Appendix C

Geotechnical Investigation Reportt

GEOTECHNICAL INVESTIGATION REPORT

ASHLEY HOME STORE EXPANSION

855 Ashley Way
City of Colton, San Bernadino County, California

CONVERSE PROJECT NO. 21-81-269-01

Prepared For:

HMC CONSTRUCTION, INC.
1461 East Cooley Drive
Colton, California 92324

Presented By:

CONVERSE CONSULTANTS
2021 Rancho Drive, Suite 1
Redlands, CA 92373
909-796-0544

April 8, 2022

April 8, 2022

Mr. Daniel E. Wallner
Vice President
HMC Construction, Inc.
1461 E. Cooley Drive
Colton, California 92324

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
Converse Project No. 21-81-269-01

Dear Mr. Wallner:

Converse Consultants (Converse) is pleased to submit this Geotechnical Investigation Report to assist with the proposed Ashley Home Store Expansion project located at 855 Ashley Way, in the City of Colton, San Bernardino County, California. This report was prepared in accordance with our proposal dated October 29, 2021 and your approval of our proposal dated November 3, 2021.

Based upon our field investigation, laboratory data, and analyses, the project site is considered feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and development of the project.

We appreciate the opportunity to be of service to HMC Construction, Inc. Should you have any questions, please do not hesitate to contact us at 909-796-0544.

CONVERSE CONSULTANTS

Hashmi S. E. Quazi, PhD, GE, PE
Principal Engineer

Dist.: 1/Addressee (e-mail)
HSQ/RLG/CN/kvg

PROFESSIONAL CERTIFICATION

This report has been prepared by the individuals whose seals and signatures appear herein.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering, engineering geologic principles, and practice in this area of Southern California. There is no warranty, either expressed or implied.

Catherine Nelson, GIT
Senior Staff Geologist

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Principal Engineer

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1.0 INTRODUCTION

This geotechnical investigation was prepared by Converse for the Ashley Home Store Expansion project located at 855 Ashley Way, in the City of Colton, San Bernardino County, California. The approximate location of the proposed project is shown in Figure No. 1, *Approximate Project Location Map.*

The purpose of this investigation was to evaluate the current nature and engineering properties of the subsurface soils and groundwater conditions and to provide geotechnical recommendations for the proposed commercial building expansion.

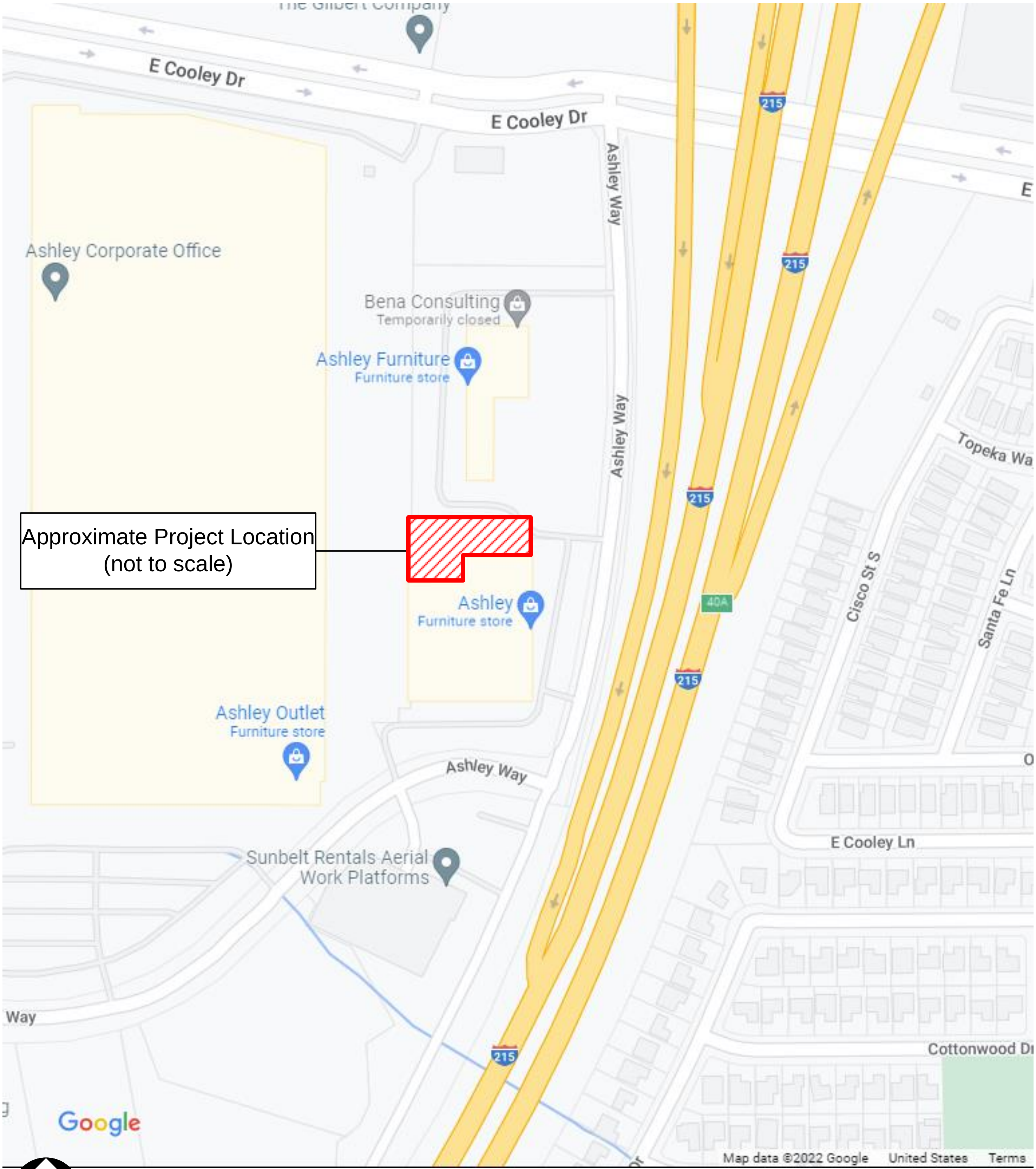
This report was prepared for the project described herein and is intended for use solely by HMC Construction and their authorized agents. This report may be made available to the prospective bidders for bidding purposes. However, the bidders are responsible for their own interpretation of the site conditions between and beyond the boring locations, based on factual data contained in this report. This report may not contain sufficient information for use by others and/or other purposes.

2.0 SITE DESCRIPTION AND BACKGROUND

The proposed "L' shaped building expansion is located on the north side of the existing Ashley Home Store at 855 Ashley Way, in the City of Colton, San Bernardino County, California. The site is bounded on the south by the existing Ashley Home Store and on the north, east and west by parking and roadway areas. Currently the site is a portion of an existing asphalt parking lot and concrete loading dock area.

The site is relatively flat and slopes gently to the north and west. Site elevations range from approximately 956 feet above mean sea level (msl) in the east portion of the site to approximately 950 feet above msl in the northwest portion of the site.

The present site conditions are depicted in Photographs Nos. 1 through 3.



Approximate Project Location Map

Project: Ashley Home Store Expansion
Location: 855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

PROJECT NO.
21-81-269-01

Converse Consultants

FIGURE NO.
1

Photograph No. 1: Current site conditions at the existing parking lot, facing west.

Photograph No. 2: Current site conditions at the existing parking lot, facing east.

Photograph No. 3: Current site conditions at the existing loading dock, facing southwest.

3.0 PROJECT DESCRIPTION

Based on conversations with Daniel E. Wallner of HMC Construction, Inc. and the referenced conceptual site plans provided, we understand the property will be developed for an Ashley Home Store expansion. The new proposed building footprint is approximately 35,000 square feet. The structure will be a 2-story building and likely be founded on shallow footings with concrete tilt-up wall and slab-on-grade construction. Associated with the development will be below ground utilities and parking areas, as well as possible retaining walls. No grading plans are available, at this time. However, grading is anticipated to have cuts and fills of up to 3 feet or less.

4.0 SCOPE OF WORK

The scope of Converse's investigation is described in the following sections.

4.1 Project Set-up

The project set-up consisted of the following tasks.

- Review of existing plans, publications and data relevant to the project.

- Conducted a site reconnaissance to mark the exploration locations and to verify drill rig access to the proposed locations were available.
- Notified Underground Service Alert (USA) at least 48 hours prior to conducting field work to clear the exploration locations of any conflict with existing underground utilities.
- Engaged a California-licensed drill rig company to conduct exploratory borings.

4.2 Subsurface Exploration

Three exploratory borings (BH-01 through BH-03) were drilled using a truck-mounted CME 75 drill rig equipped with 8-inch diameter hollow-stem augers to investigate the subsurface conditions on February 18, 2022. The borings were drilled to depths ranging from 16.5 feet to 51.5 feet below the existing ground surface (bgs).

The approximate locations of the current exploratory borings are shown on Figure No. 2, *Approximate Boring and Overexcavation Locations Map*. A detailed discussion of the subsurface exploration is presented in Appendix A, *Field Exploration*.

4.3 Laboratory Testing

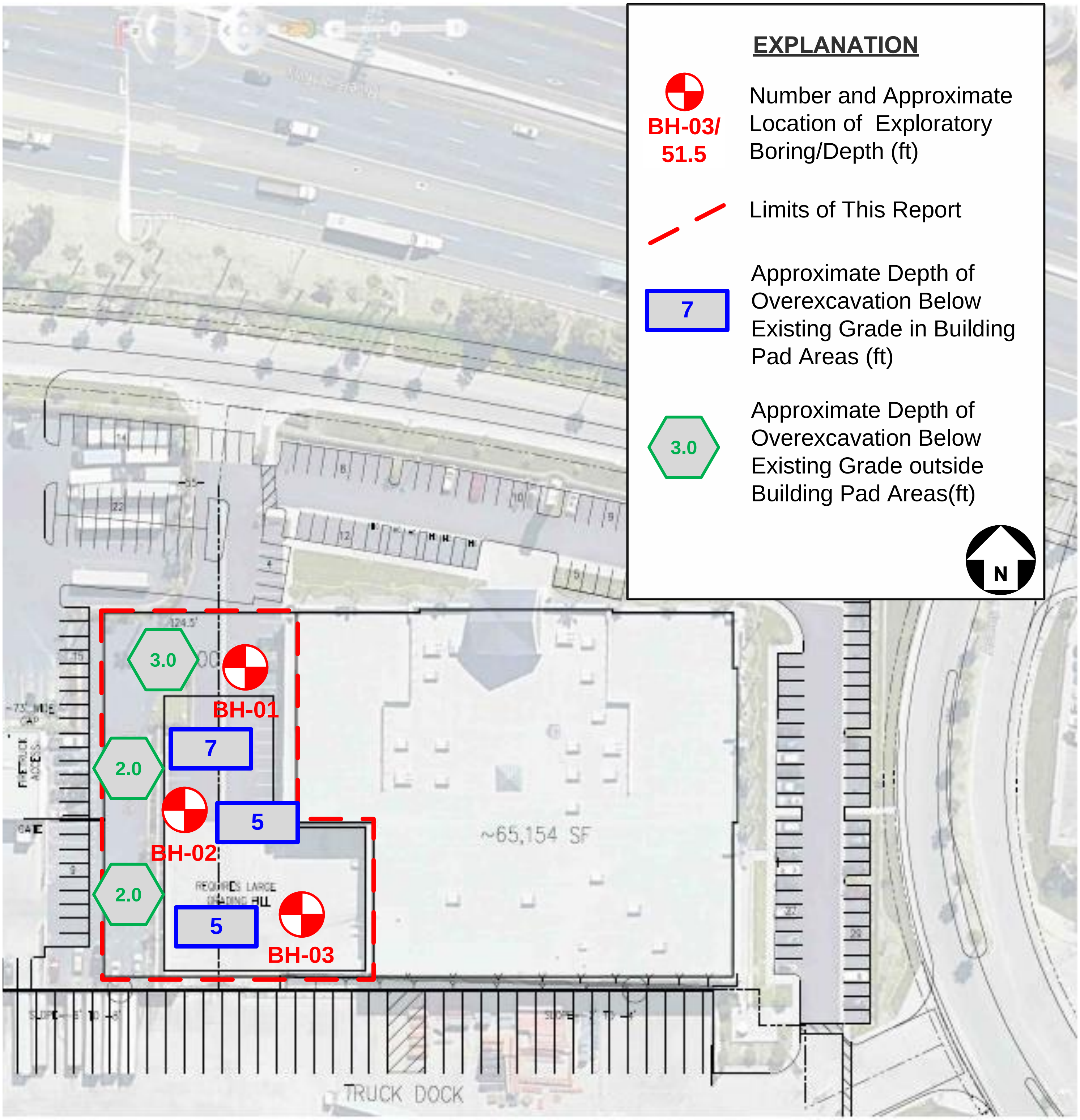
Representative samples of the site soils were tested in the laboratory to aid in soil classification, and to evaluate relevant engineering properties. These tests included the following.

- *In-situ* moisture contents and dry densities (ASTM D2216 and D2937)
- Expansion Index (ASTM D4829)
- R-value (California Test 301)
- Soils Corrosivity (CTM 643, 422, 417, 532)
- Grain size analysis (ASTM D6913)
- Maximum dry density and optimum-moisture content (ASTM D1557)
- Direct shear (ASTM D3080)
- Consolidation (ASTM D2435)

For *in-situ* moisture and dry density data, see the logs of boring in Appendix A, *Field Exploration*. For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

4.4 Report Analysis and Preparation

Data obtained from the field exploration and laboratory testing program was assembled and evaluated. Geotechnical analyses of the compiled data were performed, followed by the preparation of this report to present our findings, conclusions, and recommendations for the project.



Approximate Boring and Overexcavation Locations Map

Project: Ashley Home Store Expansion
Location: 855 Ashley Way
City of Colton, San Bernardino County, California

Project No. **21-81-269-01**

For: HMC Construction, Inc.

Converse Consultants

Figure No. 2

5.0 SITE CONDITIONS

A general description of the subsurface conditions, various materials and groundwater conditions encountered at the site during our field exploration is discussed below.

5.1 Subsurface Profile

Based on our field exploration and research, the subsurface soil at the project site consisted of Holocene-aged alluvial fan deposits in all borings. This material was generally comprised of sand, silty sand, sandy silt and some sandy clay, which was fine-grained or fine to coarse-grained, with few gravel up to 1-inch maximum dimension, slightly to very desiccated, oxidation staining, pinhole porosity, loose to very dense/soft to very stiff, moist and various shades of brown, orange, and gray.

For a detailed description of the subsurface materials encountered in the exploratory borings, see the logs in Drawing Nos. A-2 through A-4, in Appendix A, *Field Exploration.*

5.2 Groundwater

Groundwater was not encountered during our field investigation to the maximum explored depth of 51.5 feet. For comparison, regional and national groundwater databases were reviewed for nearby groundwater statistics. As addressed in the regional geology section below, the San Jacinto Fault approximately one quarter of a mile to the north acts as a groundwater barrier for subsurface water that typically would be flowing from northeast to southwest beneath this project site. Therefore, results from sites north of the fault were not included in presented data as they vary significantly from the groundwater conditions at the project site.

The GeoTracker database (SWRCB, 2022) was reviewed to evaluate the current and historical groundwater levels within a 1.0 mile radius of the project site. Two sites with groundwater data were identified within a 1.0-mile radius of the project site south of the fault. Details of those records are presented below.

- ARCO #6144 (T0607116745), located approximately 3,986 feet south of the project site, most recently reported groundwater depths ranging from 60.21 to 73.89 feet bgs in 2003 and 2004

The National Water Information System (USGS, 2022) was reviewed for current and historical groundwater data from sites within an approximately 1.0-mile radius of the project site and no groundwater data was identified.

The California Department of Water Resources database (DWR, 2022) was reviewed for historical groundwater data from sites within a 1.0-mile radius of the project site. One site was identified within a 1.0-mile radius of the project site that contained groundwater elevation data. Details of that record are listed below.

- Well No. 01S04W27M002S (Station 3400544N1172947W002), located approximately 1,815 feet southeast of the project site, most recently reported groundwater at depths ranging from 81.9 to 101.62 feet bgs between 2005 and 2010.

Based on the available data and the findings of our investigation, the historical high groundwater level and the current groundwater level is estimated to be deeper than approximately 60 feet to 73 feet bgs. Groundwater is not expected to be encountered during construction of the proposed project. It should be noted that the groundwater level could vary depending upon the seasonal precipitation and possible groundwater pumping activity in each site vicinity. Perched water layers at depth may be present locally, particularly following high precipitation and irrigation events.

5.2 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade. Depending on the extent and location below finish subgrade, expansive soils can have a detrimental effect on structures.

Based on our laboratory test result, the expansion index of the upper 10 feet of site soils was 34, corresponding to a low expansion potential.

5.3 Collapse Potential

Soil deposits subjected to collapse/hydro-consolidation generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Moreover, some soils may have a different degree of collapse/hydro-consolidation based on the amount of proposed fill or structure loads. Soils susceptible to collapse/ hydro-consolidation include wind-blown silt, weakly cemented sand, and silt where the cementing agent is soluble (e.g., soluble gypsum, halite), alluvial or colluvial deposits within semi-arid to arid climate, and certain weathered bedrock above the groundwater table.

Granular soils may have a potential to collapse upon wetting in arid climate regions. Collapse/hydro-consolidation may occur when the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose/low density configuration from deposition.

The degree of collapse of a soil can be defined by the collapse potential value, which is expressed as a percent of collapse of the total sample using the Collapse Potential Test (ASTM D4546). According to the ASTM guideline, the severity of collapse potential is commonly evaluated by the following Table No. 1, *Collapse Potential Values*.

Table No. 1, Collapse Potential Values

Collapse Potential Value (%)	Severity of Problem
0	None
0.1 to 2	Slight
2.1 to 6.0	Moderate
6.0 to 10.0	Moderately Severe
>10	Severe

Two consolidation tests were conducted for this project. A collapse potential of 0.2 percent at a depth of 5.0 feet bgs in boring BH-03 and a collapse potential of 0.7 percent at a depth of 10.0 feet bgs in boring BH-03 were measured. These indicate only a slight problem at the site. Collapse potential distress is typically considered a concern when collapse potential is over 2% (LA County, 2013).

5.4 Excavatability

The subsurface materials of the project are expected to be excavatable by conventional heavy-duty earth moving and trenching equipment.

The phrase “conventional heavy-duty excavation equipment” is intended to include commonly used equipment such as excavators, scrapers, and trenching machines. It does not include hydraulic hammers (“breakers”), jackhammers, blasting, or other specialized equipment and techniques used to excavate hard earth materials. Selection of an appropriate excavation equipment models should be done by an experienced earthwork contractor.

5.5 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface soil conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations.

5.6 Caving

Caving was not encountered in any of the exploratory borings. Localized caving may occur in excavations that extend into granular soils that are encountered on-site.

6.0 ENGINEERING GEOLOGY

The regional and local geology within the proposed project area is discussed below.

6.1 Regional Geology

The project site is located within the northern Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges Geomorphic Province consists of a series of northwest-trending mountain ranges and valleys bounded on the north by the San Bernardino and San Gabriel Mountains, on the west by the Los Angeles Basin, and on the south by the Pacific Ocean.

The province is a seismically active region characterized by a series of northwest-trending strike-slip faults. The most prominent of the nearby fault zones include the San Jacinto, Cucamonga, and San Andreas Fault Zones, all of which have been known to be active during Quaternary time.

Topography within the province is generally characterized by broad alluvial valleys separated by linear mountain ranges. This northwest-trending linear fabric is created by the regional faulting within the granitic basement rock of the Southern California Batholith. Broad, linear, alluvial valleys have been formed by erosion of these principally granitic mountain ranges.

The project site is located at the southern edge of the Santa Ana River Basin at the western tip of the San Timoteo Badlands. The Santa Ana River is located approximately a half-mile west of the project site location. The San Jacinto Fault Zone is located approximately one-tenth of a mile north of the project site. This fault is a north to northwest trending reverse fault that dips steeply towards the southwest. The San Jacinto Fault acts as a groundwater barrier along the southern margin of the Santa Ana River Basin.

6.2 Local Geology

The project site is primarily underlain by Middle Holocene young alluvial-fan deposits, consisting of slightly to moderately consolidated clay, silt, and sand, having slightly to moderately dissected surfaces.

6.2 Flooding

Review of National Flood Insurance Rate Maps indicates that the project site has two different flood hazard zone designations. The western portion of the site is within a Flood Hazard Zone "X" designated to be an area of minimal flood hazard. The eastern portion of the site is also in a zone “X” but is designated as an area with a 0.2 percent annual chance flood hazard (FEMA, 2008).

7.0 FAULTING AND SEISMICITY

The location of the site with respect to active faults and associated seismicity is discussed below.

7.1 Faulting

The project site is situated in a seismically active region. As is the case for most areas of Southern California, ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project is high.

The project site is not located within a currently mapped State of California or San Bernadino County Earthquake Fault Zone for surface fault rupture.

Table No. 2, *Summary of Regional Faults,* summarizes selected data of known faults capable of seismic activity within 50 kilometers of the proposed project site (coordinate 34.0574N and 117.2997W). The data presented below was calculated using the National Seismic Hazard Maps Database and other published geologic data.

Table No. 2, Summary of Regional Faults

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
San Jacinto	1.38	strike slip	241	n/a	7.88
S. San Andreas	12.41	strike slip	548	n/a	8.18
Cucamonga	18.96	thrust	28	5	6.70
Cleghorn	24.25	strike slip	25	3	6.80
North Frontal (West)	28.93	reverse	50	1	7.20
Chino, alt 2	35.55	strike slip	29	1	6.80
Elsinore	35.58	strike slip	241	n/a	7.85
Chino, alt 1	35.74	strike slip	24	1	6.70
San Jose	36.64	strike slip	20	0.5	6.70

Fault Name and Section	Closest Distance (km)	Slip Sense	Length (km)	Slip Rate (mm/year)	Maximum Magnitude
Sierra Madre	41.36	reverse	57	2	7.20
Sierra Madre Connected	41.36	reverse	76	2	7.30

(Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

7.2 Seismic Design Parameters

Seismic parameters based on the 2019 California Building Code (CBSC, 2019) and ASCE 7-16 are provided in the following table. These parameters were determined using the coordinate (33.0574N and 117.2997W) and the Seismic Design Maps ATC online tool.

Table No. 3, CBC Seismic Design Parameters

Seismic Parameters	
Site Coordinates	34.0574N, 117.2997W
Site Class	D
Risk Category	II
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	2.358g
Mapped 1-second Spectral Response Acceleration, S_1	0.944g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.000
Site Coefficient (from Table 1613.5.3(2)), F_v	1.700
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.358g
MCE 1-second period Spectral Response Acceleration, SM_1	1.604g
Design Spectral Response Acceleration for short period S_{DS}	1.572g
Design Spectral Response Acceleration for 1-second period, S_{D1}	1.069g
Site Modified Peak Ground Acceleration, PGA_M	1.092g

7.3 Secondary Effects of Seismic Activity

In general, secondary effects of seismic activity include surface fault rupture, soil liquefaction, landslides, lateral spreading, and settlement due to seismic shaking, tsunamis, seiches, and earthquake-induced flooding. The site-specific potential for each of these seismic hazards is discussed in the following sections.

Surface Fault Rupture: No portion of the project site is located within a currently designated State of California or San Bernadino County Earthquake Fault Zone (CGS, 2007 and San Bernadino County, 2022a). The potential for surface rupture resulting from the movement of nearby or distant faults is not known with certainty but is considered very low.

Liquefaction: Liquefaction is defined as the phenomenon in which a cohesionless soil mass within the upper 50 feet of the ground surface suffers a substantial reduction in its shear strength, due the improvement of excess pore pressures. During earthquakes, excess pore pressures in saturated soil deposits may develop as a result of induced cyclic shear stresses, resulting in liquefaction.

Soil liquefaction generally occurs in submerged granular soils and non-plastic silts during or after strong ground shaking. There are several general requirements for liquefaction to occur and they are as follows.

- Soils must be submerged.
- Soils must be loose to medium-dense.
- Ground motion must be intense.
- Duration of shaking must be sufficient for the soils to lose shear resistance.

Based on review of hazard maps, the site is located within a State of California and San Bernadino County designated zone of liquefaction susceptibility (CGS, 2007; San Bernadino County, (2022a). However, based on groundwater being deeper than approximately 101 feet bgs, we estimate that the liquefaction induced settlement of the site is negligible.

Seismic Settlement: Dynamic dry settlement may occur in loose, granular, unsaturated soils during a large seismic event. Based on dense soil conditions, site-specific boring logs, soil types, soil conditions and blow counts, dry seismic settlement is expected to be minimal.

Landslides: Seismically induced landslides and slope failures are common occurrences during or soon after large earthquakes. Due to the flat nature of the project site, the potential for seismically induced landslides affecting the project site is considered to be very low.

Lateral Spreading: Seismically induced lateral spreading involves primarily lateral movement of earth materials over underlying materials which are liquefied due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. Generally due to the negligible risk for liquefaction and flat nature of project site, the risk of lateral spreading is considered low.

Tsunamis: Tsunamis are large waves generated in open bodies of water by fault displacement or major ground movement. Due to the inland location of the project site, tsunamis are not considered to be a risk.

Seiches: Seiches are large waves generated in enclosed bodies of water in response to ground shaking. There are no enclosed bodies of water near the project site, therefore, seiching is not considered to be a risk during construction.

Earthquake-Induced Flooding: Dams or other water-retaining structures may fail as a result of large earthquakes. The project site is not located within a designated dam inundation area (DSOD, 2022).

8.0 LABORATORY TEST RESULTS

Laboratory testing was performed to determine the physical and chemical characteristics and engineering properties of the subsurface soils. Current physical test results are included in Appendix A, *Field Exploration* and Appendix B, *Laboratory Testing Program*. Discussions of the various test results are presented below.

8.1 Physical Testing

- *In-situ* Moisture and Dry Density: *In-situ* dry densities and moisture contents of the site soils were determined in accordance with ASTM Standard D2216 and D2937**.** Dry densities of the soils in the upper 10 feet of all borings ranged from 88.5 to 107.7 pounds per cubic foot (pcf) with moisture contents of 9 to 31 percent. Results are presented on the Log of Borings in Appendix A, *Field Exploration.*

- Expansion Index: One representative bulk soil sample was tested to evaluate the expansion potential in accordance with ASTM Standard D4829. The test result indicated an expansion index of 34, corresponding to low expansion potential.

- R-Value: One representative bulk sample was tested in accordance with Caltrans Test Method 301. The result of the R-value test was 45.

- Grain Size Analysis: Four representative samples were tested to determine the relative grain size distribution in accordance with the ASTM Standard D6913. Test results are graphically presented in Drawing No. B-1, *Grain Size Distribution Results.* Based on the test results, soils are typically sand, silty sand, and sandy silt.

- Maximum Dry Density and Optimum Moisture Content: Two typical moisture-density relationships of representative soil samples were tested, in accordance with ASTM Standard D1557, with the results presented in Drawing No. B-2, *Moisture-Density Relationship Results*, in Appendix B, *Laboratory Testing Program.* The laboratory maximum dry densities were 124.8 and 130.0 pounds per cubic feet (pcf), with optimum moisture contents of 10.5 and 9.2 percent, respectively.

- Direct Shear: Two direct shear tests were performed in accordance with ASTM Standard D3080. One test was performed on a relatively undisturbed sample and one test was performed on a sample remolded to 90 percent of the maximum dry density under soaked moisture condition in accordance with ASTM Standard D3080. The results of the direct shear tests are presented in Drawing Nos. B-3 through B-4, *Direct Shear Test Results* in Appendix B, *Laboratory Testing Program*.

- Consolidation Test – Two consolidation tests were performed on relatively undisturbed samples of the site soil, in accordance with ASTM Standard D2435. The test results are shown on Drawing No. B-5 and B-6, *Consolidation Test Results,* in Appendix B, *Laboratory Testing Program.*

8.2 Chemical Testing – Corrosivity Evaluation

One representative soil sample was tested during the current geotechnical investigation to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of these tests was to determine the corrosion potential of site soils when placed in contact with common pipe materials. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with California Test Methods 643, 422, and 417. The current and previous test results are presented in Appendix B, *Laboratory Testing Program and* are summarized in below.

- The pH measurement of the sample tested was 8.7.
- The sulfate content of the sample tested was 63 ppm (0.063 percent by weight).
- The chloride content of the sample tested was 61 ppm.
- The minimum electrical resistivity when saturated was 1,789 ohm-cm.

9.0 EARTHWORK AND SITE GRADING RECOMMENDATIONS

Earthwork for the project will include grading, trench excavation, pipe subgrade preparation, pipeline bedding placement and trench backfill, as well as roadway pavement construction. Recommendations for earthwork are presented in the following subsections. General Earthwork Specifications are presented in Appendix C*, Earthwork Specifications.*

9.1 General

This section contains our general recommendations regarding earthwork for the proposed commercial development of the Ashley Home Store Expansion in the City of Colton, San Bernardino County, California.

These recommendations are based on the results of our field exploration and laboratory testing as well as our experience with similar projects, and data evaluation as presented in the preceding sections. These recommendations may require modification by the geotechnical consultant based on observation of the actual field conditions during remedial grading.

Prior to the start of construction, all underground existing utilities and appurtenances should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications. All excavations should be conducted in such a manner as not to cause loss of bearing and/or lateral support of existing structures or utilities.

All existing structures, debris, deleterious material and surficial soils containing roots and perishable materials should be stripped and removed from the project site. Deleterious material, concrete, and debris generated during excavation, should not be placed as fill.

It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.

9.2 Private Sewage System Abandonment

From a geotechnical standpoint, any seepage pits, other private sewage systems, and/or other subsurface structures that may be encountered should be located, mapped on the grading plans, removed and/or properly abandoned. Abandonment and/or removal of septic systems that may exist should be in accordance with local codes and recommendations by Converse. Seepage pits, if abandoned in-place, should be pumped clean, backfilled with gravel or clean sand jetted into place, and then capped with a minimum of 2 feet of a 2-sack or greater slurry or concrete for a minimum distance of 2 feet outside the edge of the seepage pit. The top of the slurry or concrete cap should be at a minimum 10 feet below proposed grade.

9.3 Overexcavation

The site is generally underlain by approximately 2.0 to 7.0 feet of potentially compressible soils (the upper low-density portions of the alluvium), which may be prone to future settlement under the surcharge of foundation, improvements and/or fill loads. Therefore, these materials should be over-excavated to competent alluvium, within all areas of proposed structures and other improvements, and replaced with compacted fill soils.

Building Pad: Within the entire level portions of the building pad areas overexcavations should be at least 5.0 feet to 7.0 below existing grade, as well as 3.0 feet below the lowest proposed building footings, whichever is deeper. All over-excavations should extend laterally at least five feet or equal to the depth of over-excavation, whichever is greater, outside the entire portions of the building pad area.

***Improvements Outside of the Building Area**:* For areas of proposed parking, flatwork, walls and other improvements, overexcavations should be at least 2.0 to 3.0 feet below existing grade. Within wall areas overexcavations should also be a minimum of 2.0 feet below the proposed wall footings. All over-excavations should extend laterally at least 3.0 feet or equal to the depth of over-excavation, whichever is greater.

The final bottom surfaces of all excavations should be observed and approved by the project geotechnical consultant prior to placing any fill or structures. However, localized, deeper over-excavation could be encountered, based on findings in subsurface explorations and testing by the geotechnical consultant during grading of the final bottom surfaces of all excavations.

The estimated locations and approximate depths of over-excavation of unsuitable, compressible soil materials are indicated on Figure No. 2, *Approximate Boring and Overexcavation Locations Map*.

If isolated pockets of very soft, loose, eroded, or pumping soil are encountered, the unstable soil should be excavated as needed to expose undisturbed, firm, and unyielding soils.

The contractor should determine the best manner to conduct the excavations, such that there are no losses of bearing and/or lateral support to the existing structures or utilities (if any).

Areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

9.4 Cut and Shallow Fill Below Building Pad Areas

Fill building pads with shallow cut and fill areas should be capped with a minimum of 5.0 feet of engineered structural fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 3.0 feet of fill below footings for proposed structures and 2.0 feet below footings for proposed walls. Over-excavation should extend to the entire level portions of the building pad area with proposed structures or walls, to the depth of fill.

9.5 Cut/Fill Transition and Fill Differentials

To mitigate distress to structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all level portions of the building pad areas. This should be accomplished by overexcavating the entire “cut” portion of the building pad area by at least 5.0 feet below proposed grade and replacing the excavated materials as properly compacted fill, so that all footings for structures and walls are founded into engineered fill with a minimum of 3.0 feet of fill below footings for proposed structures and 2.0 feet below footings for proposed walls. Recommended depths of over-excavation are provided in the following table.

Table No. 4, Overexcavation Depth for Cut/Fill Transitions

Depth of Fill ("Fill" Portion)	Depth of Overexcavation ("Cut" Portion)
Up to 5.0 feet	5.0 feet
Greater than 5.0 feet	One-third the maximum thickness of fill placed on the "fill" portion (15 feet maximum)

9.6 Engineered Fill

No fill should be placed until excavations and/or natural ground preparation have been observed by the geotechnical consultant. The native soils encountered within the project sites are generally considered suitable for re-use as compacted fill. Excavated soils should be processed, including removal of roots and debris, removal of oversized particles, mixing, and moisture conditioning, before placing as compacted fill. On-sites soils used as fill should meet the following criteria.

- No particles larger than 8 inches in largest dimension.
- Rocks larger than 4 inches should not be placed within the upper 12 inches of subgrade soils.
- Free of all significant organic matter, debris, or other deleterious material.
- Expansion index of 50 or less.
- Sand Equivalent greater than 15 (greater than 30 for pipe bedding).
- Contain less than 30 percent by weight retained in 3/4-inch sieve.
- Contain less than 40 percent fines (passing #200 sieve).

Based on field investigation and laboratory testing results, on-sites soils may be suitable as fill materials.

Imported materials, if required, should meet the above criteria prior to being used as compacted fill. Any imported fills should be tested and approved by geotechnical representative at least 72 hours prior to delivery to the site.

9.7 Compacted Fill Placement

All surfaces to receive structural fills should be scarified to a depth of 12 inches. The soil should be moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. The scarified soils should be recompacted to at least 90 percent of the laboratory maximum dry density.

Fill soils should be thoroughly mixed, and moisture conditioned to within ±3 percent of optimum moisture content for coarse soils and 0 to 2 percent above optimum moisture content for fine soils. Fill soils should be evenly spread in horizontal lifts not exceeding 8 inches in uncompacted thickness.

Fill soils should be thoroughly mixed, and moisture conditioned to at least 0 to 2 percent above optimum moisture content. Fill soils should be evenly spread in horizontal lifts not exceeding 8 inches in uncompacted thickness.

All fill placed at the site should be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein. The upper 12 inches of subgrade soils underneath pavements intended to support vehicle loads should be scarified, moisture conditioned, and compacted to at least 95 percent of the laboratory maximum dry density.

Fill materials should not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations should not resume until the geotechnical consultant approves the moisture and density conditions of the previously placed fill.

Observations and field tests should be performed by the project soils consultant to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort should be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

Additional expansion index and corrosion testing should be completed after all fill has been placed and compacted at the site in order to confirm the soil properties and revise the foundation design parameters if necessary.

9.8 Backfill Recommendations Behind Walls

Compaction of backfill adjacent to retaining walls, which may be proposed, can produce excessive lateral pressures. Improper types and locations of compaction equipment and/or compaction techniques may damage the walls. The use of heavy compaction equipment should not be permitted within a horizontal distance of 5 feet from the wall. Backfill behind any structural walls within the recommended 5-foot zone should be compacted using lightweight construction equipment such as handheld compactors to avoid overstressing the walls.

9.9 Shrinkage and Subsidence

The volume of excavated and recompacted soils will decrease as a result of grading. The shrinkage would depend on, among other factors, the depth of cut and/or fill, and the grading method and equipment utilized. Based on our exploration as well as previous experience in the other projects in close vicinity of this site, for the preliminary estimation, shrinkage factors for various units of earth material at the site may be taken as presented below.

- The shrinkage factor (defined as a percentage of soil volume reduction when moisture conditioned and compacted to the average of 92 percent relative compaction) for the upper 17 feet of soils is estimated to range from approximately 8 to 25 percent. An average value of 12 percent to 16 percent may be used for preliminary earthwork planning.
- Subsidence (defined as the settlement of native materials from the equipment load applied during grading and proposed fill loads) would depend on the construction methods including type of equipment utilized. Ground subsidence is estimated to be approximately 0.20 foot to 0.25 foot.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

9.10 Site Drainage

Adequate positive drainage should be provided away from the structures and excavation areas to prevent ponding and to reduce percolation of water into the foundation soils. A desirable drainage gradient is 1 percent for paved areas and 2 percent in landscaped areas. Surface drainage should be directed to suitable non-erosive devices.

9.11 Utility Trench Backfill

The following sections present earthwork recommendations for utility trench backfill, including subgrade preparation and trench zone backfill.

Open cuts adjacent to existing roadways or structures are not recommended within a 1:1 (horizontal: vertical) plane extending down and away from the roadway or structure perimeter (if any).

Soils from the trench excavation should not be stockpiled more than 6 feet in height or within a horizontal distance from the trench edge equal to the depth of the trench. Soils should not be stockpiled behind the shoring, if any, within a horizontal distance equal to the depth of the trench, unless the shoring has been designed for such loads.

9.11.1 Pipeline Subgrade Preparation

The final subgrade surface should be level, firm, uniform, and free of loose materials and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. Protruding oversize particles larger than 2 inches in dimension, if any, should be removed from the trench bottom and replaced with compacted on-site materials.

Any loose, soft and/or unsuitable materials encountered at the pipe subgrade should be removed and replaced with an adequate bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

9.11.2 Pipe Bedding

Bedding is defined as the material supporting and surrounding the pipe to 1 foot above the pipe. Recommendations for pipe bedding are provided below.

To provide uniform and firm support for the pipe, compacted granular materials such as clean sand, gravel or ¾-inch crushed aggregate, or crushed rock may be used as pipe bedding material. Typically, soils with sand equivalent value of 30 or more are used as pipe bedding material. The pipe designer should determine if the soils are suitable as pipe bedding material.

The type and thickness of the granular bedding placed underneath and around the pipe, if any, should be selected by the pipe designer. The load on the rigid pipes and deflection of flexible pipes and, hence, the pipe design, depends on the type and the amount of bedding placed underneath and around the pipe.

Bedding materials should be vibrated in-place to achieve compaction. Care should be taken to densify the bedding material below the spring line of the pipe. Prior to placing the pipe bedding material, the pipe subgrade should be uniform and properly graded to provide uniform bearing and support to the entire section of the pipe placed on bedding material. During the digging of depressions for proper sealing of the pipe joints, the pipe should rest on a prepared bottom for as near its full length as is practicable.

Based on the design groundwater depth, migration of fines from the surrounding native and/or fill soils may not be considered in selecting the gradation of any imported bedding material.

9.11.3 Trench Zone Backfill

The trench zone is defined as the portion of the trench above the pipe bedding extending up to the final grade level of the trench surface. Excavated sites soil free of oversize particles and deleterious matter may be used to backfill the trench zone. Detailed trench backfill recommendations are provided below.

- Trench excavations to receive backfill should be free of trash, debris or other unsatisfactory materials at the time of backfill placement.
- Trench zone backfill should be compacted to at least 90 percent of the laboratory maximum dry density as per ASTM D1557 test method. At least the upper 1 foot of trench backfill underlying pavement should be compacted to at least 95 percent of the laboratory maximum dry density as per ASTM D1557 test method.
- Particles larger than 1 inch should not be placed within 12 inches of the pavement subgrade. No more than 30 percent of the backfill volume should be larger than ¾-inch in the largest dimension. Gravel should be well mixed with finer soil. Rocks larger than 3 inches in the largest dimension should not be placed as trench backfill.
- Trench backfill should be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers or mechanical tampers to achieve the density specified herein. The backfill materials should be brought to within ± 3 percent of optimum moisture content for coarse-grained soil, and between optimum and 2 percent above optimum for fine-grained soil, then placed in horizontal layers. The thickness of uncompacted layers should not exceed 8 inches. Each layer should be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.
- The contractor should select the equipment and processes to be used to achieve the specified density without damage to adjacent ground, structures, utilities and completed work.
- It should be the responsibility of the contractor to maintain safe working conditions during all phases of construction.
- The field density of the compacted soil should be measured by the ASTM D1556 (Sand Cone) or ASTM D6938 (Nuclear Gauge) or equivalent.
- Trench backfill should not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations should not resume until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are in compliance with project specifications.

10.0 DESIGN RECOMMENDATIONS

Design recommendations for the structures are provided in the following section.

10.1 General Evaluation

The various design recommendations provided in this section are based on the field exploration and laboratory testing as well as the assumption that in preparing the site, the earthwork recommendations provided in this report will be implemented.

10.2 Preliminary Shallow Foundation Design Parameters

The proposed one- and two-story concrete tilt up buildings and possible retaining walls may be supported on continuous or isolated spread footings founded completely within in competent compacted fill. The design of the shallow foundations should be based on the recommended parameters presented in the table below.

Table No. 5, Recommended Foundation Parameters

Parameter	1 or 2-Story Value
Minimum continuous width (interior and exterior)	15 inches
Minimum continuous or isolated footing depth of embedment below lowest adjacent grade (interior and exterior)	18 inches
Allowable net bearing capacity	2,500 psf

Isolated interior and exterior footings should be at least 24 inches wide. The footing dimensions and reinforcement should be based on structural design. The allowable bearing capacity can be increased by 500 pounds per square foot (psf) with each foot of additional embedment and 100 psf with each foot of additional width up to a maximum of 3,500 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent for short duration loadings, which will include loadings induced by wind or seismic forces.

10.3 Lateral Earth Pressures and Resistance to Lateral Loads

In the following subsections, the lateral earth pressures and resistance to lateral loads are estimated by using on-site native soils strength parameters obtained from laboratory testing.

10.3.1 Active Earth Pressures

The active earth pressure behind any buried walls or foundation depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall or foundation inclination, surcharges, and any hydrostatic pressures. The lateral earth pressures for the project site are presented in the following table.

Table No. 6, Active and At-Rest Earth Pressures

Loading Conditions	Lateral Earth Pressure[1] (psf)	Lateral Earth Pressure[2] (psf)
	Level backfill	2:1 backfill
Active earth conditions (wall is free to deflect at least 0.001 radian)	50	85
At-rest (wall is restrained)	70	125

These pressures assume no surcharge and no hydrostatic pressure. If water pressure is allowed to build up behind the structure, the active pressures should be reduced by 50 percent and added to a full hydrostatic pressure to compute the design pressures against the structure.

10.3.2 Passive Earth Pressure

Resistance to lateral loads can be assumed to be provided by a combination of friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.30 between formed concrete and soil may be used with the dead load forces. An allowable passive earth pressure of 230 psf per foot of depth may be used for the sides of footings poured against recompacted soils. A factor of safety of 1.5 was applied in calculating passive earth pressure. The maximum value of the passive earth pressure should be limited to 2,300 psf for compacted fill.

Vertical and lateral bearing values indicated above are for the total dead loads and frequently applied live loads. If normal code requirements are applied for design, the above vertical bearing and lateral resistance values may be increased by 33 percent for short duration loading, which will include the effect of wind or seismic forces.

Due to the low overburden stress of the soil at shallow depth, the upper 1 foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

10.4 Retaining Walls Drainage

The recommended lateral earth pressure values do not include lateral pressures due to hydrostatic forces. Therefore, wall backfill should be free draining and provisions should be made to collect and dispose of excess water that may accumulate behind earth retaining structures. Behind wall drainage may be provided by free-draining gravel surrounded by synthetic filter fabric or by prefabricated, synthetic drain panels or weep holes. In either case, drainage should be collected by perforated pipes and directed to a sump, storm drain, or other suitable location for disposal. We recommend drain rock should consist of durable stone having 100 percent passing the 1-inch sieve and less than 5 percent passing the No. 4 sieve. Synthetic filter fabric should have an equivalent opening size (EOS), U.S. Standard Sieve, of between 40 and 70, a minimum flow rate of

110 gallons per minute per square foot of fabric, and a minimum puncture strength of 110 pounds.

10.5 Slabs-on-Grade

Slabs-on-grade should be supported on properly compacted fill. Compacted fill used to support slabs-on-grade should be placed and compacted in accordance with Section 9.7 *Compacted Fill Placement*.

Structural design elements of slabs-on-grade, including but not limited to thickness, reinforcement, joint spacing of more heavily loaded slabs will be dependent upon the anticipated loading conditions and the modulus of subgrade reaction (200 kcf) of the supporting materials and should be designed by a structural engineer.

Unless a more stringent design is recommended by the architect or the structural engineer, concrete interior floor slabs for commercial structures should be 6 inches or more thick. Interior floor slabs should be reinforced with No. 3 bars spaced 18 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs so that the desired placement is properly placed per the design engineer.

Slabs should be designed and constructed as promulgated by the American Concrete Institute (ACI) and the Portland Cement Association (PCA). Care should be taken during concrete placement to avoid slab curling. Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

Subgrade for slabs-on-grade should be firm and uniform. All loose or disturbed soils including under-slab utility trench backfill should be recompacted.

Prior to placing concrete, the subgrade soils below all floor slabs should be pre-watered to achieve a moisture content that is equal to 110% of the optimum moisture content of the subgrade soils. The moisture content should penetrate to a minimum depth of 18 inches. This will also promote uniform curing of the concrete and minimize the development of shrinkage cracks.

If moisture-sensitive flooring or environments are planned, slabs-on-grade should be protected by 10-mil-thick polyethylene vapor barriers. Laps within the membrane should be sealed and overlapped 12 inches. The sub-grade surface should be free of all exposed rocks or other sharp objects prior to placement of the barrier. The barrier should be overlain and overlain by 2 inches of sand, to minimize punctures and to aid in the concrete curing. At discretion of the structure engineer, the sand layer may be eliminated. These recommendations must be confirmed (and/or modified) by the foundation engineer with our concurrence, based upon the performance expectations of the foundation. It is the responsibility of the contractor to ensure that the moisture/vapor barrier systems are placed in accordance with the project plans and specifications, and that the moisture/vapor retarder materials are free of tears and punctures prior to concrete placement. Additional moisture reduction and/or prevention measures may be needed, depending on the performance requirements of future interior floor coverings.

In hot weather, the contractor should take appropriate curing precautions after placement of concrete to minimize cracking or curling of the slabs. Temperatures throughout the day should be considered when planning a concrete pour. The potential for slab cracking may be lessened by the addition of fiber mesh to the concrete and/or control of the water/cement ratio.

Concrete should be cured by protecting it against loss of moisture and rapid temperature change for at least seven days after placement. Moist curing, waterproof paper, white polyethylene sheeting, white liquid membrane compound, or a combination thereof may be used after finishing operations have been completed. The edges of concrete slabs exposed after removal of forms should be immediately protected to provide continuous curing.

10.6 Expansion Potential

Based on the results of expansion testing the site has an expansion index of **34**.

For design purposes, the site should be designed for low expansion potential **(EI from 21 to 50)**, and the following mitigation could be used.

- Moisture variation within the site soils must be minimized during and after construction.
- Fill should be placed at two percent over optimum moisture content.
- Irrigation should be minimized, and the use of drought tolerant landscaping should be considered.
- Controlled pre-wetting of the soil before placement of the foundation.

The expansion indices of the final finish-grade soils may vary from the results obtained during our investigation. The expansion potential of the finish-grade soils should be tested at the completion of grading. During construction, the contractor should determine effective methods to minimize moisture variations.

10.7 Settlement

The total settlement of shallow footings, designed as recommended above, from static structural loads and short-term settlement of properly compacted fill is anticipated to be 1.0 inch or less. The static differential settlement can be taken as equal to one-half of the static total settlement over a lateral distance of about 40 feet.

10.8 Pipe Design for Underground Utilities

Structural design of pipes requires proper evaluation of all possible loads acting on pipes. The stresses and strains induced on buried pipes depend on many factors, including the type of soil, density, bearing pressure, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and native soils. The recommended values of the various soil parameters for the pipe design are provided in Table No. 6, *Soil Parameters for Pipe Design*.

Where pipes are connecting to rigid structures near, or at its lower levels, and then are subjected to significant loads as the backfill is placed to finish grade, we recommend that provisions be incorporated in the design to provide support of these pipes where they exit the structure. Consideration can be given to flexible connections, concrete slurry support beneath the pipes where they exit the structures, overlaying and supporting the pipes with a few inches of compressible material, (i.e., Styrofoam, or other materials), or other techniques. Automatic shutoffs should be installed to limit the potential leakage from seismic event related damage.

Table No. 7, Soil Parameters for Pipe Design

Soil Parameters	Parameters
Total unit weight of compacted backfill (assuming 92% average relative compaction), γ	127 pcf
Angle of internal friction of soils, ϕ	28º
Soil cohesion, c	210 psf
Coefficient of friction between concrete and native soils, fs	0.30
Coefficient of friction between pipe and native soils, fs	0.25 for metal or HDPE pipe 0.30 for CML&C pipe
Bearing pressure against Compacted Fill or Natural Soils	2,500 psf
Coefficient of passive earth pressure, Kp	1.54
Coefficient of active earth pressure, Ka	0.65
Modulus of Soil Reaction, E’	1,500 psi

10.9 Soil Corrosivity

The results of current chemical testing by Converse of a representative samples of site soils were evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel.

The sulfate content of the sampled soils corresponds to American Concrete Institute (ACI) exposure category S0 for these sulfate concentrations (ACI 318-14, Table 19.3.1.1). No concrete type restrictions are specified for exposure category S0 (ACI 318-14, Table 19.3.2.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures such as footings, slab, and concrete pad will be exposed to moisture from precipitation and irrigation. Based on the site location and the results of chloride testing of the sites soils, we do not anticipate that concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure category C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

According to Romanoff, 1957, the following table provides general guideline of soil corrosion based on electrical resistivity.

Table No. 8, Correlation Between Resistivity and Corrosion

Soil Resistivity (ohm-cm) per Caltrans CT 643	Corrosivity Category
Over 10,000	Mildly corrosive
2,000 – 10,000	Moderately corrosive
1,000 – 2,000	corrosive
Less than 1,000	Severe corrosive

The measured value of the minimum electrical resistivity of the samples by Converse when saturated was 1,789 ohm-cm for the site. This indicates that the soils tested are corrosive to ferrous metals in contact with the soil.

Converse does not practice in the area of corrosion consulting. A qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils.

10.10 Asphalt Concrete Pavement

One soil sample was tested by Converse to determine the R-value of the subgrade soils. Based on laboratory testing, the R-value was 45. For pavement design, we have utilized a design R-value of 45, and Traffic Indices (TIs) ranging from 5.0 to 6.0.

Based on the above information, asphalt concrete and aggregate base thickness results are presented using the Caltrans Highway Design Manual (Caltrans, 2020), Chapter 630 with a safety factor of 0.2 for asphalt concrete/aggregate base section and 0.1 for full depth asphalt concrete section. Preliminary asphalt concrete pavement sections are presented in the following table below. City of Ontario minimum asphalt pavement and aggregate base thickness requirements were also considered in the pavement designs.

Table No. 9, Recommended Preliminary Pavement Sections

R-value	Area	Traffic Index (TI)	Pavement Section	
			Asphalt Concrete (inches)	Aggregate Base (inches)
45	Parking Lot and Drive Areas	5	3.0	4.0
	Entrance Apron and Heavy Traffic Areas	6	4.0	5.0

At or near the completion of grading, subsurface samples should be tested to evaluate the actual subgrade R-value for final pavement design.

Prior to placement of aggregate base, at least the upper 12 inches of finish grade should be scarified, moisture-conditioned if necessary, and recompacted to at least 95 percent of the laboratory maximum dry density as defined by ASTM Standard D1557 test method.

Base materials should conform with Section 200-2.2,"*Crushed Aggregate Base*," of the current Standard Specifications for Public Works Construction (SSPWC; Public Works Standards, 2018) and should be placed in accordance with Section 301.2 of the SSPWC.

Asphaltic concrete materials should conform to Section 203 of the SSPWC and should be placed in accordance with Section 302-5 of the SSPWC.

10.11 Concrete Flatwork

Except as modified herein, concrete walks, driveways, access ramps, curb and gutters should be constructed in accordance with Section 303-5, *Concrete Curbs, Walks, Gutters, Cross-Gutters, Alley Intersections, Access Ramps, and Driveways*, of the Standard Specifications for Public Works Construction (Public Works Standards, 2018).

The subgrade soils under the above-mentioned improvements should consist of compacted fill placed as described in section 9.7 of this report. Prior to placement of concrete, the upper 12 inches of finish grade should be moisture conditioned to within 3 percent of optimum moisture content for coarse-grained soils and 0 and 2 percent above optimum for fine-grained soils.

The thickness of driveways for passenger vehicles should be at least 4 inches, or as required by the civil or structural engineer. Transverse control joints for driveways should be spaced not more than 10 feet apart. Driveways wider than 12 feet should be provided with a longitudinal control joint.

Concrete walks subjected to pedestrian and bicycle loading should be at least 4 inches thick, or as required by the civil or structural engineer. Transverse joints should be spaced 15 feet or less and should be cut to a depth of one-fourth the slab thickness.

Positive drainage should be provided away from all driveways and sidewalks to prevent seepage of surface and/or subsurface water into the concrete base and/or subgrade.

11.0 CONSTRUCTION RECOMMENDATIONS

Temporary sloped excavation recommendations are presented in the following sections.

11.1 General

Prior to the start of construction, all existing underground utilities (if any) should be located at the project site. Such utilities should either be protected in-place or removed and replaced during construction as required by the project specifications.

Vertical braced excavations can be considered for the foundations. Sloped excavations may not be feasible in locations adjacent to existing utilities, pavement, or structure (if any). Recommendations pertaining to temporary excavations are presented in this section.

Excavations near existing structures may require vertical side wall excavation. Where the side of the excavation is a vertical cut, it should be adequately supported by temporary shoring to protect workers and any adjacent structures.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the geotechnical consultant and the competent person designated by the contractor. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

11.2 Temporary Sloped Excavations

Temporary open-cut trenches may be constructed with side slopes as recommended in the following table. Temporary cuts encountering soft and wet fine-grained soils; dry loose, cohesionless soils or loose fill from trench backfill may have to be constructed at a flatter gradient than presented below.

Table No. 10, Slope Ratios for Temporary Excavations

Soil Type	OSHA Soil Type	Depth of Cut (feet)	Recommended Maximum Slope (Horizontal:Vertical)[1]
Silty Sand/Sandy Silt (SM/ML)	C	0-10	1.5:1

[1] Slope ratio assumed to be uniform from top to toe of slope.

For shallow excavations up to 4 feet bgs, slopes can be vertical. For steeper temporary construction slopes or deeper excavations, or unstable soil encountered during the excavation, shoring or trench shields should be provided by the contractor to protect the workers in the excavation. Design recommendations for temporary shoring are provided in the following section.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction materials, should not be placed within 5 feet of the unsupported slope edge. Stockpiled soils with a height higher than 6 feet will require greater distance from trench edges.

12.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

The project geotechnical consultant should review plans and specifications as the project design progresses. Such review is necessary to identify design elements, assumptions, or new conditions which require revisions or additions to our geotechnical recommendations.

The project geotechnical consultant should be present to observe conditions during construction. Geotechnical observation and testing should be performed as needed to verify compliance with project specifications. Additional geotechnical recommendations may be required based on subsurface conditions encountered during construction.

13.0 CLOSURE

This report is prepared for the project described herein and is intended for use solely by HMC Construction, Inc. and their authorized agents, to assist in the development of the proposed project. Our findings and recommendations were obtained in accordance with generally accepted professional principles practiced in geotechnical engineering. We make no other warranty, either expressed or implied.

Converse Consultants is not responsible or liable for any claims or damages associated with interpretation of available information provided to others. Site exploration identifies actual soil conditions only at those points where samples are taken, when they are taken. Data derived through sampling and laboratory testing is extrapolated by Converse employees who render an opinion about the overall soil conditions. Actual conditions in areas not sampled may differ. In the event that changes to the project occur, or additional, relevant information about the project is brought to our attention, the recommendations contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing. In addition, the recommendations can only be finalized by observing actual subsurface conditions revealed during construction. Converse cannot be held responsible for misinterpretation or changes to our recommendations made by others during construction.

As the project evolves, a continued consultation and construction monitoring by a qualified geotechnical consultant should be considered an extension of geotechnical investigation services performed to date. The geotechnical consultant should review plans and specifications to verify that the recommendations presented herein have been appropriately interpreted, and that the design assumptions used in this report are valid. Where significant design changes occur, Converse may be required to augment or modify the recommendations presented herein. Subsurface conditions may differ in some locations from those encountered in the explorations, and may require additional analyses and, possibly, modified recommendations.

Design recommendations given in this report are based on the assumption that the recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

14.0 REFERENCES

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SAN BERNARDINO COUNTY, 2010b, San Bernardino County General Plan Geologic Hazard Overlays, Map Sheet FH30C, scale 1:14,400, dated March 9, 2010.

U.S. GEOLOGICAL SURVEY (USGS), 2008, 2008 National Seismic Hazard Maps (https://earthquake.usgs.gov/cfusion/hazfaults_2008_search, accessed April 2022.

U.S. GEOLOGICAL SURVEY (USGS), 2022, National Water Information System: Web Interface (http://nwis.waterdata.usga.gov/nwis/gwlevels), accessed April 2022.

APPENDIX A

FIELD EXPLORATION

Our field investigation included a site reconnaissance and a subsurface exploration program consisting of drilling exploratory borings. During the site reconnaissance, the surface conditions were noted, and the boring locations were marked in the field using the referenced site plans and from approximate distances from local streets as a guide and should be considered accurate only to the degree implied by the method used to locate them.

Three exploratory borings (BH-01 through BH-03) were drilled on February 18, 2022, within the project site limits to investigate the subsurface conditions. The borings were drilled to depths ranging from 16.5 to 51.5 feet (bgs).

The borings were advanced using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers for soils sampling. Encountered materials were continuously logged by a Converse geologist and classified in the field by visual classification in accordance with the Unified Soil Classification System. Where appropriate, the field descriptions and classifications have been modified to reflect laboratory test results.

Relatively undisturbed samples were obtained using California Modified Samplers (2.4 inches inside diameter and 3.0 inches outside diameter) lined with thin sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches. Blow counts at each sample interval are presented on the boring logs. Samples were retained in brass rings (2.4 inches inside diameter and 1.0 inch in height) and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Representative bulk samples were collected from selected depths and placed in large plastic bags for delivery to our laboratory.

Standard Penetration Testing (SPT) were also performed in boring BH-02 in accordance with the ASTM Standard D1586 using a standard (1.4 inches inside diameter and 2.0 inches outside diameter) split-barrel sampler. The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every 6 inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings.

Following completion of logging and sampling, borings BH-01 through BH-03 were backfilled with excavated soil cuttings and compacted by pushing down with the auger using the weight of the drill rig. If construction is delayed the ground surface at the boring locations may settle over time. We recommend the owner monitor the boring locations and backfill any depressions that occur or provide protection around the boring locations to prevent trip and fall injuries from occurring.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)

BORING LOG SYMBOLS

SAMPLE TYPE

- STANDARD PENETRATION TEST — Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE 2.42" I.D. sampler (CMS).
- DRIVE SAMPLE No recovery
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

DRILLING METHOD SYMBOLS

Auger Drilling | Mud Rotary Drilling | Dynamic Cone or Hand Driven | Diamond Core

UNITED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No. 21-81-269-01 | Drawing No. A-1a

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	SPT Blow Counts	Pocket Penetrometer (tsf)	CA Sampler	Torvane (tsf)	Field Approximation
Very Soft	<0.25	< 2	<0.25	<3	<0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	2 - 4	0.25 - 0.50	3 - 6	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	5 - 8	0.50 - 1.0	7 - 12	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	9 - 15	1.0 - 2.0	13 - 25	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	16 - 30	2.0 - 4.0	26 - 50	1.0 - 2.0	Readily indented by thumbnail
Hard	>4.0	>30	>4.0	>50	>2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60}- Value (blows / foot)	CA Sampler
Very Loose	<4	<5
Loose	4- 10	5 - 12
Medium Dense	11 - 30	13 - 35
Dense	31 - 50	36 - 60
Very Dense	>50	>60

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OF PROPORTION OF SOILS

Descriptor	Criteria
Trace (fine)/ Scattered (coarse)	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION/ Induration

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

UNITED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No. 21-81-269-01
Drawing No. A-1b

Log of Boring No. BH-01

Dates Drilled: 2/18/2022 Logged by: Catherine Nelson Checked By: Robert Gregorek II

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 965 Depth to Water (ft, bgs): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

3" ASPHALT/9" AGGREGATE BASE

ALLUVIUM

SILTY SAND/SANDY SILT (SM/ML): fine-grained, trace gravel up to 0.5 inches maximum dimension, trace clay, oxidation staining, slightly desiccated, medium dense/stiff, moist, light grayish brown to orangish brown.

SANDY CLAY (CL): fine-grained sand, stiff, moist, grayish brown to orangish brown.

CLAYEY SAND (SC): fine-grained, slightly desiccated, medium dense, moist, grayish brown.

Depth (ft)	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
2.5	X	X	8/8/9	24	97	R
5	X	X	4/6/7	31	89	
7.5	X	X	6/10/13	13	108	CP
10	X		5/8/11	9	100	
15	X		6/9/10	10	98	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Backfilled with soil cuttings/cement, compacted by pushing down with an auger using the weight of the drill rig, and surface patched with cold asphalt concrete on 02/18/2022.

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No. 21-81-269-01 Drawing No. A-2

Log of Boring No. BH-02

Dates Drilled: 2/18/2022 Logged by: Catherine Nelson Checked By: Robert Gregorek II

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 965 Depth to Water (ft, bgs): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		3" ASPHALT/12" AGGREGATE BASE						
		ALLUVIUM **SANDY SILT (ML):** fine-grained sand, few to little gravel up to 0.5 inches maximum dimension, trace clay, roots and rootlets, stiff, moist, orangish brown to brown.			6/6/8	16	107	CP, CR, DS, EI, PA
5					3/6/8	19	101	
					4/7/11	15	101	DS
10					5/7/9	17	103	
15					7/9/13	8	104	
								PA
20		- @20.0': interbedded layers of fine to coarse -grained sand .			4/8/6			
25		**SAND (SP):** fine to coarse-grained, little gravel up to 1 inches maximum dimension, trace silt, slightly to moderately desiccated, moist, grayish brown.			14/33/45	3	115	PA
30					7/12/17			

Log of Boring No. BH-02

Dates Drilled: 2/18/2022 Logged by: Catherine Nelson Checked By: Robert Gregorek II

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 965 Depth to Water (ft, bgs): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
	SILTY SAND/SANDY SILT (SM/ML): fine-grained, trace gravel up to 0.5 inches maximum dimension, trace clay, medium dense/very stiff to stiff, moist, light grayish brown.			12/13/15	15	102	PA
40	**SAND (SP):** fine to coarse-grained, trace silt, few gravel up to 0.5 inches maximum dimension, slightly desiccated, dense, moist, brownish gray.			4/4/34			
	CLAY (CL): medium stiff, moist, orangish brown.						
	SAND (SW): fine to coarse-grained, few gravel up to 0.5 inches mximum dimension, pinhole porosity, severely desiccated, oxidation staining, dense, moist, reddish brown to brown.						
45	**SAND (SP):** fine-grained, very dense, moist, light brownish gray.			23/50-6"	3	105	
50				24/39/48			

End of boring at 51.5 feet bgs.
No groundwater encountered.
Backfilled with soil cuttings/cement, compacted by pushing down with an auger using the weight of the drill rig, and surface patched with cold asphalt concrete on 02/18/2022.

Log of Boring No. BH-03

Dates Drilled: 2/18/2022 Logged by: Catherine Nelson Checked By: Robert Gregorek II

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 964 Depth to Water (ft, bgs): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the Boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
0	**9" CONCRETE/ NO AGGREGATE BASE**						
	ALLUVIUM **SILTY SAND/SANDY SILT (SM/ML):** fine-grained, trace gravel up to 0.5 inches maximum dimension, trace clay, slightly desiccated, oxidation staining, medium dense/stiff, moist, light grayish brown.	X	X	5/5/11	23	98	
5	- @5.0': moderately desiccated, less oxidation staining, medium dense/medium stiff, light brownish gray	X		4/5/7	13	100	C
	- @7.5': medium dense/stiff.	X		6/9/12	9	107	
10		X	X	5/7/11	16	107	C
15	- @15.0': medium dense/very stiff.	X		9/15/20	9	102	

End of boring at 16.5 feet bgs.
No groundwater encountered.
Backfilled with soil cuttings/cement, compacted by pushing down with an auger using the weight of the drill rig, and surface patched with concrete on 02/18/2022.

APPENDIX B

LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their physical properties and engineering characteristics. The amount and selection of tests were based on the geotechnical parameters required for this project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the various laboratory tests conducted for this project.

In-Situ Moisture Content and Dry Density

In-situ dry density and moisture content tests were performed on relatively undisturbed ring samples, in accordance with ASTM Standard D2216 and D2937 to aid in soils classification and to provide qualitative information on strength and compressibility characteristics of the site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Expansion Index

One representative bulk sample was tested to evaluate the expansion potential of materials encountered at the site in accordance with ASTM D4829 Standard. The test result is presented in the following table.

Table No. B-1, Expansion Index Test Result

Boring No.	Depth feet)	Soil Description	Expansion Index	Expansion Potential
BH-02	1.3-5.0	Sandy Silt (ML)	34	Low

R-value

One representative bulk soil sample was tested for resistance value (R-value) in accordance with California Test Method CT301. This test provides a relative measure of soil strength for use in pavement design. The test result is presented in the table below.

Table No. B-2, R-Value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-01	1.0-5.0	Silty Sand Sandy Silt (SM/ML)	45

Soil Corrosivity

One representative soil sample was tested during to determine minimum electrical resistivity, pH, and chemical content, including soluble sulfate and chloride concentrations. The purpose of this test was to determine the corrosion potential of site soils when placed in contact with common construction materials. The tests were performed by AP Engineering and Testing, Inc. (Pomona, CA) in accordance with Caltrans Test Methods 643, 422 and 417. Test results are presented in the following table.

Table No. B-3, Summary of Soil Corrosivity Test Results

Boring No.	Depth (feet)	pH	Soluble Sulfates (CA 417) (ppm)	Soluble Chlorides (CA 422) (ppm)	Min. Resistivity (CA 643) (Ohm-cm)
BH-02	1.3-5.0	8.7	63	61	1,789

<u>Grain-Size Analysis</u>

To assist in soil classification, mechanical grain-size analyses was performed on four select samples in accordance with the ASTM Standard D6913 test method. Grain-size distribution is summarized in the table below and plotted in Drawing No. B-1, *Grain Size Distribution Results*.

Table No. B-4, Grain Size Distribution Test Results

Boring No.	Depth (ft)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-02	1.3-5.0	Sandy Silt (ML)	11.0	33.8	55.2	
BH-02	15.0-20.0	Sandy Silt (ML)	17.0	28.3	54.7	
BH-02	25.0-26.5	Sand (SP)	24.0	67.5	8.5	
BH-02	35.0-36.5	Silty Sand/Sandy Silt (SM/ML)	4.0	44.3	51.7	

<u>Maximum Dry Density and Optimum Moisture Content</u>

Laboratory maximum dry density-optimum moisture content relationship tests were performed on two representative bulk samples. These tests were conducted in accordance with the ASTM Standard D1557 test method. The test results are presented in Drawing No. B-2, *Moisture-Density Relationship Results,* and is summarized in the following table.

Table No B-5, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb./cft)
BH-01	5.0-10.0	Clayey Sand/Sandy Clay (SC/CL), Grayish Brown	10.5	124.8
BH-02	1.3-5.0	Sandy Silt (ML), Orangish Brown	9.2	130.0

Direct Shear

Two direct shear tests were performed; one direct shear test was performed on a relatively undisturbed sample and one direct shear test was performed on a sample remolded to 90% of the maximum dry density under soaked moisture conditions in accordance with ASTM D3080. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.02 inch/minute. Shear deformation was recorded until a maximum of about 0.25-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawings No. B-3 and B-4, *Direct Shear Test Results,* and the following table.

Table No. B-6, Summary of Direct Shear Test Results

Boring No.	Depth (feet)	Soil Description	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
*BH-02	1.3-5.0	Sandy Silt (ML)	28	270
BH-02	7.5-9.0	Sandy Silt (ML)	29	150

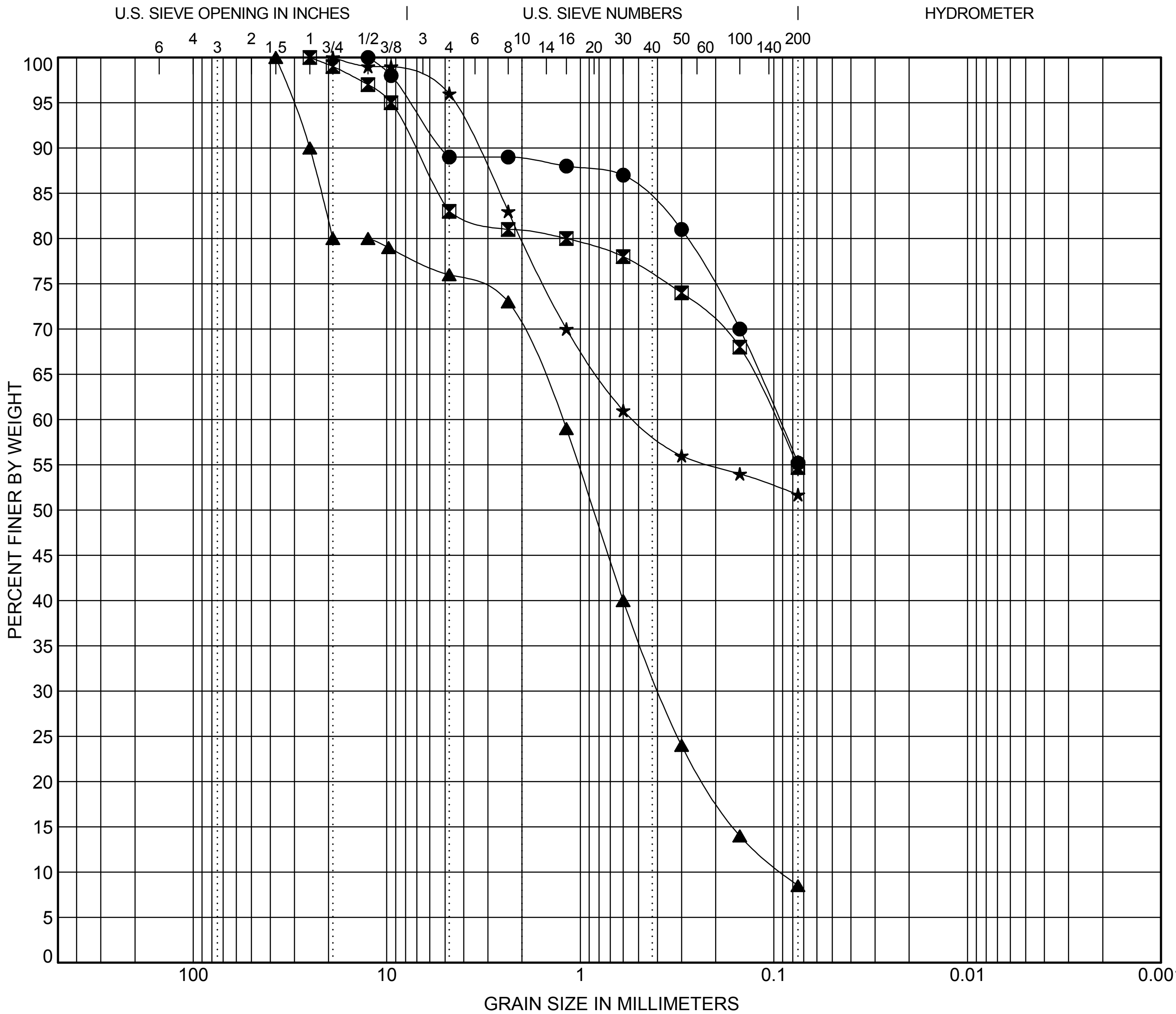
*Remolded to 90% of the maximum dry density

Consolidation

Two consolidation tests were conducted in accordance with ASTM Standard D2435 method. Data obtained from the test performed on relatively undisturbed ring samples were used to evaluate the settlement characteristics of the on-site soils under load. Preparation for these tests involved trimming the sample, placing it in a 1-inch-high brass ring, and loading it into the test apparatus, which contained porous stones to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state of equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. For test results, including sample density and initial moisture content, see Drawings No. B-5 and B-6, *Consolidation Test Results*.

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period.



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>coarse</td><td>fine</td><td>coarse</td><td>medium</td><td>fine</td></tr>
</table>

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-02**	**1.3-5**	**SANDY SILT (ML)**					
⊠	**BH-02**	**15-20**	**SANDY SILT (ML)**					
▲	**BH-02**	**25-26.5**	**SAND (SP)**				**1.35**	**13.68**
★	**BH-02**	**35-36.5**	**SILTY SAND/SANDY SILT (SM/ML)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-02**	**1.3-5**	**12.5**	**0.094**			**11.0**	**33.8**	**55.2**	
⊠	**BH-02**	**15-20**	**25**	**0.099**			**17.0**	**28.3**	**54.7**	
▲	**BH-02**	**25-26.5**	**37.5**	**1.24**	**0.389**	**0.091**	**24.0**	**67.5**	**8.5**	
★	**BH-02**	**35-36.5**	**19**	**0.522**			**4.0**	**44.3**	**51.7**	

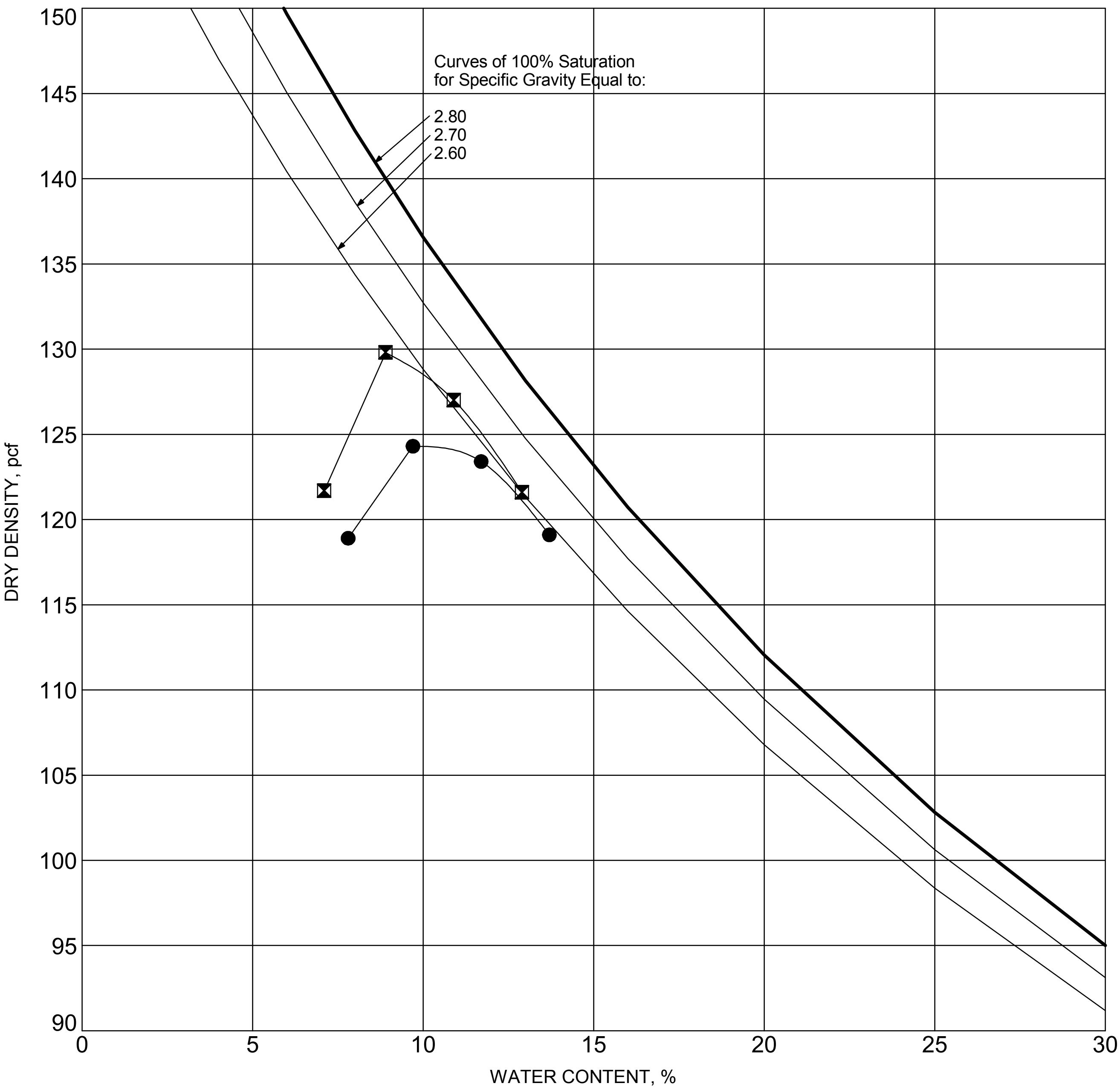
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No. **21-81-269-01**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	**BH-01**	**5-10**	**CLAYEY SAND/SANDY CLAY (SC/CL), Grayish Brown**	**D1557 - A**	**10.5**	**124.8**
⧗	**BH-02**	**1.3-5**	**SANDY SILT (ML), Orangish Brown**	**D1557 - A**	**9.2**	**130.0**

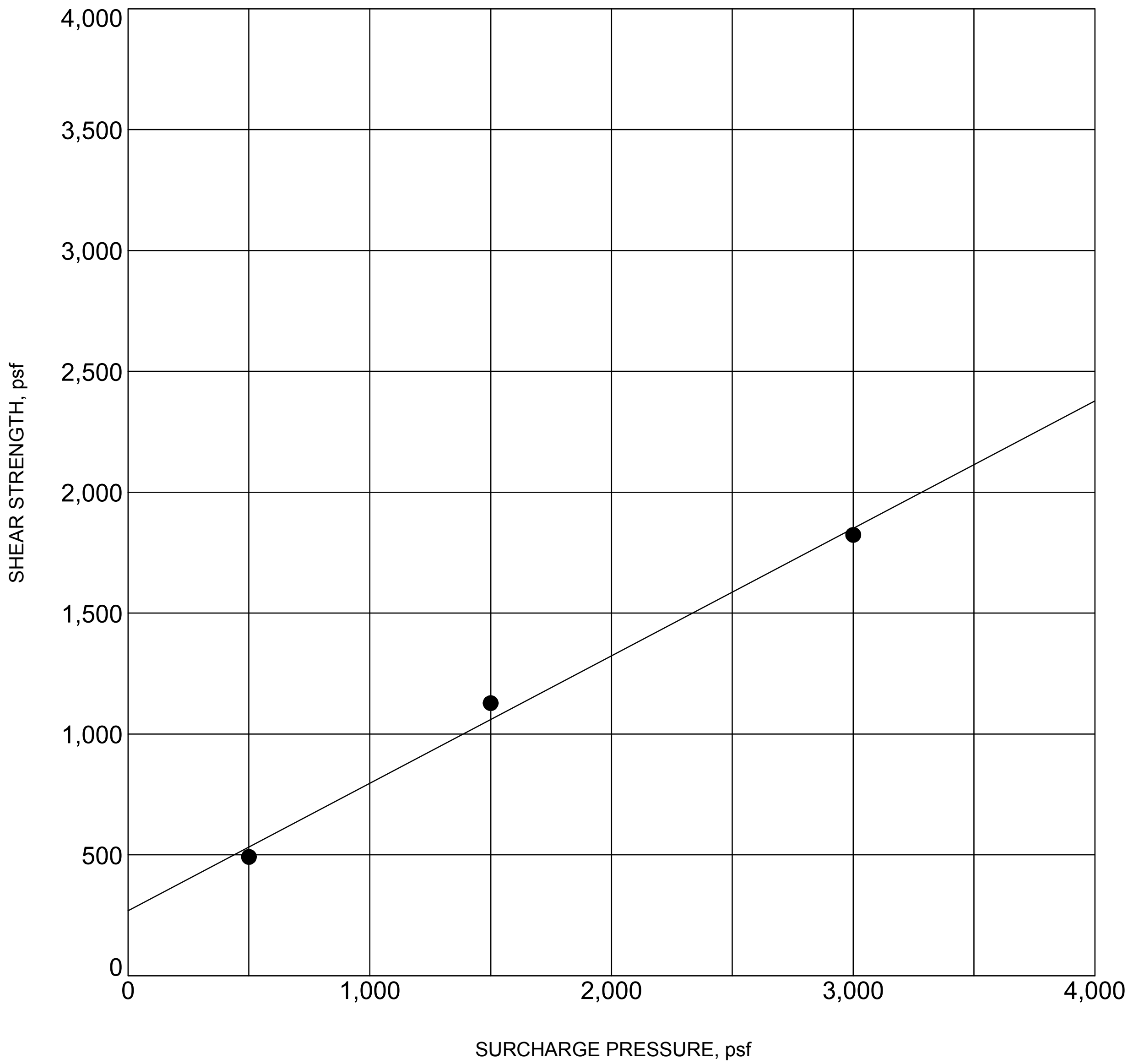
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No.
21-81-269-01

Drawing No.
B-2



BORING NO.	:	***BH-02**	DEPTH (ft)	:	**1.3-5**
DESCRIPTION	:	**SANDY SILT (ML)**			
COHESION (psf)	:	**270**	FRICTION ANGLE (degrees):		**28**
MOISTURE CONTENT (%)	:	**9.0**	DRY DENSITY (pcf)	:	**116.9**

(Remolded to 90% laboratory maximum dry density)

NOTE: Ultimate Strength.

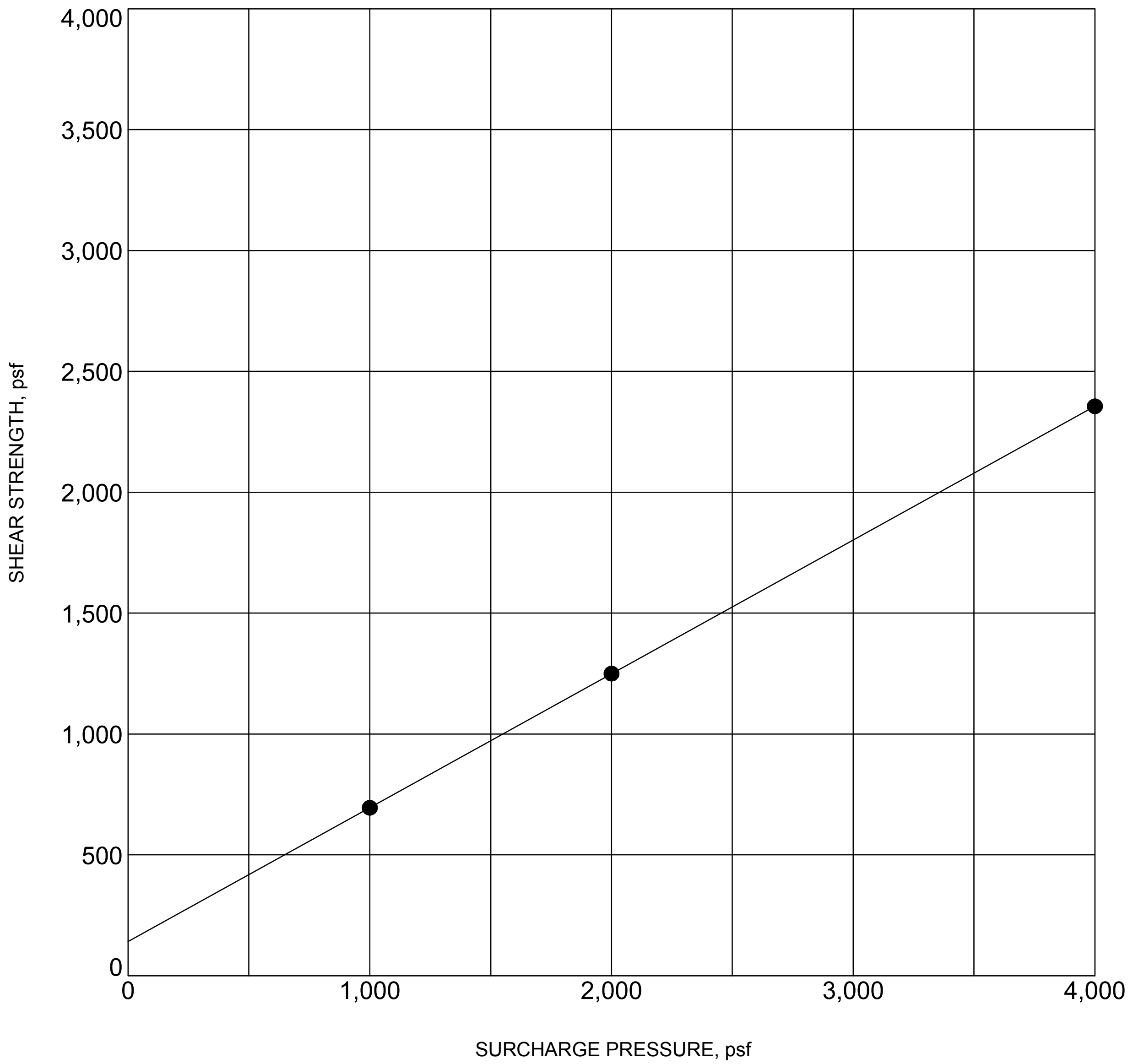
DIRECT SHEAR TEST RESULTS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No. **21-81-269-01**

Drawing No. **B-3**



BORING NO.	:	**BH-02**	DEPTH (ft)	:	**2.5-4.0**
DESCRIPTION	:	**SANDY SILT (ML)**			
COHESION (psf)	:	**140**	FRICTION ANGLE (degrees):		**29**
MOSITURE CONTENT (%)	:	**5.0**	DRY DENSITY (pcf)	:	**107.4**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

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855 Ashley Way
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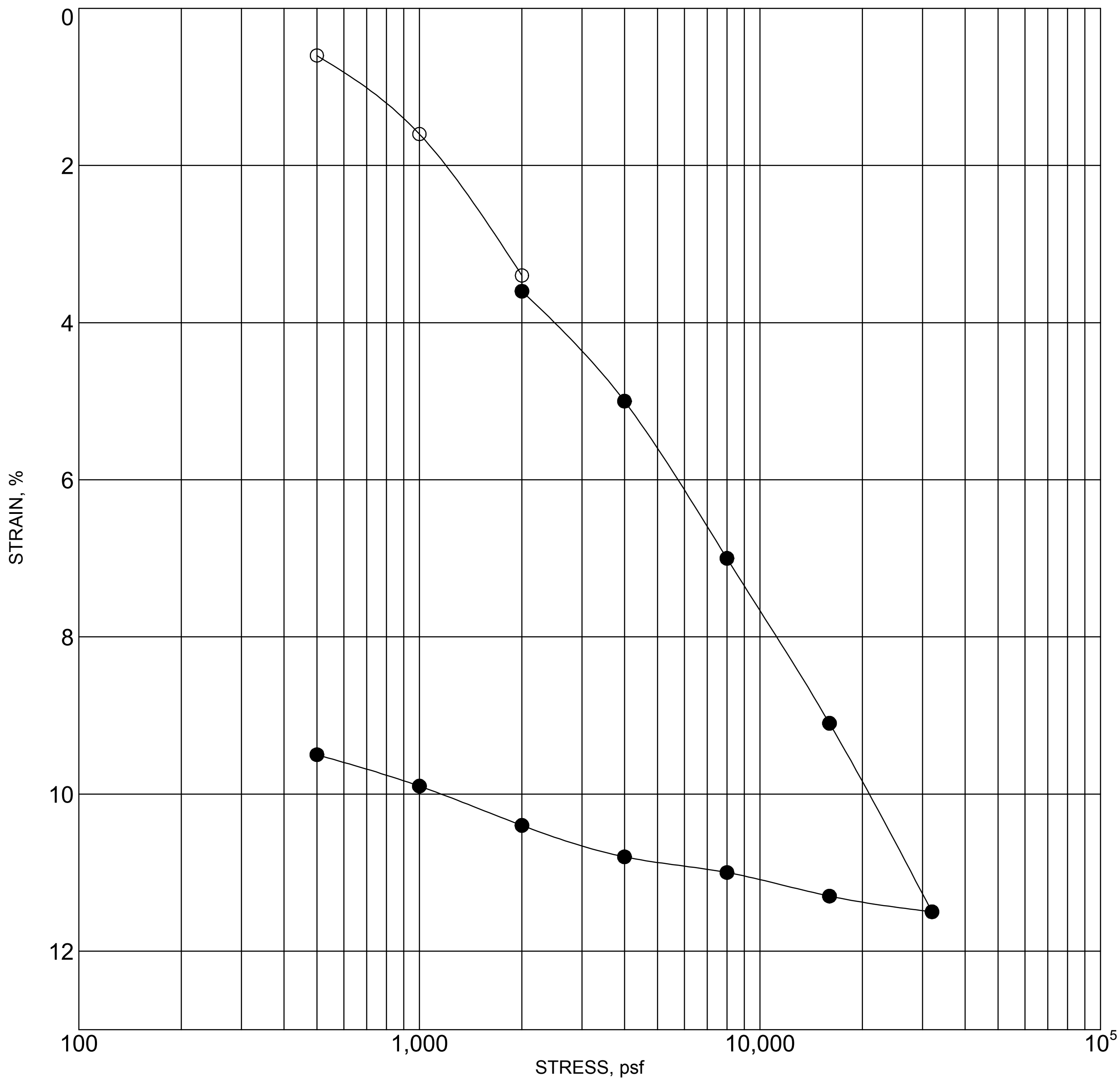
Project No. **21-81-269-01**

Drawing No. **B-4**

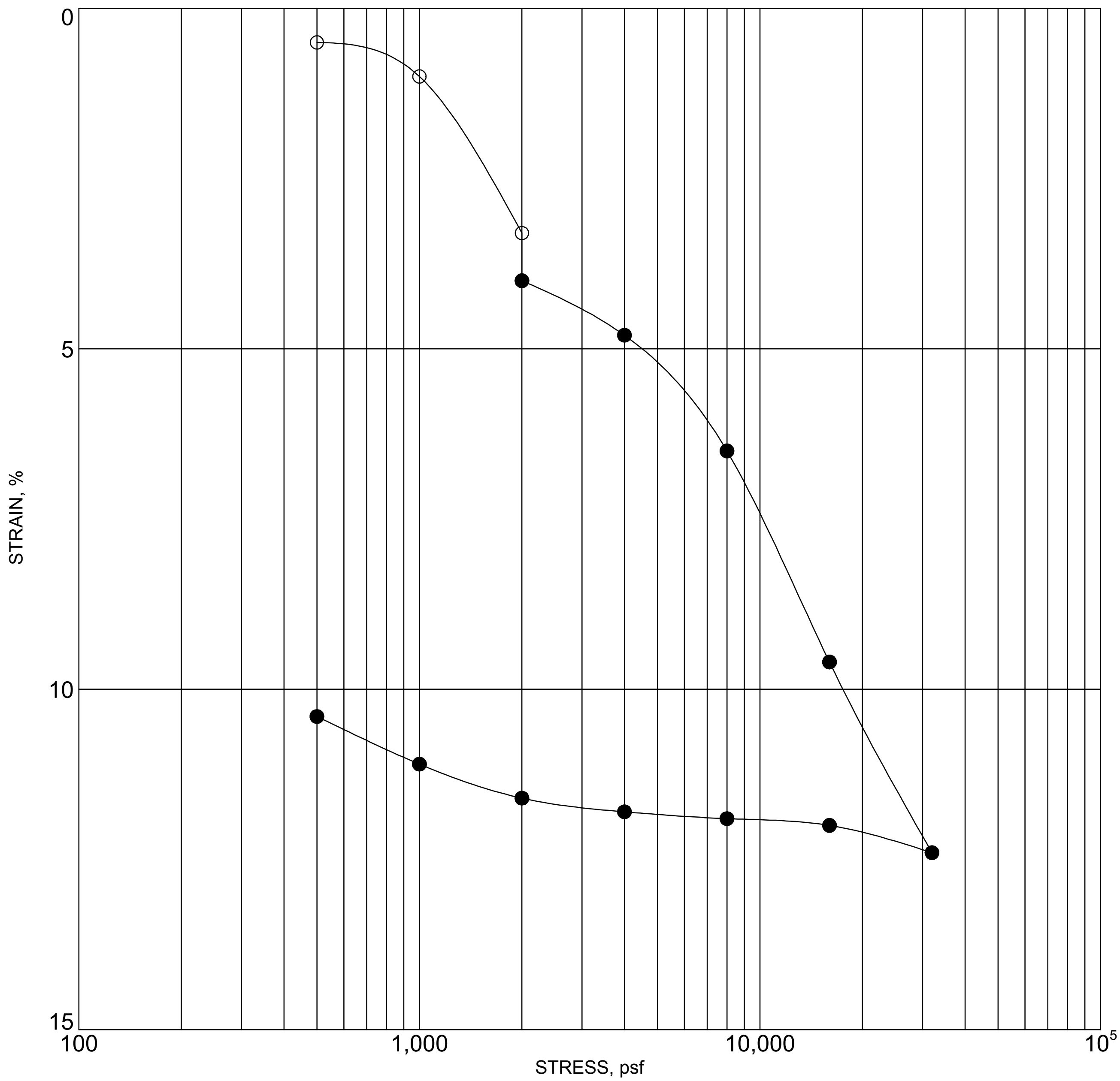


BORING NO. :	BH-03	DEPTH (ft) :	5.0-6.5
DESCRIPTION :	SILTY SAND/SANDY SILT (SM/ML)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	13	100	53	0.497
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS



BORING NO. :	BH-03	DEPTH (ft) :	10.0-11.5
DESCRIPTION :	SILTY SAND /SANDY SILT (SM/ML)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	16	107	78	0.538

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Ashley Home Store Expansion
855 Ashley Way
City of Colton, San Bernardino County, California
For: HMC Construction, Inc.

Project No.	Drawing No.
21-81-269-01	**B-6**

APPENDIX C

EARTHWORK SPECIFICATIONS

APPENDIX C

EARTHWORK SPECIFICATIONS

C1.1 Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following.

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fill
- Observation and Testing

C1.2 Site Inspection

1. The Contractor should carefully examine the site and make all inspections necessary in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor should satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor should satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.

2. This *Geotechnical Investigation Report* by Converse Consultants may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information should not relieve the Contractor from performing such independent investigation and evaluation to

satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

C1.3 Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.
2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.
3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

C1.4 Site Clearing

1. Clearing and grubbing should consist of the removal from areas to be graded: all existing pavement, utilities, and vegetation.
2. Organic and inorganic materials resulting from the clearing and grubbing operations should be hauled away from the areas to be graded.

C1.5 Excavations

Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

C1.6 Preparation of Fill Areas

1. All organic soils and debris should be removed from the proposed foundation, improvements and/or fill areas.
2. After the required removals have been made, the exposed earth materials (undocumented artificial fill and the upper low-density portions of the alluvium), should be overexcavated to provide a zone of structural fill for the support of footings, slabs-on-grade, and exterior flatwork or proposed improvements. All loose, soft or disturbed earth materials should be removed from the bottom of excavations before placing structural fill. Any structures will require a minimum of 3.0 feet of compacted fill beneath building footings and 2.0 feet below any proposed wall footings.

3. The subgrade in all areas to receive fill should be scarified to a minimum depth of 6 inches. Scarification may be terminated on moderately hard to hard, cemented earth materials with the approval of the Geotechnical Engineer. The soil moisture should be adjusted to at least 2 percent above optimum for fine-grained soils and within 3 percent of optimum moisture content for granular soils, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.
4. Compacted fill may be placed on native soils that have been properly scarified and recompacted as discussed above.
5. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

C1.7 Placement and Compaction of Fill

1. Compacted fill placed for the construction of the embankment or for any planned structures will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.
2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer should be placed in layers on approved earth materials. Soils used as compacted structural fill should have the following characteristics:
    a. All fill soil particles should not exceed 8 inches in nominal size and should not have an organic content greater than 2%, as well as free of matter and miscellaneous inorganic debris and inert rubble.
    b. Imported fill materials should have an Expansion Index (EI) less than 50. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) at about 2 percent above optimum moisture for fine-grained soils, and within 3 percent of optimum for granular soils.
3. Fill exceeding 5 feet in height should not be placed on native slopes that are steeper than 5:1 horizontal:vertical (H:V). Where native slopes are steeper than 5:1 H:V, and the height of the fill is greater than 5 feet, the fill should be benched into competent materials. The height and width of the benches should be at least 2 feet.
4. Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

5. Fill materials should not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations should not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.
6. It should be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It should be the Contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

C1.8 Fill Slope Construction

1. Fill slopes placed above existing surfaces or cut slopes should be constructed with keyways.
2. Where fill is placed against existing slopes steeper than 5:1 H:V, the new fill slopes should be keyed and benched to provide increased lateral support after removal of the unsuitable surficial soils, when present.

C1.9 Observation and Testing

1. During the progress of grading and trench backfill, the Geotechnical Engineer will provide observation of the fill placement operations.
2. Field density tests of all compacted fill will be made during grading and trench backfill to provide an opinion on the degree of compaction being obtained by the Contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content should be made as necessary, until the required degree of compaction is obtained.
3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.