



June 24, 2024
(Revised March 26, 2026)
Kleinfelder Project No.: 20236170.006A

Mr. Gil Mars
City of Ontario
1333 South Bon View Avenue
Ontario, California 91761

**SUBJECT: Revised Report of Geotechnical Study
Proposed Anthony Munoz Community Center Gymnasium
1242 West 4th Street
Ontario, California**

Dear Mr. Mars:

Kleinfelder is pleased to present this report summarizing our geotechnical study for the proposed Anthony Munoz Community Center Gymnasium located at 1242 West 4th Street in Ontario, California. This report was revised on March 27, 2026 to provide the current project site address. Although stated in our proposal submitted on August 10, 2023, the infiltration testing portion of our original scope was removed per City of Ontario request due to there being an infiltration system already onsite.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Sincerely,

KLEINFELDER

Jose A. Zuniga, EIT
Project Engineer

Jeffery D. Waller, PE, GE
Principal Geotechnical Engineer



**REVISED REPORT OF GEOTECHNICAL STUDY
PROPOSED ANTHONY MUNOZ COMMUNITY
CENTER GYMNASIUM
1242 WEST 4TH STREET
ONTARIO, CALIFORNIA**

KLEINFELDER PROJECT NO.: 20236170.006A

**JUNE 24, 2024
(REVISED MARCH 26, 2026)**

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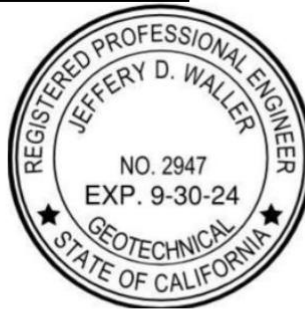
Mr. Gil Mars
City of Ontario
1333 South Bon View Avenue
Ontario, California 91761

**REVISED REPORT OF GEOTECHNICAL STUDY
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1242 WEST 4TH STREET
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EXECUTIVE SUMMARY

This report presents the results of Kleinfelder's geotechnical study for the proposed Anthony Munoz Community Center Gymnasium located at 1242 West 4th Street in Ontario, California. The purpose of our geotechnical study was to evaluate soil and groundwater conditions at the site and provide geotechnical recommendations for project design and construction.

Kleinfelder understands that the proposed project will consist of constructing a new building to be used as a Community Center Gymnasium in Ontario, California. The area of the proposed building is currently used as a public park that consists of open grass fields and occasional medium to large trees. The proposed building will contain space for a gymnasium, workout area, restrooms, lobby/reception, and storage areas. The proposed building is anticipated to be one story in height with a footprint of approximately 15,500 square feet. The remainder of the site will consist of asphalt concrete or Portland cement concrete surface parking and driveways. Appurtenant construction includes landscape and hardscape improvements, and underground utilities.

Subsurface conditions at the site were explored by drilling five (5) borings to depths of approximately 11½ to 51½ feet below the existing ground surface (bgs). The site is underlain by artificial fill overlying alluvial soil. Boreholes drilled at the site generally encountered granular soil with varying amounts of silt and gravel. Artificial fill was encountered in boreholes drilled at the site extending to a depth of approximately 2 feet bgs. Based on our review of aerial images, it appears that a sand volleyball court was previously located in the eastern portion of the site. The sand volleyball court was removed in 2020 and is now part of the open grass fields, however, the depth of artificial fill is unknown in this area but should be reviewed in demolition documents. Groundwater was not encountered in any of the borings drilled during this investigation to the maximum depth explored of approximately 51½ feet bgs.

Based on the results of our field exploration, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in the geotechnical report are incorporated into the project design and construction. We identified the following key geotechnical considerations during our study.

- The proposed building may be supported on conventional shallow spread footing foundations. Based on our field investigation, artificial fill up to approximately 2 feet bgs was encountered in the proposed building area at the site. Based on the similarity in appearance of the fill and the underlying alluvial soils, the depth of fill should be confirmed during grading. In order to provide uniform support for proposed structures, we

recommend the existing soils be overexcavated to a depth of at least 5 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper, and replaced as engineered fill. The overexcavation should extend beyond the proposed improvements a horizontal distance of at least 5 feet, or equal to the depth of the engineered fill placed. As discussed in our report, deeper overexcavation may be required in some areas.

- Due to the potential for consolidation due to excessive moisture, landscaping should not be placed directly adjacent to the building. Additionally, planters should be detailed such that irrigation water will not seep into the foundation areas or beneath slabs and pavement.
- The on-site soils, minus debris, organic matter, or other deleterious materials, may be used as structural fill. Rock or other soil fragments greater than 3 inches in size should not be used in the fills beneath proposed structures. The on-site soils should be compacted to at least 90 percent of the maximum dry unit weight (ASTM D1557). The upper 12 inches of fill below the floor slab and pavements should be compacted to at least 95 percent of the maximum dry unit weight.
- The minimum resistivity of the soil tested indicates that the soil may be moderately corrosive to ferrous metals. The concentrations of soluble sulfates indicate that the subsurface soils represent a Class S0 exposure to sulfate attack on concrete in contact with the soil based on ACI 318-19 Table 19.3.1.1 (ACI, 2019). Therefore, in accordance with ACI Building Code 318-19, no special provisions for selection of cement type are required.

The findings, conclusions, and recommendations presented in this executive summary should not be relied upon without consulting our geotechnical report for more information. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 7.

1 INTRODUCTION

This report presents the results of Kleinfelder's geotechnical study for the proposed Anthony Munoz Community Center Gymnasium located at 1242 West 4th Street in Ontario, California. The location of the project site is presented on Figure 1, Site Vicinity Map. The purpose of our geotechnical study was to evaluate soil and groundwater conditions at the site and provide geotechnical recommendations for project design and construction. The scope of our services was presented in our proposal dated August 10, 2023.

This report presents a description of the services performed, a discussion of the geotechnical conditions observed at the site, and recommendations developed from our engineering analyses of field and laboratory data. Individuals using this report should read the limitations presented in Section 7.

1.1 PROJECT DESCRIPTION

Kleinfelder understands that the proposed project will consist of constructing a new building to be used as a Community Center Gymnasium in Ontario, California. The proposed building is located at 1242 West 4th Street. The area of the proposed gymnasium is currently used as a public park with open grass fields. The proposed building is anticipated to be one story in height and the footprint is approximately 15,500 square feet. The remainder of the site will consist of asphalt concrete or Portland cement concrete surface parking and driveways. Appurtenant construction includes landscape and hardscape improvements, and underground utilities. Structural loading conditions were not available at the time of the preparation of this report. For the purposes of our recommendations, we assumed the structural loading for the building will be less than 100 kips for columns and less than 4 kips per lineal foot for perimeter wall loads. We anticipate slab loads will be less than 300 pounds per square foot (psf).

We received a preliminary plan prepared by Robert R. Coffee Architect & Associates (RCA+A) dated, February 12, 2019. Based on review of the preliminary documents provided to us by City of Ontario the topography at the site is nearly flat to gently descending to the south, with topographic relief estimated to be on the order of less than 5 feet. Drainage is generally toward the south at an average gradient estimated at approximately less than 2 percent. At the time of our field investigation there was open grass field with few sporadic medium to large trees.

1.2 SCOPE OF SERVICES

The scope of our geotechnical study consisted of a literature review, subsurface explorations, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this report. Studies to assess environmental hazards that may affect the soil and groundwater at the site were beyond our geotechnical scope of services. The following paragraphs present a description of our services.

1.2.1 Task 1 – Background Data Review

We reviewed readily-available published and unpublished geologic literature in our files and the files of public agencies, including selected publications prepared by the California Geological Survey and the U.S. Geological Survey (USGS). Readily-available on-line photos were reviewed for past site development and use. We also reviewed readily available seismic and faulting information, including data for designated earthquake fault zones as well as our in-house database of faulting in the general site vicinity.

1.2.2 Task 2 – Field Exploration

Subsurface conditions at the site were explored by drilling five (5) borings to depths of approximately 11½ to 51½ feet below the existing ground surface (bgs). The site location and approximate boring locations are shown on Figure 2, Exploration Location Map.

Prior to commencement of the fieldwork, Underground Service Alert (USA) was notified and various geophysical techniques were used at the boring locations to identify potential conflicts with subsurface structures. A Kleinfelder staff engineer supervised the field operations and logged the explorations. Selected bulk and drive samples were retrieved, placed in plastic bags or sealed in sample tubes, and transported to our laboratory for further evaluation. The number of blows necessary to drive a Standard Penetration Test (SPT) sampler or California-type sampler was recorded. Soil descriptions used on the logs result from field observations and data, as well as from laboratory test data. Stratification lines on the logs represent the approximate boundary between soil and/or rock types, and the actual transition may vary and can be gradual. Appendix A, Field Explorations presents a description of the field exploration program, exploration logs, and a legend of terms and symbols used on the logs.

1.2.3 Task 3 – Laboratory Testing

Laboratory testing was performed on representative samples to assist in soil classification and development of engineering parameters for geotechnical design. We performed laboratory testing consisting of in-situ moisture content, dry unit weight, sieve analysis modified Proctor and R-value. Collapse potential, corrosivity testing, consolidation, and direct shear was performed by AP Engineering & Testing, Inc., of Pomona, California. Appendix B, Laboratory Testing presents the results of the laboratory testing performed for this study.

1.2.4 Task 4 – Geotechnical Analyses

We analyzed field and laboratory data in conjunction with the assumed finished grades, site layout, and assumed structural loads to provide geotechnical recommendations for design and construction. We evaluated feasible foundation systems, concrete slab support, pavement design, and earthwork.

1.2.5 Task 5 – Report Preparation

This report summarizes the services performed, data acquired, and our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed improvements. Our report includes the following items:

- Vicinity map and exploration location map showing the approximate boring and infiltration test locations;
- Logs of borings (Appendix A);
- Results of laboratory tests (Appendix B);
- Discussion of general site conditions;
- Discussion of general subsurface conditions as encountered in our field exploration, including the depth to groundwater, if encountered;
- An assessment of seismic hazards;
- Recommendations for site preparation, earthwork, temporary slope inclinations, fill placement, and compaction specifications;
- Recommendations for foundation design, including allowable bearing pressures, embedment depths, etc., under various loading conditions, and discussion of potential foundation alternatives, if needed;

- Anticipated total and differential settlements based on loading conditions presented above;
- Recommendations for seismic design parameters in accordance with Chapter 16A of the 2022 CBC and ASCE 7-16;
- Preliminary evaluation of the corrosion potential of the on-site soils.

2 SITE CONDITIONS

2.1 SITE DESCRIPTION

The site is currently used as a public park that consists of open grass fields and occasional medium to large trees. Based on review of historical imagery, it appears that a sand volleyball court was previously located in the eastern portion of the site. The sand volleyball court was removed in 2020 and used as construction space from 2020 to 2021. Now it is part of the open grass fields. The site is bounded by the existing Anthony Munoz Community Center to the east, 4th Street to the south, single-family residences to the west and additional open grass field to the north. The site is relatively level.

3 SUBSURFACE CONDITIONS

3.1 REGIONAL AND LOCAL GEOLOGY

California is divided into eleven natural geomorphic provinces that are recognized based on geology, landscape or landform, topographic relief, and climate (CGS, 2002). The project is in the northeastern part of the greater Los Angeles Basin, which is located between the Peninsular Ranges geomorphic province to the south and the Transverse Ranges geomorphic province to the north. The Peninsular Ranges are a series of northwest-trending mountain ranges comprised of mostly granitic rocks intruding sedimentary rocks and older metamorphic rocks (Norris and Webb, 1990). The Transverse Ranges are an east-west trending series of steep mountain ranges and valleys that are oblique to the overall northwest structural trend of most of California.

The site is in the San Bernardino sub-basin part of the greater Los Angeles Basin. The site occupies a broad alluvial plain that extends from the San Gabriel Mountains on the north to the Jurupa Hills on the south and connects the Chino Valley on the west to the San Bernardino Valley on the east.

3.2 SUBSURFACE CONDITIONS

The site is underlain by artificial fill overlain by alluvial soil. Boreholes drilled at the site generally encountered silty sands with varying amounts gravel, typical of the area. Artificial fill was encountered in boreholes drilled at the site extending to a depth of approximately 2 feet bgs.

A discussion of the subsurface materials encountered is presented in the following sections. Detailed descriptions of the deposits are provided in our boring logs presented in Appendix A.

3.2.1 Artificial Fill

Artificial fill was encountered in all the geotechnical borings drilled at the site. Artificial fill extended to a depth of approximately 2 feet bgs. Based on the similarity in appearance of the fill and the underlying alluvial soils, the depth of fill is difficult to estimate with certainty during drilling, so it should be confirmed by Kleinfelder personnel during grading. The fill generally consists of silty sands.

3.2.2 Alluvium

Alluvial deposits underlie the fill to the maximum depth explored of approximately 51½ feet bgs. The alluvium consists of medium dense to very dense silty sand with varying amounts of gravel.

Trace cobbles were encountered in our borings and there is a minor potential for limited boulders to be present within the alluvial deposits based on our experience in the area. In-situ moisture content ranged from 1.5 to 9.6 percent and dry unit weight ranged from 107.7 to 128.1 pcf.

3.3 GROUNDWATER

Groundwater was not encountered in our borings to the maximum depth explored of approximately 51½ feet bgs during our geotechnical investigation. Historic groundwater data obtained from the California Department of Water Resources, Water Data Library (CDWR, 2023). One well (340777N1176616W001) located approximately 0.7 miles to the west measured water level at approximately 409 feet bgs in October 1954. More recent readings at the same well measured water level at approximately 464 feet bgs in March 2023. The current depth to ground water is estimated to be greater than 51½ feet bgs based on the observations and historical data.

Fluctuations of the groundwater level, localized zones of perched water, and high soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or immediately adjacent to the site can also cause a fluctuation of local groundwater levels.

3.4 ASSESSMENT OF POTENTIAL GEOLOGIC HAZARDS

3.4.1 Faulting and Seismicity

The project is in seismically active southern California. The project is within the zone of influence for several faults which are considered active. The California Alquist-Priolo (A-P) Earthquake Fault Zoning Act defines an active fault as one which has “had surface displacement within Holocene time” (the last 11,700 years).

Faults are identified by the State of California or the County of San Bernardino as being active are not present within the project limits (San Bernardino Co., 2009). No portion of the site is located within an A-P Fault Zone (CGS, 2020). The nearest active faults to the site is the Red Hill-Etiwanda Avenue Fault located approximately 3 miles to the northeast of the site and also the San Jose fault located approximately 3 miles northwest. The Chino fault is located approximately 5.2 miles southwest (USGS, 2020).

There are no known active faults that cross the area of the project or that have been mapped in the vicinity of the project (Morton and Miller, 2006; USGS, 2020), consequently the risk of surface rupture at the site resulting from faulting is considered low.

The project is within the zone of influence for several faults which are considered active and which are capable of producing seismic shaking at the site that could be potentially damaging to buildings and appurtenant structures. It is anticipated that the project site will periodically experience ground acceleration as the result of moderate to large magnitude earthquakes on faults in or outside the project area.

3.4.2 Flooding and Inundation

The site is located away from the floodplain areas of the two nearest drainages, San Antonio Creek to the west and Cucamonga Creek to the east. The site is not within any currently designated flood hazard zone (FEMA, 2021). The site is outside the inundation areas for the Upland Basin on San Antonio Creek and the Cucamonga Creek Debris Basin on Cucamonga Creek (DSOD, 2021). The site is not located adjacent to or downgradient of any surface water impoundments that present a seiche hazard. There is no risk of tsunami hazard due to the elevated and inland location of the site. Consequently, the flooding and inundation hazard at the site is considered to be low to very low.

3.4.3 Landslide

Landslides and other forms of mass wasting, including mud flows, debris flows, soil slips, and rock falls occur as soil or rock moves down slope under the influence of gravity. Landslides may be triggered by intense rainfall or seismic shaking. The site is not within a State or County designated hazard zone for landslides (CGS, 2020; San Bernardino Co., 2009). Because the site is relatively flat, the risk of landslides at the site is considered very low.

3.4.4 Liquefaction

Liquefaction occurs when loose, saturated, coarse-grained or silty soils are subjected to strong shaking resulting from earthquake motions. Coarse-grained or silty soils typically lose part or all of their shear strength while undergoing shaking and regain strength sometime after the shaking stops. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils.

The site is not within a San Bernardino County designated zone of liquefaction susceptibility (San Bernardino Co., 2009). Groundwater was not encountered in borings drilled during this investigation to the maximum depth explored of approximately 51 feet bgs, and the current depth to groundwater in the vicinity of the site is over 400 feet bgs.

3.4.5 Subsidence and Earth Fissures

In general, land subsidence is the sinking or settlement of the ground surface due to the rearrangement of subsurface materials. Over 80 percent of the documented cases of land subsidence in the United States have been caused by groundwater extractions from the underlying aquifer-system (USGS, 1999). However, subsidence in the Chino sub-basin is concentrated in the Chino Valley southwest of the site, and the site is not located on or near the boundary of the area of subsidence concern (WEI, 2019). Consequently, the risk of land subsidence and earth fissures due to groundwater withdrawal at the site is considered low.

3.4.6 Expansive Soils

The fill and alluvial soils encountered in our borings consisted primarily of interbedded sand and silty sand with varying amounts of gravel. Accordingly, the near surface soil can be considered to have very low potential for expansion.

3.4.7 Hydro-Collapsible Soils

Hydro-collapsible soils are characterized by their ability to undergo significant shrinkage (collapse) during inundation. Inundation in soils can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may result in unacceptable settlement of structures or concrete slabs supported on grade. Based on the results of laboratory testing, the collapse potential of the soils is approximately 0.2 percent collapse under inundation. Collapse potential less than 2 percent is considered low.

3.4.8 Oil Fields

There are no oil or gas wells within 3 miles of the site according to the State online well tracking system (DOC, 2023). Because of the distance of the site from known oil fields or wells and the nature of the geology materials at the site, the potential for naturally occurring hydrocarbons in subsurface soils at the site is considered very low.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our field exploration, laboratory testing and engineering analyses conducted during this study, it is our professional opinion the proposed project is geotechnically feasible, provided the design and construction recommendations presented in this report are incorporated into the project.

The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in the borings, the results of the laboratory-testing program, and our engineering analyses performed, and should be incorporated into project design and construction.

4.2 SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active region and the proposed development can be expected to be subjected to moderate to strong seismic shaking during its design life. The following sections discuss seismic design considerations with respect to the project site.

4.2.1 2022 CBC Seismic Design Parameters

According to the 2022 California Building Code, every structure, and portion thereof, including non-structural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE/SEI 7-16. The Seismic Design Category for a structure may be determined in accordance with Section 1613 of the 2022 CBC.

Based on data obtained from our field explorations, published geologic literature and maps, and on our interpretation of the 2022 CBC criteria, it is our opinion that the project site may be classified as Site Class D, Stiff Soil, according to Section 1613 of 2022 CBC and Table 20.3-1 of ASCE/SEI 7-16 (2016). Approximate coordinates for the site are noted below.

- Latitude: 34.078401°N
- Longitude: 117.674679°W

The Risk-Targeted Maximum Considered Earthquake (MCER) mapped spectral accelerations for 0.2 seconds and 1 second periods (S_s and S_1) were estimated using Section 1613 of the 2022 CBC and the California Office of Statewide Planning and Development (OSHPD) seismic design maps web-based application (available at <https://seismicmaps.org/>). In accordance with Section

11.4.8 of ASCE 7-16, a site-specific ground motion hazard analysis is required for Site Class D sites with an S_1 greater than 0.2 g. However, a site-specific ground motion hazard analysis is not required if the exception in Section 11.4.8 of ASCE 7-16 are taken. In accordance with the 2022 CBC, which adopts Supplement 3 of ASCE 7-16, the exception requires the values of parameters S_{M1} and S_{D1} to be increased by 50 percent. The assumption that the exception will be used should be verified by the project structural engineer. Based on the assumption that the exception will be taken, the 2022 CBC Seismic Design Parameters (non-site-specific) for the project site are provided in Table 1.

TABLE 1
2022 CBC SEISMIC DESIGN PARAMETERS

DESIGN PARAMETER	RECOMMENDED VALUE
Site Class	D
S_s (g)	1.642
S_1 (g)	0.6
F_a	1.0
F_v	N/A*
S_{MS} (g)	1.642
S_{M1} (g)	N/A*
S_{DS} (g)	1.095
S_{D1} (g)	N/A*
PGA_M (g)	0.753

* N/A = Not Applicable; Section 11.4.8 of ASCE 7-16 requires a site-specific ground motion hazard analysis be performed for Site Class D sites with S_1 values greater than or equal to 0.2g unless exceptions are taken in which the values of S_{M1} and S_{D1} are increased by 50 percent. If exceptions are taken, then a F_v value of 1.7 may be used in accordance with Table 11.4-2 of ASCE 7-16, Supplement 3 (per the 2022 CBC).

4.2.2 Liquefaction

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and some silt.

The site is not within a San Bernardino County designated zone of liquefaction susceptibility (San Bernardino Co., 2023). Because of the depth to groundwater, the potential for liquefaction at the site is not considered a hazard at the site.

4.3 FOUNDATIONS

4.3.1 General

Based on the results of our field exploration, laboratory testing, and geotechnical analyses, the proposed building may be supported on conventional shallow spread footing foundations founded on subgrade prepared in accordance with Section 5.2.2. Recommendations for the design and construction of shallow foundations are presented below.

4.3.2 Shallow Foundations

Allowable Soil Bearing Pressure

Continuous and isolated spread footings may be designed for a net allowable soil bearing pressure of 3,000 pounds per square foot (psf) for dead plus sustained live loads. Footings should be embedded at least 24 inches below the lowest adjacent exterior grade. The footing dimension and reinforcement should be designed by the structural engineer; however, continuous and isolated spread footings should have minimum widths of 18 and 24 inches, respectively. A one-third increase in the above bearing pressures can be used for transient wind or seismic loads.

If any utility lines are within a 1:1 (horizontal: vertical) projection from the bottom of a footing, they may be within the influence zone of the proposed footing load. If this condition exists, the proposed footing should be deepened so that the utility is outside the zone of influence; the utility line could also be relocated or encased with concrete slurry. These conditions should be evaluated on a case-by-case basis. Additionally, there may be utility lines within the vicinity of temporary backcuts for footing over-excavations. Should that occur, shoring may be required to protect the utility lines and site improvements during excavating.

Estimated Settlements

We estimate total static settlement for foundations designed and constructed in accordance with the recommendations presented above to be less than 1 inch. Differential static settlement between similarly loaded footings is estimated to be less than ½ inch over 50 feet.

Lateral Resistance

Lateral load resistance may be derived from passive resistance along the vertical sides of the footings, friction acting at the base of the footing, or a combination of the two. An allowable passive resistance of 300 psf per foot of depth may be used for design. Allowable passive resistance values should not exceed 3,000 psf. An allowable coefficient of friction of 0.35 between the base of the footings and the engineered fill soils can be used for sliding resistance using the dead load normal stresses. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab, pavement or in a similar manner.

4.4 BUILDING SLAB-ON-GRADE

Concrete slab-on-grade floors are appropriate for the proposed building, provided the subgrade is prepared in accordance with Section 5.2.2. A modulus of subgrade reaction of 150 pounds per cubic inch (pci) may be used for design of concrete slab-on-grade floors. We recommend the floor slab be a minimum nominal thickness of 6 inches. The reinforcement requirements for the concrete slabs should be determined by the structural engineer.

Floor slab control joints should be used to reduce damage due to shrinkage cracking. Control joint spacing is a function of slab thickness, aggregate size, slump and curing conditions. The requirements for concrete slab thickness, joint spacing, and reinforcement should be established by the designer, based on experience, recognized design guidelines and the intended slab use. Placement and curing conditions will have a strong impact on the final concrete slab integrity.

A vapor barrier should be placed beneath the slab in areas where moisture sensitive flooring will be used. The moisture vapor retarder product should meet the performance standards of an ASTM E1745, Class A material, and be properly installed in accordance with ACI publication 302. The vapor retarder should be at least 10 mils thick and be properly lapped and sealed. The joints between the sheets and the openings for utility piping should be lapped and taped. The sheeting should also be lapped into the sides of the footing trenches a minimum of 6 inches. Any puncture of the vapor retarder should be repaired prior to casting concrete.

Normally, a thin layer of clean sand (about 2 inches thick) is placed on the sheeting to facilitate concrete curing and to decrease the likelihood of slab curling. The final decision for the need and thickness of sand above the vapor barrier is the purview of the slab designer/structural engineer. The moisture vapor retarder is intended only to reduce moisture vapor transmission from the soil

beneath the concrete and will not provide a waterproof or vapor proof barrier or reduce vapor transmission from sources above the retarder.

It should be noted that this system, although currently the industry standard, may not be completely effective in preventing moisture transmission through the floor slab and related floor covering problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels will be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building, and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete (water-cement ratio) and the permeability of the on-site soils affect slab moisture and can influence performance. In many cases, floor moisture problems are the result of water-cement ratio, improper curing of floor slabs, improper application of flooring adhesives, or a combination of these factors. Studies have shown that concrete water-cement ratios lower than 0.5 and proper slab curing can significantly reduce the potential for vapor transmission through floor slabs. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications. Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

4.5 EXTERIOR FLATWORK

Exterior concrete slabs for pedestrian traffic or landscape should be at least 4 inches thick. The subgrade should be prepared in accordance with Section 5.2.2.

4.6 SITE DRAINAGE

Foundation and slab performance depends greatly on proper irrigation and how well runoff water drains from the site. This drainage should be maintained both during construction and over the

entire life of the project. The ground surface around structures should be graded such that water drains away from structures without ponding. The surface gradient needed to do this depends on the landscaping type. In general, landscape areas within 5 feet of buildings should slope away at gradients of at least 2 percent. Surface gradients should conform to the 2022 CBC.

Planters should be detailed such that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. Pavement underdrains at drainage inlets and catch basins should be included and should extend at least 15 feet from the catch basins. Perimeter foundation drains are not necessary.

Where slabs or pavement areas abut landscaped areas, the aggregate base and subgrade soil should be protected against saturation. Vertical cut off structures are recommended to reduce lateral seepage under slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened concrete perimeters, or equivalent, extending at least three inches below the base/subgrade interface. Vertical cut-off structures should be poured neat against undisturbed native soil or compacted fill. The cut-off structures should be continuous.

Operations personnel should be instructed to limit irrigation to the minimum level necessary to properly sustain landscaping plants. Should excessive irrigation, waterline breaks or unusually high rainfall occur, saturated zones and “perched” groundwater may develop, which could soften subgrade and reduce pavement life, and could also create potholes. We also recommend that the downspouts from roof drains be connected to a designed subsurface drainage system such as a storm sewer, etc. to avoid discharging water onto pavement areas as well as backfill zones around the proposed building.

Potential sources of water such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired.

Sewer lines beneath the proposed building should have a sufficient slope (at least 1 percent). Plumbing and utility lines should be provided with flexible joints or oversized sleeves where they penetrate floor slabs to prevent breakage caused by different slab movement. In addition, utility trenches should be plugged with cohesive backfill where they enter the building to reduce moisture infiltration along pipe bedding material. The cohesive backfill materials should have a plasticity index (PI) between 15 and 30 and no less than 70 percent of the particles passing the No. 200 sieve.

4.7 RETAINING STRUCTURES

Design earth pressures for retaining structures depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. Fine-grained silts and clays should not be used as retaining structure backfill. The onsite granular (sandy) soils may be used as retaining structure backfill. The earth pressures provided assume that on-site or imported granular (sandy) soils will be used as retaining wall backfill. Granular backfill, which meets the requirements for imported fill as defined in Section 5.2.3, should extend behind walls a horizontal distance of at least one-half the height of the wall. If a drainage system is not installed, the wall should be designed to resist hydrostatic pressure in addition to the earth pressure. Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least $0.002 \times H$, where H is the unbalanced wall height) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The recommended active and at-rest earth pressures and passive resistance values are provided in Table 2.

TABLE 2
LATERAL EARTH PRESSURES FOR RETAINING STRUCTURES
(ON-SITE OR IMPORTED GRANULAR BACKFILL)

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (pcf)	Seismic Increment * (pcf)
Free to Deflect (active condition)	Level	40	20
Restrained (at-rest condition)		60	N/A **

*Note: * Walls supporting more than 6 feet of backfill should be designed to support an incremental seismic lateral pressure, which is applied as a triangular pressure distribution with a maximum pressure at the bottom of the wall, not inverted.*

*** for restrained wall, use the static active earth pressure and seismic increment to check the seismic condition; use at-rest earth pressure only to check the static condition; the larger loading of both cases should be used for the design of restrained wall.*

The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live, including traffic, or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls under active conditions and 0.50 for restrained walls under at-rest conditions. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (250 psf). Lateral load contributions from other surcharges located behind walls may be provided once the load configurations and layouts are known.

Walls should be properly drained or designed to resist hydrostatic pressures. Adequate drainage is essential to provide a free-drained backfill condition so that there is no hydrostatic buildup behind the wall. Walls should also be appropriately waterproofed to reduce the potential for staining. Drainage behind retaining structures can consist of weepholes placed along the base of the wall. Weepholes should be spaced 10 to 15 feet apart and connected with a gravel drain consisting of approximately 3 cubic feet of clean gravel per foot of wall length wrapped with filter fabric. Other types of retaining walls should have a continuous back drain as described below.

Except for the upper 2 feet, the backfill immediately behind retaining walls (minimum horizontal distance of 2 feet measured perpendicular to the wall) should consist of free-draining granular soil or ¾-inch crushed rock wrapped with filter fabric. The upper 2 feet of cover backfill should consist of relatively impervious material. A 4-inch-diameter perforated PVC pipe, placed perforations down at the bottom of the rock layer leading to a suitable gravity outlet, should be installed at the base of the walls.

As an alternative to the gravel drain noted above, a manufactured drain panel may be utilized behind retaining walls in addition to normal waterproofing. This system generally consists of a prefabricated drain panel lined with filter fabric. At the wall base, we recommend that a gravel drain be installed to collect and discharge drainage to a suitable outlet. The drain should consist of a 4-inch-diameter perforated PVC pipe, placed perforations down at the bottom of approximately 3 cubic feet of clean gravel per foot of wall length. The gravel drain should be wrapped in filter fabric (Mirafi 140N or equivalent). The pipe should be sloped to drain to a suitable outlet and cleanouts should be provided at appropriate intervals. If drainage behind the wall is omitted, the wall should be designed for full hydrostatic pressure. The design of any drain panel system should be submitted to Kleinfelder for review to check that our recommendations have been properly incorporated into the design. Installation of the drainage system should be reviewed and documented by a Kleinfelder representative.

4.8 PRELIMINARY PAVEMENT SECTIONS

4.8.1 General

We have provided new asphalt concrete and Portland cement concrete (PCC) pavement sections for the proposed building site. Positive drainage of the paved areas should be provided since moisture infiltration into the subgrade may decrease the life of pavements. Curbing located adjacent to paved areas should be founded in the subgrade, not the aggregate base, in order to provide a cutoff, which reduces water infiltration into the base course.

The anticipated traffic loading for the site was not known at the time of this report. Kleinfelder has provided pavement sections using assumed Traffic Indices of 5 and 7 over an assumed 20-year pavement design life. The following pavement sections are based on the soil conditions encountered during our field exploration, assumptions regarding final site grades, and limited laboratory testing. Subgrade preparation recommendations are provided in Section 5.2.2. Our assumptions and the provided pavement section thickness should be evaluated by the project engineer prior to placement.

4.8.2 Asphalt Concrete Pavement

We have provided the following new preliminary asphalt concrete pavement sections for the proposed building site based on the soil conditions encountered and assumptions stated above. Asphalt concrete pavement sections were evaluated using the design criteria of the latest Caltrans highway design manual and the recommendations are presented below in Table 3.

TABLE 3
RECOMMENDED MINIMUM ASPHALT CONCRETE PAVEMENT SECTIONS

Design R-Value	Assumed Traffic Index, TI	Asphalt Concrete (inches)	Aggregate Base (inches)
50	5	3.0	4.5
	7	4.0	4.5

Asphalt paving materials and placement methods should meet current Caltrans or “Greenbook” specifications for asphalt concrete.

4.8.3 Portland Cement Concrete Pavement

We have provided the following new preliminary PCC pavement sections for the proposed building site based on the soil conditions encountered and assumptions stated above. The PCC pavement section was based on the design procedures from the Portland Cement Association (PCA) and the recommendations are presented below in Table 4.

TABLE 4
RECOMMENDED MINIMUM PCC PAVEMENT SECTIONS

Assumed Traffic Index TI	Concrete Thickness (inches; using a 28-day compressive strength of 4,000 psi)	Concrete Thickness (inches; using a 28-day compressive strength of 3,500 psi)	Concrete Thickness (inches; using a 28-day compressive strength of 3,000 psi)
5	6.0	6.5	6.5
7	6.5	7.5	7.5

The design assumes that the PCC will have a 28-day minimum compressive strength of 3,000, 3,500 or 4,000 psi. It was also assumed that aggregate interlock would be developed at the control joints. Longitudinal and transverse joint spacing should not exceed 12 feet and 15 feet, respectively. Joint details should conform to PCA guidelines. Expansion joints in concrete slabs should be sealed with petroleum resistant sealant to prevent minor releases from impacting subsurface soil.

4.8.4 Aggregate Base

Aggregate base material should meet the specifications for untreated base materials (crushed aggregate base or crushed miscellaneous base) as defined in Section 200-2 of the current edition of the Standard Specifications for Public Works Construction (Greenbook).

4.8.5 Construction Considerations

The pavement sections provided above are contingent on the following recommendations being implemented during construction.

- The subgrade should be prepared in accordance with the recommendations in Section 5.2.2.
- Aggregate base should be compacted to at least 95 percent of the soil's maximum dry unit weight (ASTM D1557). Moisture contents should be maintained near optimum during compaction.
- Subgrade soils should be in a stable, non-pumping condition at the time the aggregate base materials are placed and compacted.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.
- The depth and location of any existing utilities should be verified prior to any earthwork. Based on the utility location, additional recommendations may be needed during construction to limit disturbance of any buried utilities.

4.9 PRELIMINARY SOIL CORROSIVITY

We performed laboratory testing for parameters commonly used to evaluate corrosivity of soils, including pH, minimum resistivity, chloride and soluble sulfate content. Table 5 presents the preliminary corrosivity test results.

**TABLE 5
PRELIMINARY CORROSION TEST RESULTS**

Location	Depth (ft)	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate Content (ppm)	Soluble Chloride Content (ppm)
B-1	0-5	9,685	7.6	21	29

These tests are a generalized indicator of soil corrosivity for the sample tested. Other soils on site may be more, less, or similarly corrosive in nature. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than those noted.

Resistivity values between 5,000 and 10,000 ohm-cm are considered moderately corrosive to buried ferrous metals (Roberge, 2006). The concentrations of soluble sulfates indicate that the

subsurface soils represent a Class S0 exposure to sulfate attack on concrete in contact with the soil based on ACI 318-19 Table 19.3.1.1 (ACI, 2019). Therefore, in accordance with ACI Building Code 318-19, no special provisions for selection of cement type are required.

Kleinfelder's scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included. Kleinfelder recommends that a qualified corrosion engineer should be retained to further investigate and evaluate and design protective systems, if considered necessary.

5 CONSTRUCTION RECOMMENDATIONS

5.1 GENERAL

The following recommendations should be used by the contractor for construction of the project.

5.2 EARTHWORK

5.2.1 General

Site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications, and the recommendations included in this report. References to maximum dry unit weights are established in accordance with the latest version of ASTM Test Method D1557 (modified Proctor). The earthwork operations should be observed and tested by a representative of Kleinfelder.

5.2.2 Site Preparation

All abandoned utilities, foundations, and other existing improvements within the proposed improvements should be removed and the excavation backfilled with engineered fill. All debris produced by demolition operations, including wood, steel, piping, plastics, etc., should be disposed off-site. Existing utility pipelines or conduits that extend beyond the limits of the proposed construction and are to be abandoned in place, should be plugged with non-shrinking cement grout to prevent migration of soil and/or water. Demolition, disposal and grading operations should be observed and tested by a representative of the geotechnical engineer. Areas to receive fill should be stripped of all dry, loose or soft earth materials and undocumented fill materials to the satisfaction of the Geotechnical Engineer.

- Structural Areas: Based on our field investigation, artificial fill up to approximately 2 feet bgs was encountered at the site. Based on the similarity in appearance of the fill and the underlying alluvial soils, the depth of fill is difficult to estimate with certainty during drilling, so it should be confirmed during grading by Kleinfelder personnel. In order to provide uniform support for proposed structures, we recommend the existing soils be overexcavated to a depth of at least 5 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper, and replaced as engineered fill. The overexcavation should extend beyond the proposed improvements a horizontal distance of at least 5 feet, or equal to the depth of the engineered fill placed. As discussed below, deeper overexcavation may be required in some areas.

- Non-Structural Areas: For non-structural areas, such as pavements, sidewalks and other flatwork, etc., we recommend that the existing soils be overexcavated a minimum of 18 inches below existing grade or finished subgrade, whichever is deeper, and be replaced as engineered fill. The overexcavation should extend beyond the proposed improvements a horizontal distance of at least 2 feet. In our experience, the existing undocumented fill within non-structural areas is not always overexcavated and recompacted. The risk of settlement associated with leaving undocumented fill in place can be reduced by inspection of the final overexcavation by a Kleinfelder engineer or geologist. Additionally, the project site should include appropriate drainage to minimize the likelihood of adverse moisture infiltration. Mitigation may include surface drainage to storm drains; utilizing low-volume emitter irrigation; and timely repair of any malfunctioning irrigation.

After site preparation and prior to placement of compacted fills, the excavation bottom should be observed and approved by a Kleinfelder engineer or geologist. If unsatisfactory material, such as soft or disturbed soil, debris or otherwise unsuitable soil is present, deeper overexcavation will be required. After approval, the overexcavation bottom should be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content and compacted to at least 90 percent of the maximum dry unit weight.

Due to the potential for soil consolidation due to excessive moisture, landscaping should not be placed directly adjacent to the building. Additionally, planters should be detailed such that water existing from them will not seep into the foundation areas or beneath slabs and pavement.

5.2.3 Fill Material and Compaction Criteria

The on-site soils, minus debris, organic matter, or other deleterious materials, may be used as structural fill. Rock or other soil fragments greater than 3 inches in size should not be used in the fills beneath the proposed structures. Particles up to 6 inches in diameter may be placed outside of the structure areas. Nesting of cobbles should be avoided if included within the fill materials.

The on-site soils should be compacted to at least 90 percent of the maximum dry unit weight (ASTM D1557). The upper 12 inches of fill below the floor slab and pavements should be compacted to at least 95 percent of the maximum dry unit weight. Fill should be placed in loose horizontal lifts not more than 8 inches thick (loose measurement). The moisture content of the fill should be maintained near optimum during compaction and until the aggregate base is placed and compacted for pavement areas.

Import soils, if required, should have no particles greater than 3 inches in maximum dimension, no less than 70 percent of the particles passing the No. 4 sieve, no more than 20 percent of particles passing the No. 200 sieve, and an Expansion Index (EI) less than 20. The contractor should provide documentation that all imported soil is free of hazardous materials, including petroleum or petroleum byproducts, chemicals and harmful minerals. Test results with the geotechnical and analytical properties of the proposed import material should be provided for approval prior to transportation and use on site.

Processing may require ripping the material, disking to break up clumps, and blending to attain uniform moisture contents necessary for compaction. Utility trench backfill should be mechanically compacted. Flooding should not be permitted.

5.2.4 Excavation Characteristics

Borings drilled as part of our field exploration were advanced using hollow-stem-auger drilling equipment. Excavation effort was easy to moderate within the fill and alluvial soils. It is anticipated that conventional heavy-duty earthmoving equipment maintained in good condition should be capable of excavating the soil. Limited cobbles may also be present within the alluvial deposits based on our experience in the area. During seasonal rains, handling of saturated soils may pose problems with equipment access and cleanup, and we suggest the materials be allowed to dry out, if possible, prior to excavation.

5.2.5 Temporary Excavations

All excavations must comply with applicable local, state, and federal safety regulations, including OSHA requirements for Type C soil. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor. We are providing this information below solely as a service to our client. Under no circumstances should this information provided be interpreted to mean that Kleinfelder is assuming responsibility for final engineering of excavations or shoring, construction site safety, or the contractors' activities; such responsibility is not being implied and should not be inferred.

Minor sloughing and/or raveling of slopes should be anticipated as they dry out. Where space for sloped embankments is not available, shoring will be necessary. In addition, excavations within a 1:1 plane extending downward from a horizontal distance of 2 feet beyond the bottom outer edge of existing improvements should not be attempted without bracing and/or underpinning the footings, as discussed above. The geotechnical engineer or their field representative should observe the excavations so that modifications can be made to the excavations, as necessary,

based on variations in the encountered soil conditions. All applicable excavation safety requirements and regulations, including OSHA requirements, should be met.

All trench excavations should be braced and shored in accordance with good construction practice and all applicable safety ordinances and codes. Stockpiled (excavated) materials should be placed no closer to the edge of an excavation than a distance equal to the depth of the excavation, but no closer than 4 feet.

5.2.6 Trench Backfill

Pipe zone backfill (i.e. material beneath and in the immediate vicinity of the pipe) should consist of imported soil less than $\frac{3}{4}$ -inch in maximum dimension. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of onsite soil or imported fill that meets the requirements for engineered fill provided above.

If imported material is used for trench zone backfill, we recommend it consist of silty sand. In general, gravel should not be used for trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local building requirements and/or bedding requirements for specific types of pipes. We recommend the project civil engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

Trench backfill should be placed and compacted in accordance with recommendations provided for engineered fill in Section 5.2.3. Mechanical compaction is recommended; ponding or jetting should be avoided, especially in areas supporting structural loads or beneath concrete slabs supported on grade, pavements, or other improvements.

5.3 UNSTABLE SUBGRADE CONDITIONS

Should grading be performed during or following extended periods of rainfall, the moisture content of the near-surface soils will also be significantly above the optimum moisture content. These conditions could seriously impede grading by causing an unstable subgrade condition. Typical remedial measures include the following:

- Drying: Drying unstable subgrade involves disking or ripping wet subgrade to a depth of approximately 18 to 24 inches and allowing the exposed soil to dry. Multiple passes of the equipment (likely on a daily basis) will be needed because as the surface of the soil dries, a crust forms that reduces further evaporation. Frequent disking will help prevent the formation of a crust and will promote drying. This process could take several days to several weeks depending on the material, the depth of ripping, the number of passes, and the weather.

5.4 EXTERIOR FLATWORK

Prior to casting exterior flatwork, the subgrade soils should be moisture conditioned and recompacted or overexcavated, as recommended in Section 5.2.2. The moisture content of the subgrade soils should be maintained near optimum prior to the placement of any flatwork or engineered fill. Careful control of the water/cement ratio should be performed to avoid shrinkage cracking due to excess water or poor concrete finishing or curing.

6 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

We recommend Kleinfelder perform a review of geotechnical related portions of the project plans and specifications before they are finalized to see that geotechnical recommendations have been properly interpreted and implemented during design. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations.

6.2 CONSTRUCTION OBSERVATION AND TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because we sample only a limited portion of the soils affecting the performance of the proposed structure, unanticipated or changed conditions can be encountered during grading. Proper geotechnical observation and testing during construction are imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Kleinfelder be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study.

Our services are typically needed at the following stages of grading:

- After demolition (if needed) and grubbing;
- During grading;
- After the overexcavation, but prior to subgrade preparation;
- After excavation for foundations.

- During utility trench backfill; and
- During base placement and site paving.

7 LIMITATIONS

This geotechnical study has been prepared for the exclusive use of the City of Ontario and their agents for specific application to the proposed Anthony Munoz Community Center Gymnasium. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The scope of services was limited to a background data review and the field exploration described in Section 1.2. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our field exploration and laboratory testing programs, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated Traffic Index or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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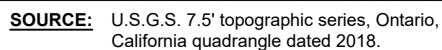
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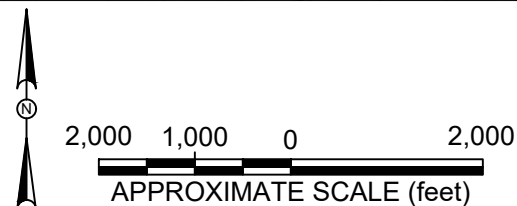
San Bernadino County, 2009, Land Use Plan, Geologic Hazard Overlays for Earthquake Fault Zones, Liquefaction Susceptibility, and Landslide Susceptibility, accessed online 10/24/2023 at <http://cms.sbcounty.gov/lus/planning/zoningoverlaymaps/geologichazardmaps.aspx#Vallery>

Wildermuth Environmental, Inc. (WEI), 2019, Final Report, 2018/19 Annual Report of the Ground-Level Monitoring Committee, Chino Basin Watermaster, Consultant's report dated October 2019.

FIGURES



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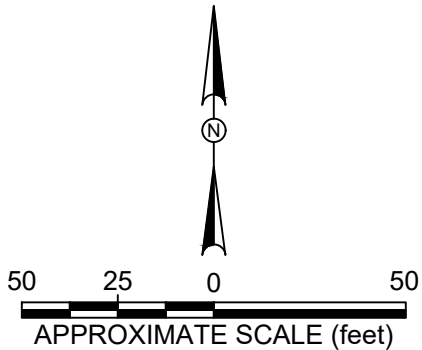
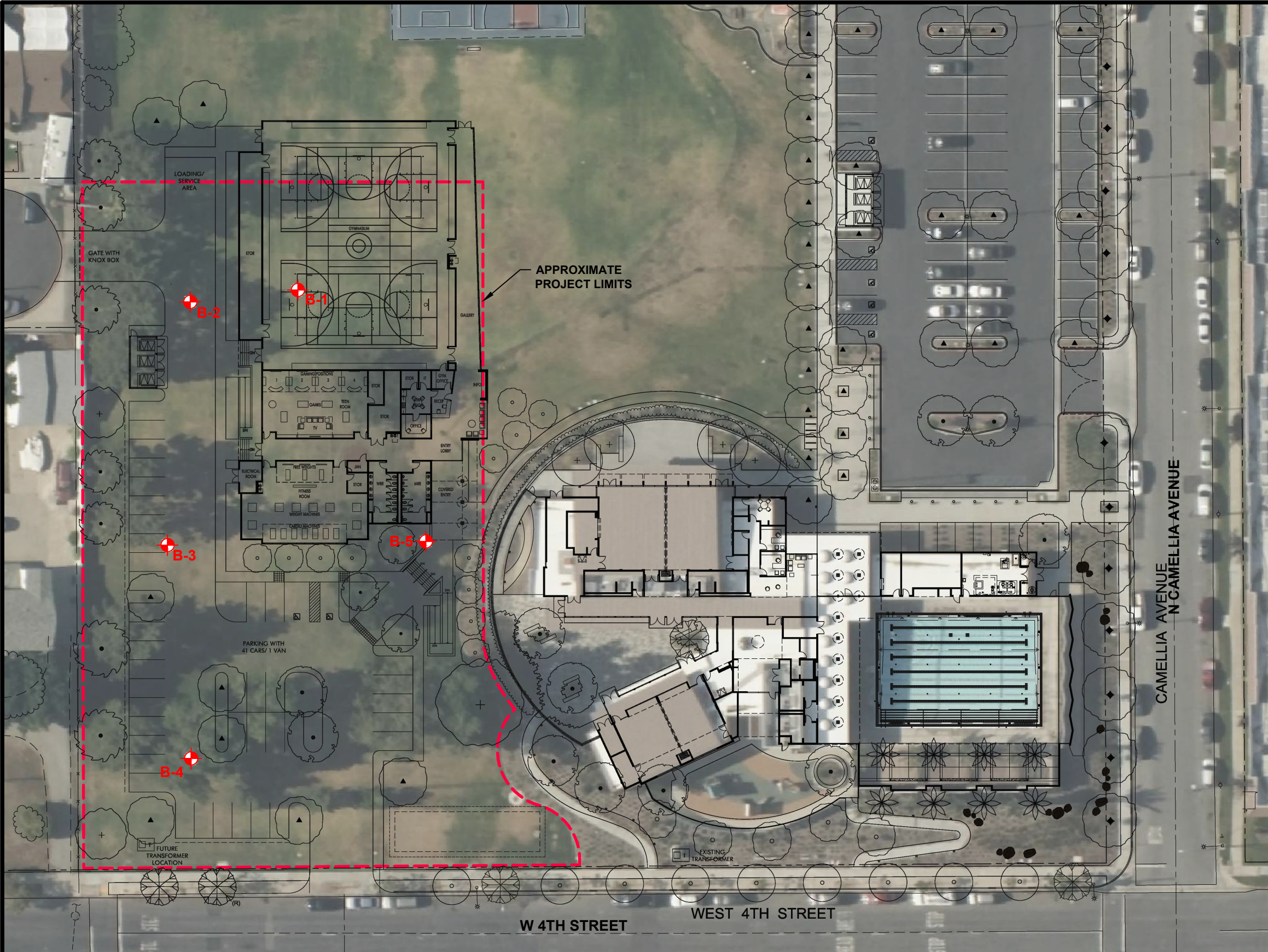


PROJ: 20236170.006A
DRAWN BY: DMF
CHECKED BY: JZ
DATE: 10/2023
REVISED: -

Proposed Anthony Munoz
Community Center Gymnasium
1242 W 4th Street
Ontario, California

FIGURE

1



EXPLANATION

- B-5**  APPROXIMATE BORING LOCATION
-  APPROXIMATE PROJECT LIMITS

REFERENCE: BASE MAP PROVIDED BY RCA+A ARCHITECTS, DATED 02/12/2019

NOTE: PLANS FOR THE LOCATION OF THE PROPOSED GYMNASIUM WERE NOT AVAILABLE AT THE TIME OF REPORT PREPARATION. UPDATED PLANS SHOWING THE PROPOSED GYMNASIUM LOCATION SHOULD BE PROVIDED TO KLEINFELDER TO UPDATE THIS GEOTECHNICAL REPORT.

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BORING LOCATION MAP

Proposed Anthony Munoz
Community Center Gymnasium
1242 W 4th Street
Ontario, California

FIGURE

2

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling five (5) borings to depths of approximately 11½ to 51½ feet below the existing ground surface (bgs). The borings were drilled by 2R Drilling of Chino, California using truck-mounted, hollow-stem auger drilling equipment. The approximate locations of the borings are presented on Figure 2.

Prior to commencement of the fieldwork, Underground Service Alert (USA) was notified and various geophysical techniques were used at the boring locations to identify potential conflicts with subsurface structures.

The boring logs are presented as Figures A-3 through A-8. An explanation to the logs is presented as Figures A-1 and A-2. The boring logs describe the earth materials encountered, samples obtained and show field and laboratory tests performed. The logs also show the location, boring number, drilling date and the name of the drilling subcontractor. The borings were logged by a Kleinfelder engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and drive samples of selected earth materials were obtained from the borings and test pits.

A California-type sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.4-inch I.D. split barrel shaft that is pushed or driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in six 1-inch brass rings for laboratory testing. An additional 2 inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The sampler was driven using a 140-pound hammer falling 30 inches. The total number of blows required to drive the sampler the final 12 inches is termed blow count and is recorded on the boring logs.

Samples were also obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18 inches. The sampler was driven using a 140-pound hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (N) and is recorded on the boring logs. The procedures we employed in the field are generally consistent with those described in ASTM Standard Test Method D1586.

Bulk and grab samples of the near-surface soils were directly retrieved from the auger cuttings from the borings.

DRILLING METHOD/SAMPLER TYPE GRAPHICS

	BULK SAMPLE
	CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
	GRAB SAMPLE
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

	WATER LEVEL (level where first observed)
	WATER LEVEL (level after stabilizing period)
	WATER LEVEL (additional levels after exploration)
	OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Solid lines separating strata on the logs represent approximate boundaries only, dashed lines are inferred or extrapolated boundaries. Actual transitions may be gradual or differ from those represented.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System (ASTM D2488/D2487) designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., CL-ML, GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer

WOR - Weight of Rod

REFERENCES

1. American Society for Materials and Testing (ASTM), 2011, ASTM D2487: Classification of Soils for Engineering Purposes (Unified Soil Classification System).

UNIFIED SOIL CLASSIFICATION SYSTEM¹

COARSE GRAINED SOILS (More than 50% retained on No. 200 Sieve)	GRAVELS (More than 50% of coarse fraction retained on No. 4 Sieve)	CLEAN GRAVEL WITH <5% FINES		GW	WELL-GRADED GRAVEL, WELL-GRADED GRAVEL WITH SAND
				GP	POORLY GRADED GRAVEL, POORLY GRADED GRAVEL WITH SAND
				GW-GM	WELL-GRADED GRAVEL WITH SILT, WELL-GRADED GRAVEL WITH SILT AND SAND
		GW-GC		WELL-GRADED GRAVEL WITH CLAY (OR SILTY CLAY), WELL-GRADED GRAVEL WITH CLAY AND SAND (OR SILT CLAY AND SAND)	
		GP-GM		POORLY GRADED GRAVEL WITH SILT, POORLY GRADED GRAVEL WITH SILT AND SAND	
		GP-GC		POORLY GRADED GRAVEL WITH CLAY (OR SILTY CLAY), POORLY GRADED GRAVEL WITH CLAY AND (OR SILTY CLAY AND SAND)	
		GRAVELS WITH 5% TO 12% FINES		GM	SILTY GRAVEL, SILTY GRAVEL WITH SAND
				GC	CLAYEY GRAVEL, CLAYEY GRAVEL WITH SAND
			GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL WITH SAND	
	SANDS (50% or more of coarse fraction passes the No. 4 Sieve)	CLEAN SANDS WITH <5% FINES		SW	WELL-GRADED SAND, WELL-GRADED SAND WITH GRAVEL
				SP	POORLY GRADED SAND, POORLY GRADED SAND WITH GRAVEL
		SANDS WITH 5% TO 12% FINES		SW-SM	WELL-GRADED SAND WITH SILT, WELL-GRADED SAND WITH SILT AND GRAVEL
				SW-SC	WELL-GRADED SAND WITH CLAY (OR SILTY CLAY), WELL-GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY AND GRAVEL)
				SP-SM	POORLY GRADED SAND WITH SILT, POORLY GRADED SAND WITH SILT AND GRAVEL
				SP-SC	POORLY GRADED SAND WITH CLAY, POORLY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY AND GRAVEL)
		SANDS WITH > 12% FINES		SM	SILTY SAND, SILTY SAND WITH GRAVEL
				SC	CLAYEY SAND, CLAYEY SAND WITH GRAVEL
				SC-SM	SILTY, CLAYEY SAND, SILTY, CLAYEY SAND WITH GRAVEL

FINE GRAINED SOILS (50% or more passes the No. #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	SILT, SILT WITH SAND, SILT WITH GRAVEL
			CL	LEAN CLAY, LEAN CLAY WITH SAND, LEAN CLAY WITH GRAVEL
			CL-ML	SILTY CLAY, SILTY CLAY WITH SAND, SILTY CLAY WITH GRAVEL
	SILTS AND CLAYS (Liquid Limit 50 or greater)		OL	ORGANIC CLAY, ORGANIC CLAY WITH SAND, ORGANIC CLAY WITH GRAVEL, ORGANIC SILT, ORGANIC SILT WITH SAND, ORGANIC SILT WITH GRAVEL
			MH	ELASTIC SILT. ELASTIC SILT WITH SAND, ELASTIC SILT WITH GRAVEL
			CH	FAT CLAY, FAT CLAY WITH SAND, FAT CLAY WITH GRAVEL
			OH	ORGANIC CLAY, ORGANIC CLAY WITH SAND, ORGANIC CLAY WITH GRAVEL, ORGANIC SILT, ORGANIC SILT WITH SAND, ORGANIC SILT WITH GRAVEL

	PROJECT NO.: 20236170.006A	GRAPHICS KEY	FIGURE A-1
	DRAWN BY: D.E CHECKED BY: J.Z DATE:		
		Proposed Anthony Munoz Community Center Gymnasium 1242 W 4th Street Ontario, California	

GRAIN SIZE¹

DESCRIPTION		SIEVE SIZE	GRAIN SIZE
Boulders		>12 in.	>12 in. (304.8 mm.)
Cobbles		3 - 12 in.	3 - 12 in. (76.2 - 304.8 mm.)
Gravel	coarse	3/4 - 3 in.	3/4 - 3 in. (19 - 76.2 mm.)
	fine	#4 - 3/4 in.	0.19 - 0.75 in. (4.8 - 19 mm.)
Sand	coarse	#10 - #4	0.075 - 0.19 in. (2 - 4.9 mm.)
	medium	#40 - #10	0.017 - 0.075 in. (0.43 - 2 mm.)
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)
Fines		Passing #200	<0.0029 in. (<0.07 mm.)

SECONDARY CONSTITUENT¹

Term of Use	AMOUNT	
	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

PLASTICITY¹

DESCRIPTION	CRITERIA
Non-Plastic	A 1/8 in. (3 mm) 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. The thread 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.

MOISTURE CONTENT¹

DESCRIPTION	FIELD TEST
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

CONSISTENCY - FINE-GRAINED SOIL^{2,3}

CONSISTENCY	SPT - N (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q _u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Easily penetrated several inches by fist
Soft	2 - 4	0.25 ≤ PP < 0.5	500 - 1,000	Easily penetrated several inches by thumb
Medium Stiff	4 - 8	0.5 ≤ PP < 1	1,000 - 2,000	Can be penetrated several inches by thumb with moderate effort
Stiff	8 - 15	1 ≤ PP < 2	2,000 - 4,000	Readily indented by thumb but penetrated only with great effort
Very Stiff	15 - 30	2 ≤ PP < 4	4,000 - 8,000	Readily indented by thumbnail
Hard	>30	4 ≤ PP	>8,000	Indented by thumbnail with difficulty

APPARENT DENSITY - COARSE-GRAINED SOIL²

APPARENT DENSITY	SPT-N (# blows / ft)
Very Loose	<4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	>50

STRUCTURE¹

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. (6mm) thick, note thickness.
Laminated	Alternating layers of varying material or color with the layers less than 1/4-in. (6 mm) thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.
Homogeneous	Same color and appearance throughout

ANGULARITY¹

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

REACTION WITH HYDROCHLORIC ACID¹

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

CEMENTATION¹

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or little finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

REFERENCES

- American Society for Materials and Testing (ASTM), 2017, ASTM D2488: Standard Practice for Description and Identification of Soils (Visual Manual Procedures).
- Terzaghi, K and Peck, R., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons, New York.
- United States Department of the Interior Bureau of Reclamation (USBR), 1998, Earth Manual, Part I.



PROJECT NO.:
20236170.006A

DRAWN BY: D.E

CHECKED BY: J.Z

DATE:

SOIL DESCRIPTION KEY (For additional tables, see ASTM D2488)

Proposed Anthony Munoz
Community Center Gymnasium
1242 W 4th Street
Ontario, California

FIGURE

A-2

Date Begin - End: 9/22/2023		Drilling Company: 2R Drilling		BORING LOG B-1													
Logged By: D. Edrees		Drill Crew: Eddie/ Matt															
Hor.-Vert. Datum: Not Available		Drilling Equipment: GT-16		Hammer Type - Drop: 140 lb. Auto - 30 in.													
Plunge: -90 degrees		Drilling Method: Hollow Stem Auger															
Weather: Sunny		Bore Diameter: 8 in. O.D.															
Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS									
			Latitude: 34.07873° Longitude: -117.67447° Approximate Ground Surface Elevation (ft.): 1,090 Surface Condition: Grass		Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks	
			Lithologic Description														
			TOPSOIL: approximately 2"		1											Hand Auger 5' bgs	
			Artificial Fill													Modified Proctor	
			Silty SAND (SM): fine sand, yellowish brown, moist, trace gravel		2												
			Alluvium														
			Silty SAND (SM): fine sand, yellowish brown, moist, trace gravel														
1085	5		fine to medium sand, light orangish brown, moist, loose, increase in sand content		3		BC=3 5 8	18"									
1080	10		dense, cobbles up to 6"				BC=22 22 20	NR									
1075	15		olive yellow, moist, loose		4		BC=5 6 8	18"		9.6	114.0						
1070	20		fine sand, olive, increase in fines		5		BC=3 4 5	18"									
1065	25		Silty SAND with Gravel (SM): fine sand, olive, moist, medium dense, fine gravel		6		BC=4 6 5	16"					43			Passing #200	
1060	30		fine to medium sand, light olive, increase in sand content, subrounded gravel		7		BC=12 15 9	12"					13			Passing #200	
			Silty SAND (SM): fine to medium sand, orangish brown, moist, loose														
			PROJECT NO.: 20236170.006A		BORING LOG B-1										FIGURE		
			DRAWN BY: D.E		Proposed Anthony Munoz Community Center Gymnasium 1242 W 4th Street Ontario, California										A-1		
			CHECKED BY: J.Z														
			DATE:												PAGE: 1 of 2		



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Date Begin - End:	9/22/2023	Drilling Company:	2R Drilling	BORING LOG B-2		
Logged By:	D. Edrees	Drill Crew:	Eddie/ Matt			
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	GT-16		Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger			
Weather:	Sunny	Bore Diameter:	8 in. O.D.			

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS								
			Latitude: 34.07874° Longitude: -117.67467° Approximate Ground Surface Elevation (ft.): 1,090 Surface Condition: Grass	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks	
			Lithologic Description													
1085	5		TOPSOIL: approximately 3"	1											Hand Auger 5' bgs	
			Artificial Fill													
			Silty SAND (SM): fine sand, dark brown, moist, trace medium sand	2												
			Alluvium													
			Silty SAND (SM): fine sand, dark brown, moist, trace medium sand													
			Silty SAND with Gravel (SM): fine to medium sand, light brown, moist, medium dense, broken cobbles, angular gravel up to 1.5"	3		BC=10 11 9	18"									
			Well-Graded GRAVEL with Sand (GW): fine to coarse gravel, gayish olive, moist, medium dense, fine to coarse sand, trace silt	4		BC=15 11 7	18"		3.0	107.7	38	4.8				Sieve Analysis
			Poorly Graded SAND with Silt (SP-SM): fine to medium sand, grayish olive, moist, very dense	5		BC=36 29 24	18"									
			Silty SAND (SM): fine sand, olive, moist, loose	6		BC=6 3 4	18"									
			fine to medium sand, increase in fines	7		BC=3 3 4	18"									

The boring was terminated at approximately 26.5 ft. below ground surface. The boring was backfilled with auger cuttings and sand on September 22, 2023.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by the preliminary topographic survey plan obtained from Robert R. Coffee Architect & Associates, dated 02/12/2019.



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GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2023.GLB [KLF_BORING/TEST/PIT/Soil LOG]



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APPENDIX B

LABORATORY TESTING

APPENDIX B LABORATORY TESTING

GENERAL

Laboratory tests were performed on selected samples as an aid in classifying the soils and to evaluate physical properties of the soils that may affect foundation design and construction procedures. The tests were performed in general conformance with the current ASTM or Caltrans standards. A description of the laboratory-testing program is presented below.

MOISTURE AND UNIT WEIGHT

Moisture content and dry unit weight tests were performed on selected samples recovered from the borings. Moisture contents were determined in general accordance with ASTM Test Method D 2216; dry unit weight was calculated using the entire weight of the samples collected in general accordance with ASTM Test Method D7263. Results of these tests are presented on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Sieve analysis of selected samples was performed to evaluate the grain size distribution of the onsite soils and to aid in classification of the soils. Sieve analysis was performed in accordance with ASTM Standard Test Method D6913. The result of this test is presented in the boring logs in Appendix A and attached as Figure B-1, Grain Size Distribution.

CONSOLIDATION TESTS

Consolidation testing was performed by AP Engineering on a relatively undisturbed sample in accordance with ASTM Standard Test Method D2435. The test result is presented on Figure B-2, Consolidation Test.

DIRECT SHEAR

Direct shear testing was performed by AP Engineering on a remolded sample for shear strength and cohesion values of the in-situ soils in accordance with ASTM Standard Test Method D 3080. The sample was remolded to 90 percent of the Modified Proctor Maximum Density at the approximate optimum moisture content. The results are presented as Figure B-3, Direct Shear Test.

ONE-DIMENSIONAL SWELL/COLLAPSE TEST

Laboratory testing was performed on a selected soil sample to study the collapse potential of the subgrade soils. During this test, the soil sample is inundated with water at a specific surcharge loading and the percent swell or collapse is measured. This test was performed by AP Engineering in accordance with ASTM D4546. The test result is presented as Figure B-4, 1-D Swell/Collapse.

MODIFIED PROCTOR

Modified Proctor (maximum density and optimum moisture content testing) was performed on bulk samples of the near surface soils to evaluate the compaction characteristics of the onsite soils. Maximum density and optimum moisture content testing was performed in general accordance with ASTM Standard Test Method D1557. The test results are presented in Table B-1, Modified Proctor Test Results.

R-VALUE

Resistance value (R-value) tests was performed on a selected bulk soil sample obtained to evaluate pavement support characteristics of the near-surface onsite soils. R-value tests were performed in accordance with ASTM Standard Test Method D2844. The test result is presented in Table B-2, R-Value Test Result.

PRELIMINARY SOIL CORROSIVITY

A series of chemical tests were performed on a selected sample of the near-surface soils to estimate pH, resistivity and sulfate and chloride contents. The sample was tested in general accordance with California Test Methods 643, 422, and 417 for pH and minimum resistivity, soluble chlorides, and soluble sulfates, respectively. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The tests were performed by AP Engineering and Laboratory, Inc. The results are presented in Table B-3, Preliminary Corrosivity Test Results.

Table B-1
Modified Proctor Test Results

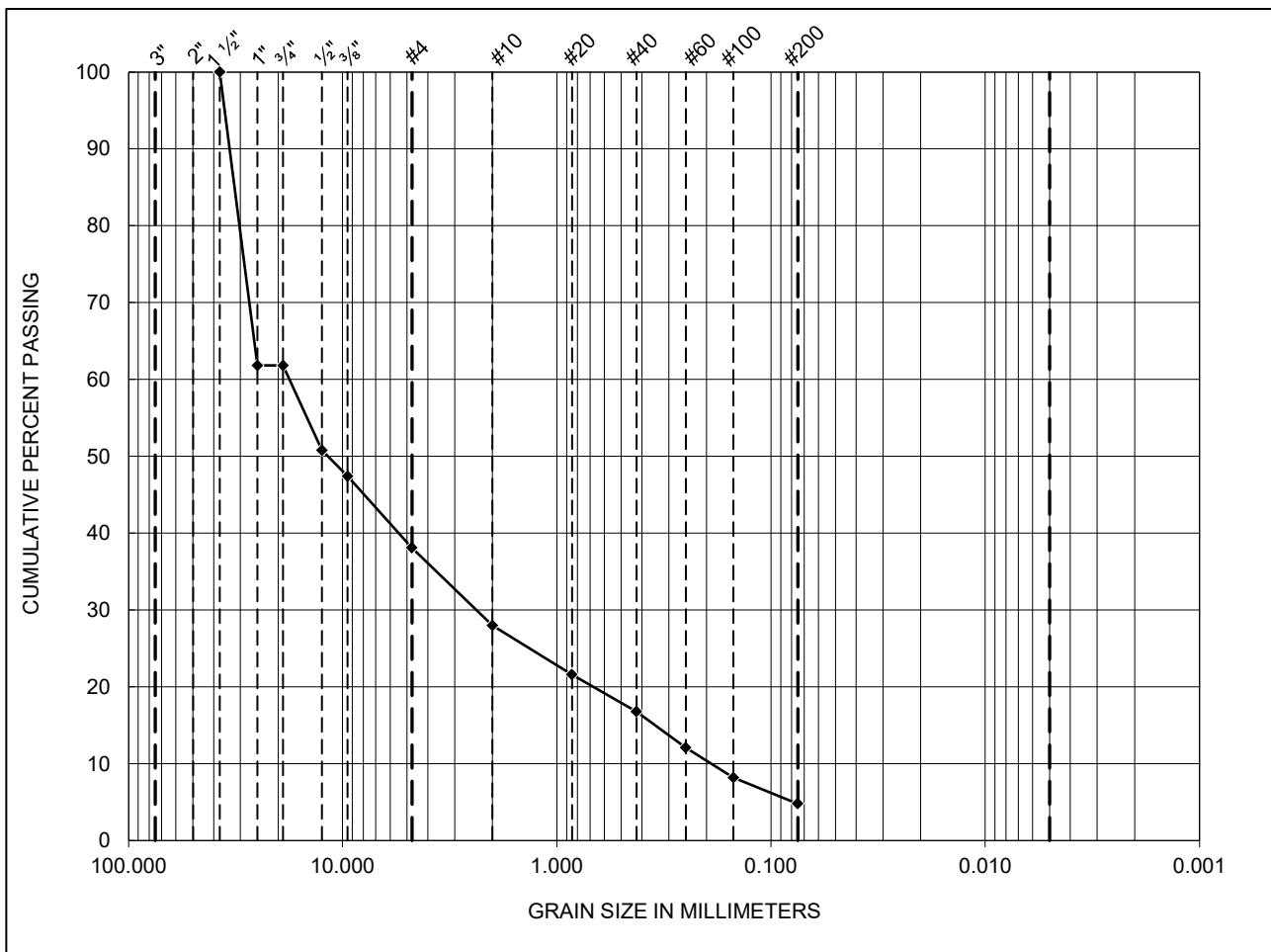
Boring Number	Depth (ft)	Maximum Density (pcf)	Optimum Moisture (%)
B – 1	0 – 5	135.2 (corrected)	7.5 (corrected)

Table B-2
R-Value Test Result

Boring Number	Depth (ft)	R-Value
B-3	0 – 5	81

Table B-3
Preliminary Corrosivity Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
B – 1	0 – 5	7.6	21	29	9,685



COBBLE	GRAVEL	SAND	SILT	CLAY
--------	--------	------	------	------

SYMBOL	SAMPLE IDENTIFICATION			PERCENTAGES			ATTERBERG LIMITS			SOIL CLASSIFICATION
	BORING NO.	SAMPLE NO.	DEPTH (ft.)	GRAVEL	SAND	FINES	LL	PL	PI	
◆	B-2	4	10	61.9	33.3	4.8				Well Graded Gravel w/Sand (GW)



PROJECT NO.: 20236170.006A
 TESTED BY: J. Calderon
 DATE: 9/27/2023
 CHECKED BY: M/Magaña
 DATE: 9/29/2023

GRAIN SIZE DISTRIBUTION

Proposed Anthony Munoz
 Community Center Gymnasium
 1242 W 4th Street
 Ontario, California

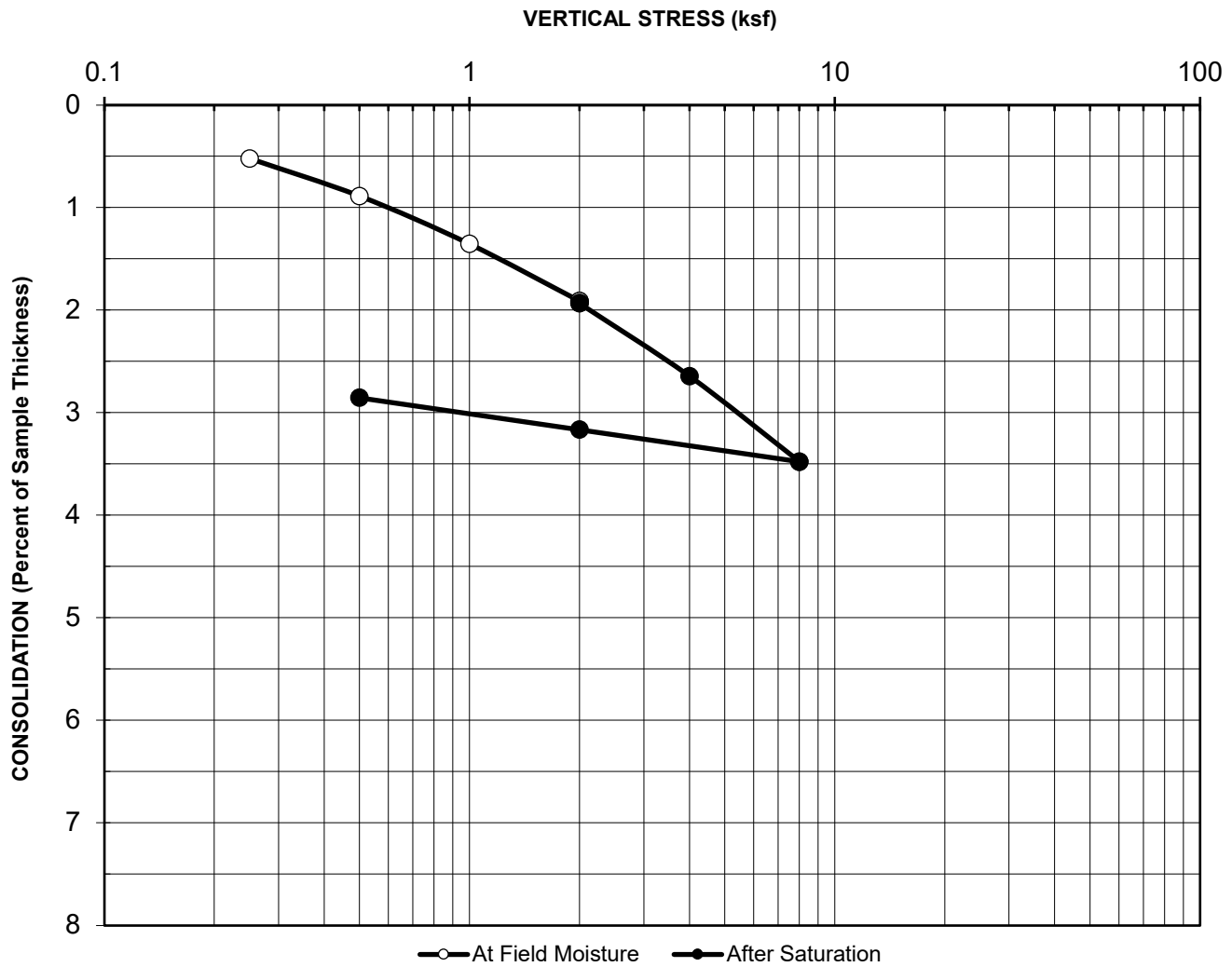
FIGURE

B-1

**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.comBoring No. : B-1Initial Dry Unit Weight (pcf): 124.5Sample No.: 3Initial Moisture Content (%): 10.0Depth (feet): 5Final Moisture Content (%): 11.5Sample Type: Mod CalAssumed Specific Gravity: 2.7Soil Description: Silty Sand w/gravelInitial Void Ratio: 0.354Remarks: Collapse= 0.02% upon inundation**CONSOLIDATION CURVE
ASTM D 2435**Project Name: Anthony Munoz ParkProject No.: 20236170.006ADate: 9/27/2023AP No: 23-0960Sheet No: 1**FIGURE
B-2**

**AP Engineering and Testing, Inc.**

DBE|MBE|SBE

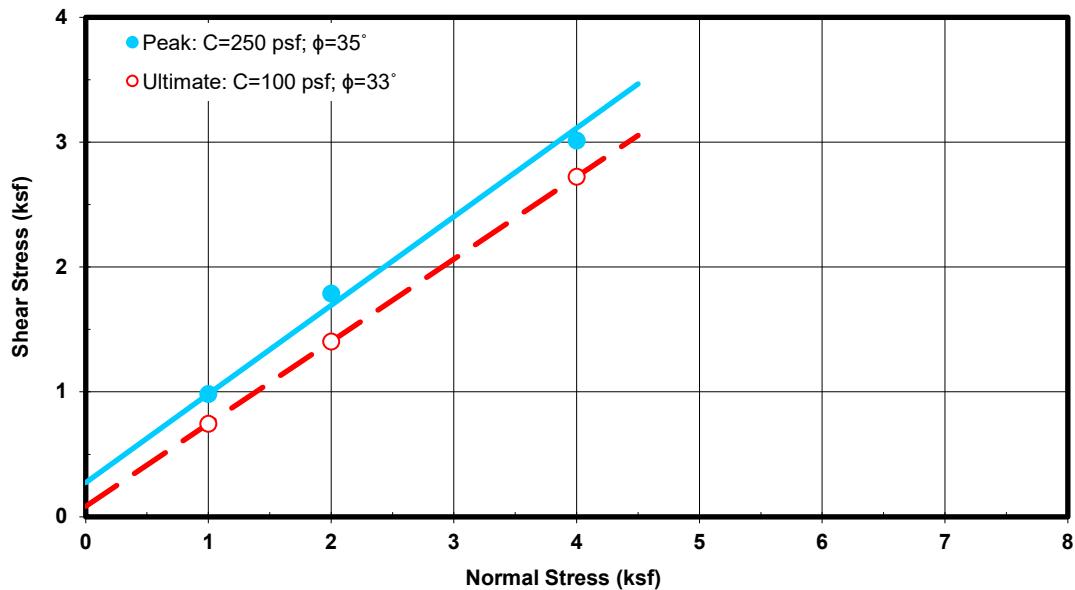
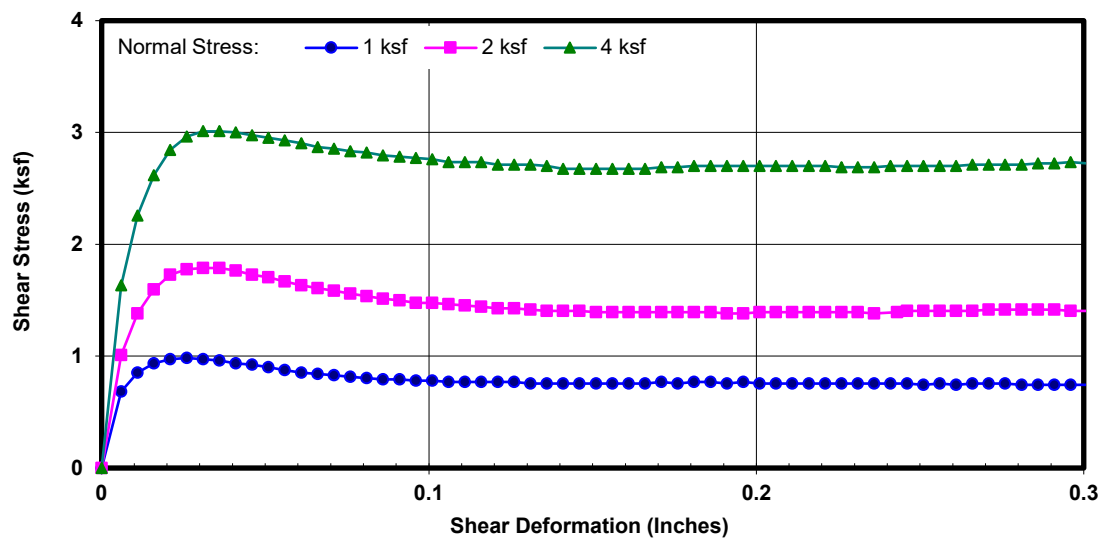
2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com**DIRECT SHEAR TEST RESULTS**
ASTM D 3080

Project Name: Anthony Munoz Park
Project No.: 20236170.006A
Boring No.: B-1
Sample No.: 1 Depth (ft): 0-5
Remold Cond.: Remolded to 90% of 133.2 pcf at 8% MC
Soil Description: Silty Sand
Test Condition: Inundated Shear Type: Regular

Tested By: KM Date: 10/02/23
Computed By: KM Date: 10/03/23
Checked by: AP Date: 10/03/23

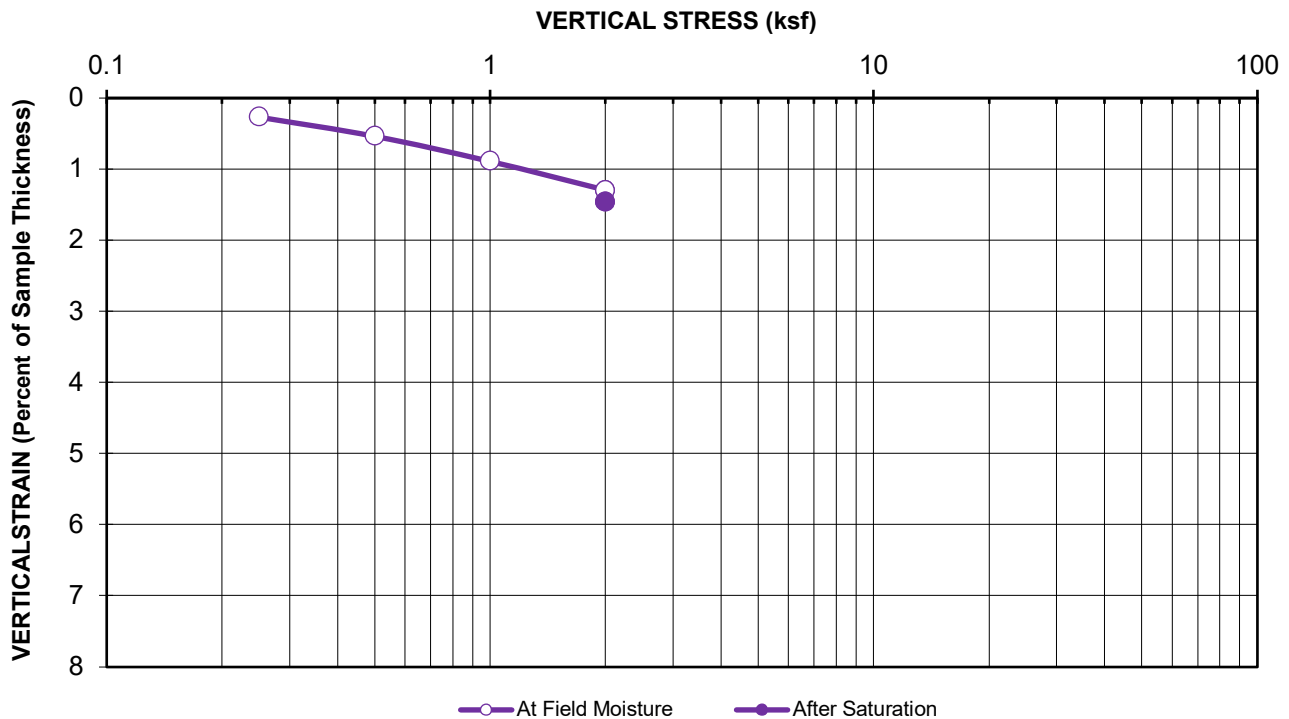
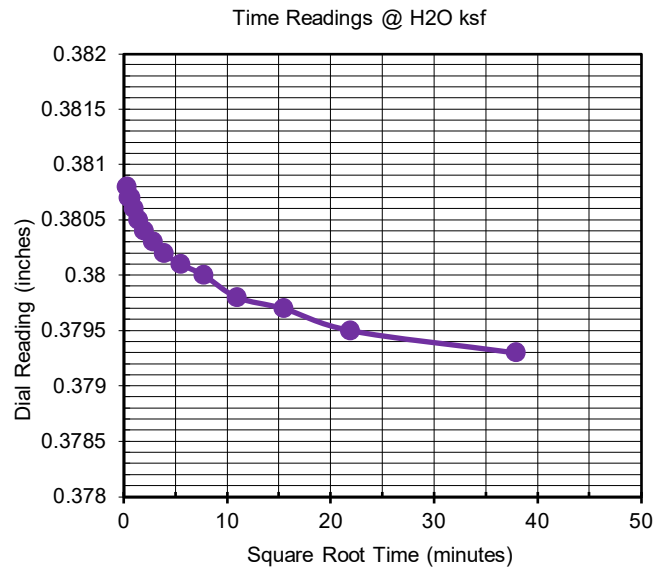
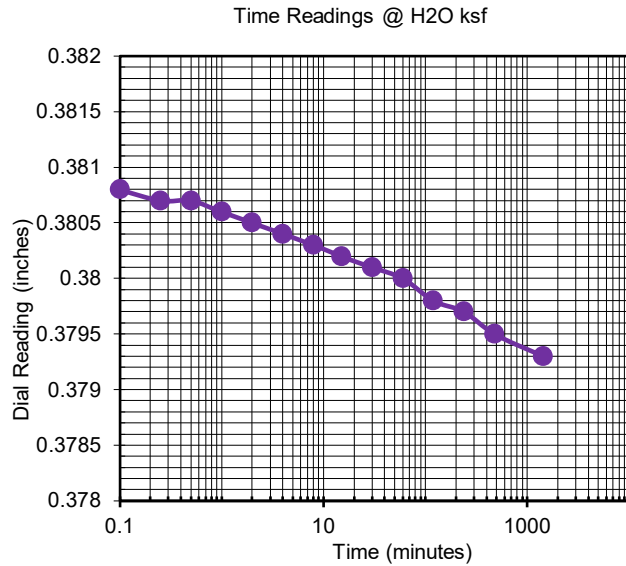
Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.1	119.8	7.8	14.7	52	97	1	0.984	0.744
						2	1.788	1.404
						4	3.012	2.724

**FIGURE**
B-3

**AP Engineering and Testing, Inc.**

DBE|MBE|SBE

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Boring No. : B-2
Sample No.: 3
Depth (feet): 5
Sample Type: Mod Cal
Soil Description: Poorly-Graded Sand w/silt & gravel
Remarks: Collapse = 0.16% upon inundation

Initial Dry Unit Weight (pcf): 117.3
Initial Moisture Content (%): 8.8
Final Moisture Content (%): 13.0
Initial Void Ratio: 0.44

1-D SWELL/COLLAPSE
ASTM D 4546-14, Method B

Project Name: Anthony Munoz Park
Project No.: 20236170.006A
Date: 9/29/23
AP No: 23-0960

FIGURE
B-4