet)	Bulk	Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 4 inches thick.
						GP	AGGREGATE BASE: Dark brown, moist, very dense, poorly graded GRAVEL; approximately 6 inches thick.
	X					SC	FILL: Dark brown, moist, medium dense, clayey SAND. Brown to red.
		X	9	17.6	108.2	SM	Brown, moist, loose, silty SAND.
10		X	19			SC	VERY OLD ALLUVIUM: Brown to red, moist, medium dense, clayey SAND.
20							
30							
40							

Total Depth = 11.5 feet.
Groundwater was not encountered during drilling.
Backfilled and patched with asphalt shortly after drilling on 6/24/21.

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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FIGURE A- 6

DATE DRILLED	6/24/21
BORING NO.	B-7
GROUND ELEVATION	930' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Martini Drilling)
DRIVE WEIGHT	140 lbs. (Auto-Trip)
DROP	30"
SAMPLED BY	KMB
LOGGED BY	KMB
REVIEWED BY	NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 3 inches thick.
	X						CL	FILL: Dark brown to reddish brown, moist, stiff, sandy lean CLAY.
	X						SM	Dark brown, moist, medium dense, silty SAND; trace clay.
		X	20	14.8	117.7			
10		X	18				SC	Dark brown to red, moist, medium dense, clayey SAND; trace gravel.
20								
30								
40								

Total Depth = 11.5 feet.
Groundwater was not encountered during drilling.
Backfilled and patched with asphalt shortly after drilling on 6/24/21.

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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FIGURE A- 7

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

DATE DRILLED 6/24/21	BORING NO. B-8	
GROUND ELEVATION 925' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Martini Drilling)		
DRIVE WEIGHT 140 lbs. (Auto-Trip)	DROP 30"	
SAMPLED BY KMB	LOGGED BY KMB	REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							SP	AGGREGATE BASE: Reddish gray, moist, dense, poorly graded SAND with gravel; approximately 4 inches thick.
	X						SC	FILL: Reddish yellow, moist, medium dense, clayey SAND.
		X	56	13.9	115.0			Dense.
10		X	23				SP-SC	VERY OLD ALLUVIUM: Light brown, moist, medium dense, poorly graded SAND with clay.
								Total Depth = 11.5 feet. Groundwater was not encountered during drilling. Backfilled and patched with asphalt shortly after drilling on 6/24/21.
								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

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APPENDIX B

Geotechnical Laboratory Testing

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. This test result was utilized in evaluating the soil classifications in accordance with the USCS.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-2 through B-4.

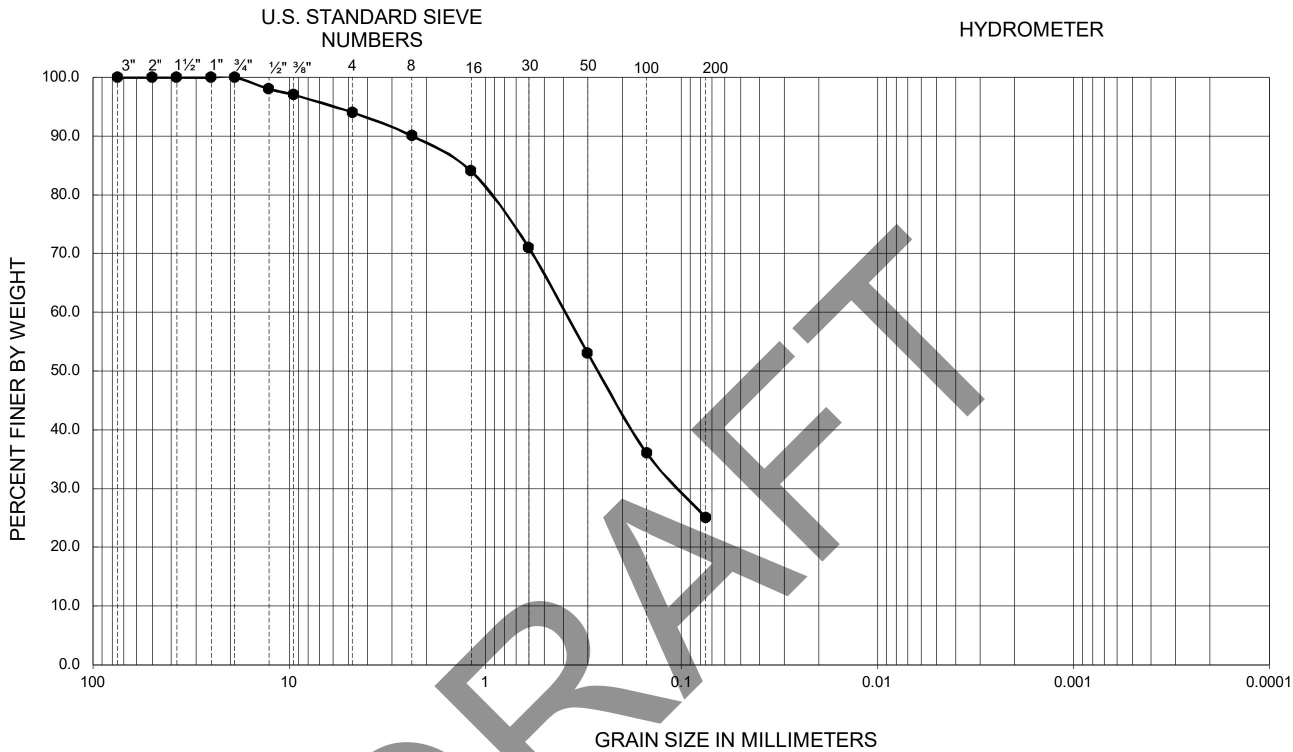
Direct Shear Tests
A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-5.

Expansion Index Tests
The expansion indices of selected materials were evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 psf and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the tests are presented on Figure B-6.

Soil Corrosivity Tests
Soil pH and electrical resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and 422, respectively. The test results are shown on Figure B-7.

R-Value
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-8.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	5.0-6.5	--	--	--	--	--	--	--	--	25	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

GRADATION TEST RESULTS

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DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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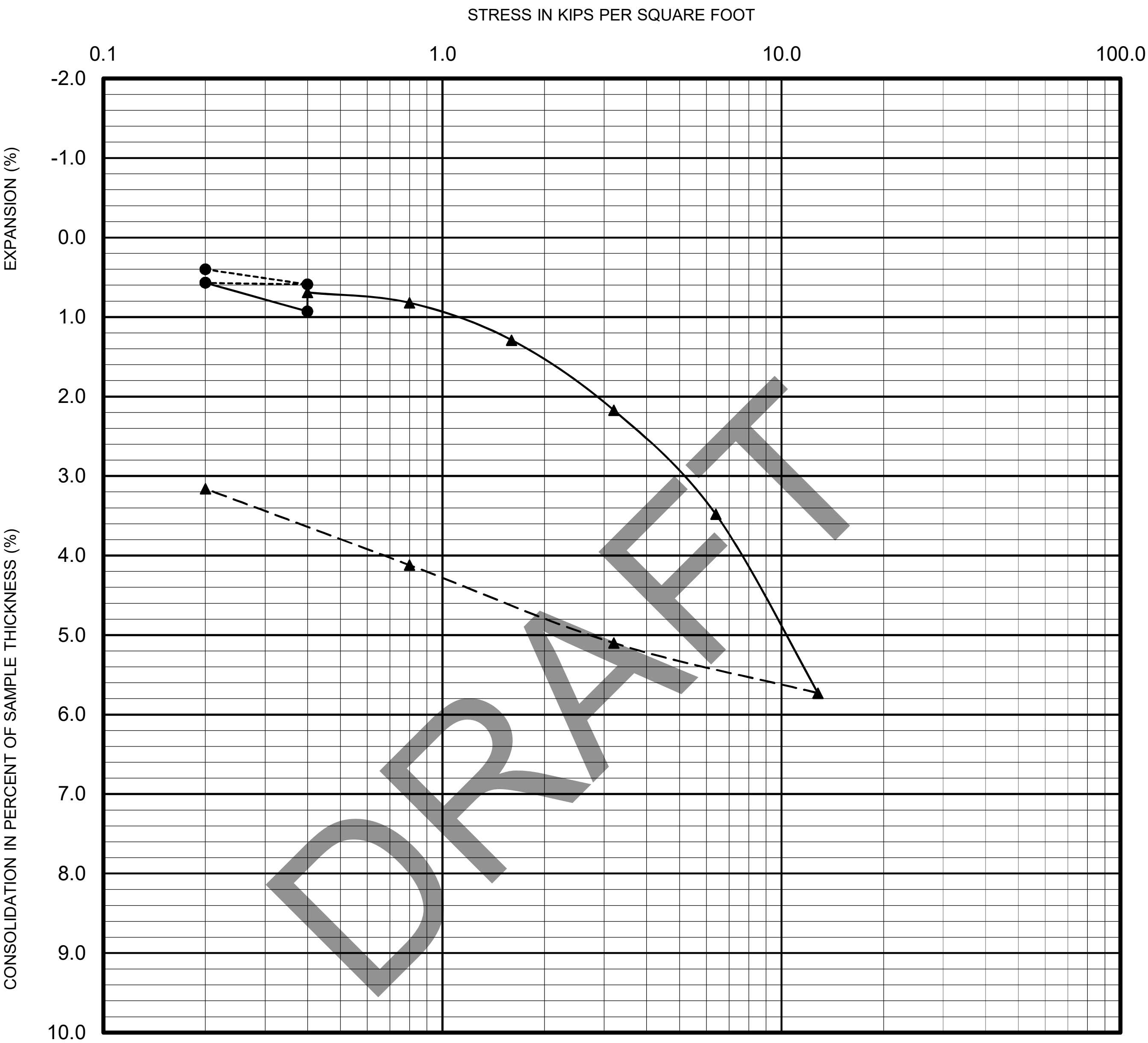

Seating Cycle		Sample Location	B-2
Loading Prior to Inundation		Depth (ft)	5.0-6.5
Loading After Inundation		Soil Type	CL
Rebound Cycle			

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-2

CONSOLIDATION TEST RESULTS

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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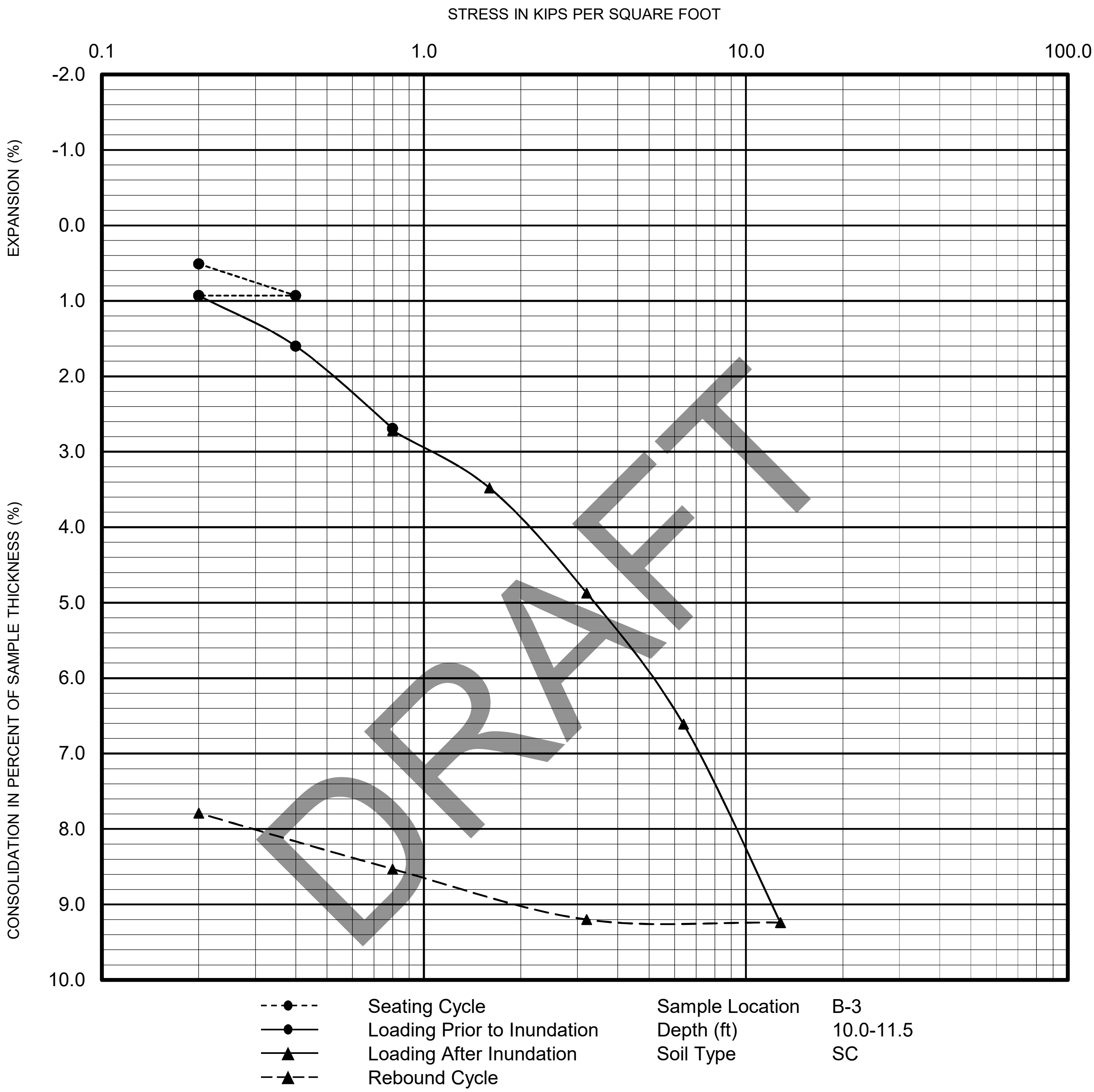

Sample Location	B-3
Depth (ft)	10.0-11.5
Soil Type	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-3

CONSOLIDATION TEST RESULTS

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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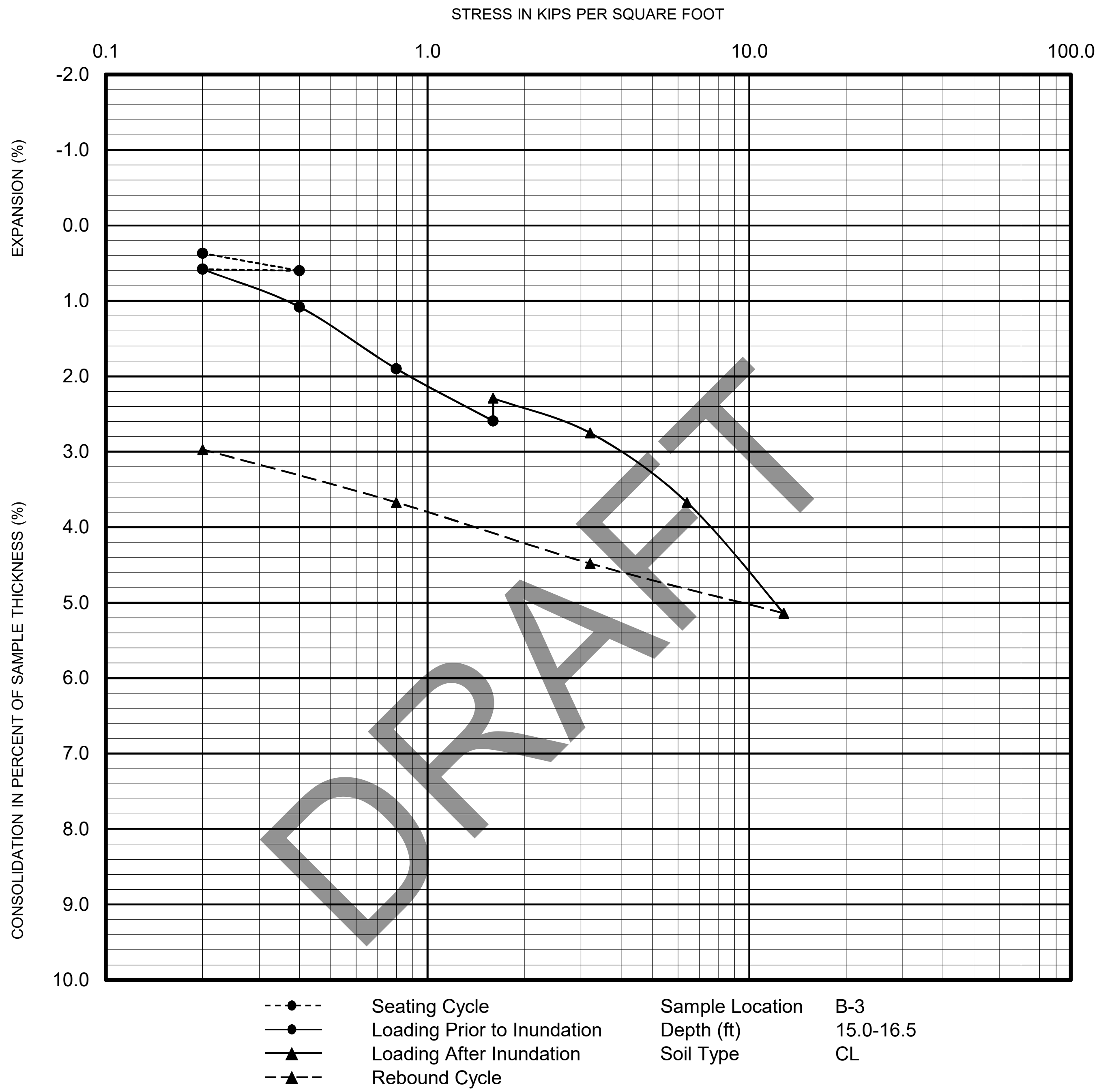

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-4

CONSOLIDATION TEST RESULTS

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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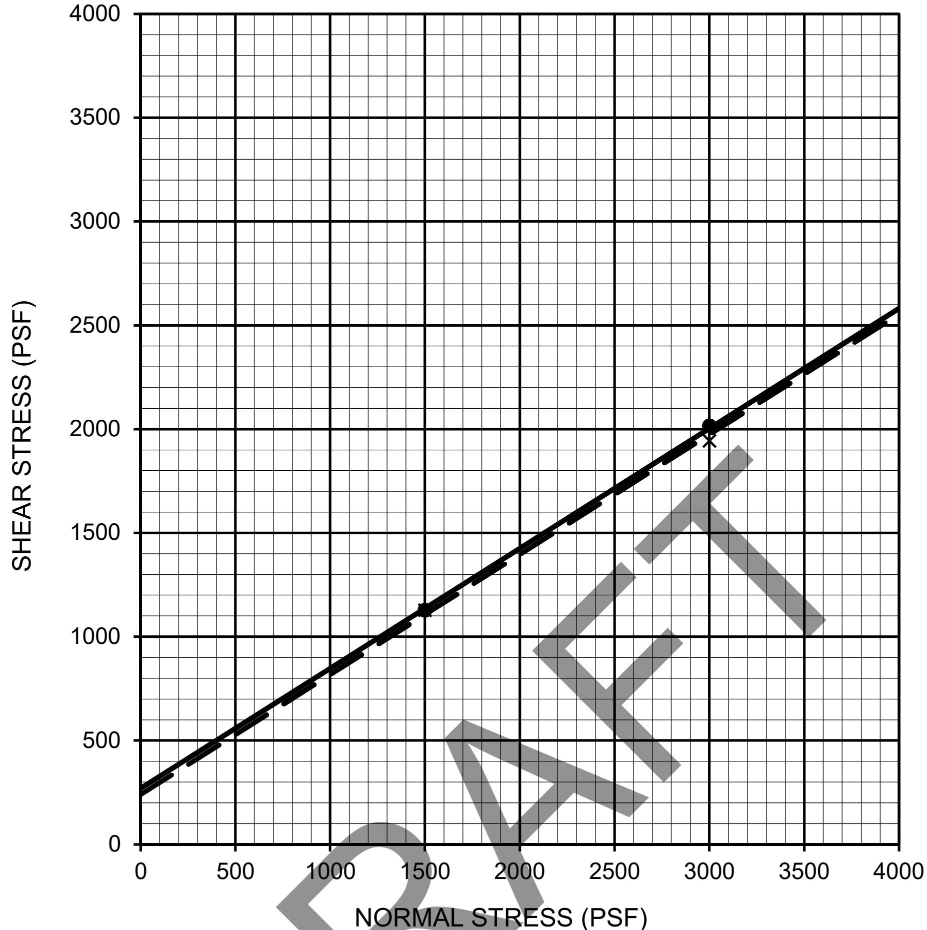

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Sandy Lean CLAY	——●——	B-2	5.0-6.5	Peak	270	30	CL
Sandy Lean CLAY	– – X – –	B-2	5.0-6.5	Ultimate	240	30	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-5

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DIRECT SHEAR TEST RESULTS

HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	1.0-3.0	10.5	107.4	21.6	0.042	42	Low
B-2	1.0-5.0	9.6	110.4	20.1	0.031	31	Low
B-3	2.5-3.5	9.5	112.7	17.7	0.017	17	Very Low
B-4	2.5-3.5	12.2	102.2	28.0	0.094	94	High

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE B-6

EXPANSION INDEX TEST RESULTS

HOME DEPOT STORE - GRANADA HILLS
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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	1.0-3.0	7.5	1,000	20	0.002	30
B-4	2.5-3.5	7.2	755	20	0.002	30

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-7

CORROSIVITY TEST RESULTS

HOME DEPOT STORE - GRANADA HILLS
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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-5	1.0-5.0	Sandy Lean CLAY (CL)	41
B-7	1.0-5.0	Sandy Lean CLAY (CL)	19

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-8

R-VALUE TEST RESULTS

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HOME DEPOT STORE - GRANADA HILLS
DEVONSHIRE STREET AND BALBOA BOULEVARD, GRANADA HILLS, CALIFORNIA

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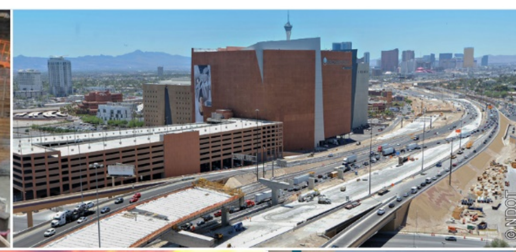

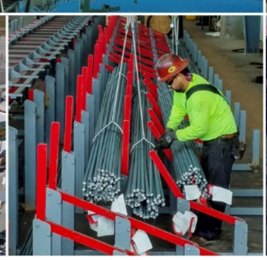


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