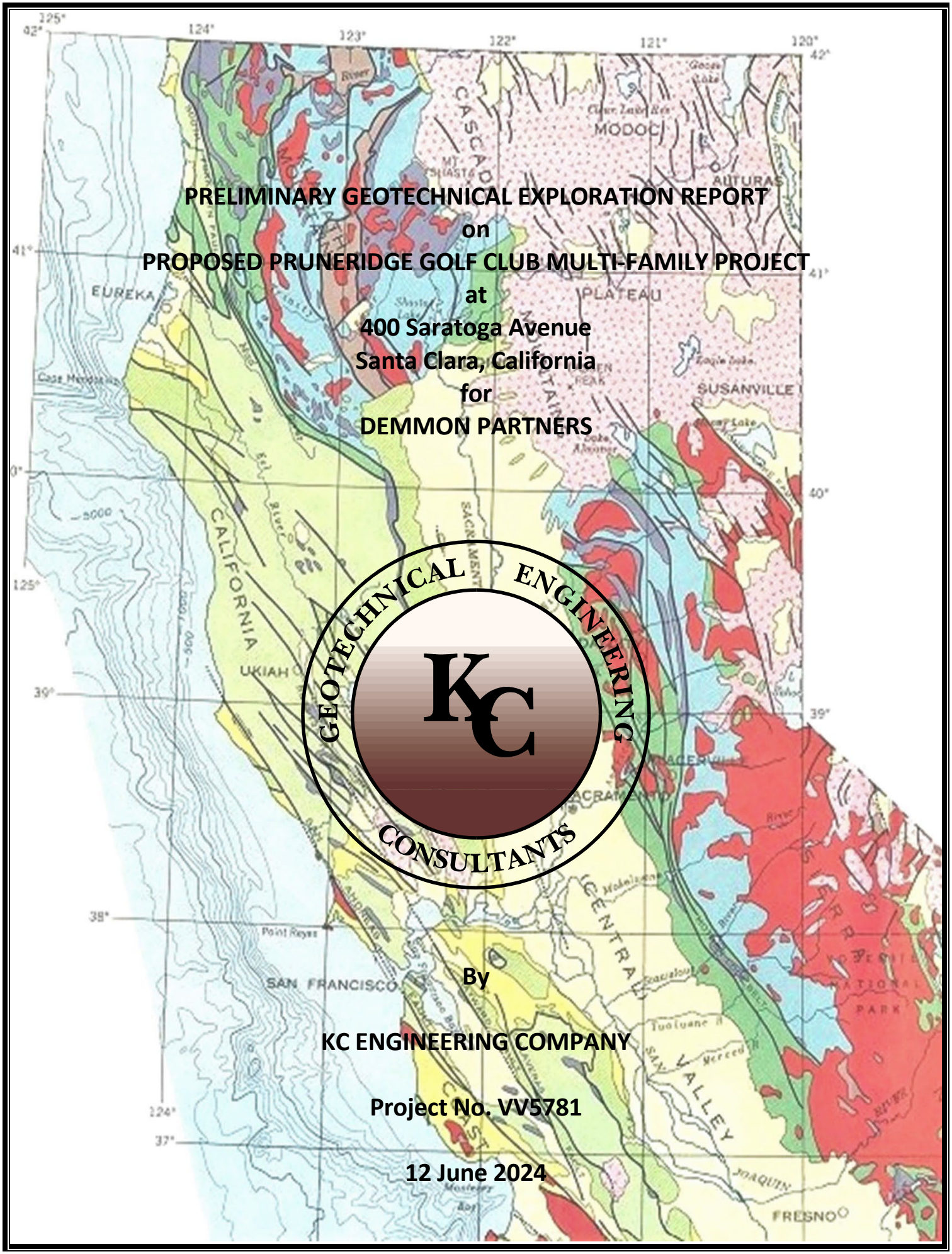

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT
on
PROPOSED PRUNERIDGE GOLF CLUB MULTI-FAMILY PROJECT
at
400 Saratoga Avenue
Santa Clara, California
for
DEMMON PARTNERS

GEOTECHNICAL ENGINEERING CONSULTANTS

KC

By

KC ENGINEERING COMPANY

Project No. VV5781

12 June 2024

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A SUBSIDIARY OF MATERIALS TESTING, INC.

Project No. VV5781
12 June 2024

Mr. Mitchell McKinzie
Demmon Partners
601 University Avenue, Suite 110
Sacramento, CA 95825

Subject: Proposed Pruneridge Golf Club Multi-Family Project
400 Saratoga Avenue
Santa Clara, California
PRELIMINARY GEOTECHNICAL EXPLORATION REPORT

Dear Mr. McKinzie:

In accordance with your authorization, **KC ENGINEERING COMPANY** has explored the geotechnical conditions of the surface and subsurface soils for the proposed Pruneridge Golf Club Multi-Family project to be constructed at the subject site.

The accompanying report presents our preliminary conclusions and recommendations based on our limited exploration. Our findings indicate that the proposed Pruneridge Golf Club Multi-Family project and associated facility improvements are geotechnically feasible for construction on the subject site provided the recommendations of this report are carefully followed and are incorporated into the project plans and specifications. We should note that additional subsurface exploration, lab testing and analysis will be required including updated recommendations.

Should you have any questions relating to the contents of this report or should you require additional information, please contact our office at your convenience.

Respectfully Submitted,
KC ENGINEERING COMPANY

David V. Cymanski, G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
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Reviewed by,

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TABLE OF CONTENTS

Page No.

LETTER OF TRANSMITTAL

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT 4
- Purpose and Scope 4
- Site Location and Description 4
- Proposed Construction 5
- Field Exploration 5
- Laboratory Testing 6
- Subsurface Conditions 7
- Soil Corrosivity 7
- Site Geology 8
- Geo-Hazards 8
- Settlement Considerations 11

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS 13
- General 13
- Geotechnical Considerations 13
- Demolition 15
- Groundwater Control & Dewatering 15
- Temporary Excavations 16
- Temporary Shoring 17
- Grading 18
- Surface & Subsurface Drainage 20
- Foundations 22
- Slab-on-Grade Construction 24
- Retaining Walls 26
- Pavement Areas 27
- Underground Utility and Excavations 28

LIMITATIONS AND UNIFORMITY OF CONDITIONS 30

APPENDIX 31
- Aerial Vicinity Map, Figure 1
- Site Plan, Figure 2
- Geologic Map, Figure 3
- Log of Test Borings, Figures 4 - 6
- Subsurface Exploration Legend
- Laboratory Test Results
- ASCE 7 Seismic Hazards Report

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT

Purpose and Scope

The purpose of the geotechnical exploration for the proposed Pruneridge Golf Club multi-family project to be constructed at 400 Saratoga Avenue in Santa Clara, California was to determine the surface and subsurface soil conditions at the subject site. Based on the results of the limited exploration, preliminary geotechnical criteria were established for the grading of the site, the design of foundations, slabs, retaining walls, pavements, drainage and the construction of other related facilities on the property.

In accordance with your authorization, our exploration services included the following tasks:

a. A review of available geotechnical and geologic literature concerning the site and vicinity;
b. Site reconnaissance by the Geotechnical Engineer to observe and map surface conditions;
c. Drilling and logging of three exploratory borings and sampling of the subsurface soils;
d. Laboratory testing of the samples obtained to determine their classification and engineering characteristics;
e. Analysis of the data and formulation of conclusions and recommendations; and
f. Preparation of this written report.

Site Location and Description

The subject property is located within the Pruneridge Golf Club property at 400 Saratoga Avenue in Santa Clara, California as shown on Figure 1, “Aerial Vicinity Map” included in the Appendix of this report. The golf club is bounded on the north by existing multi-family developments, on the southeast by Saratoga Avenue, on the south by Pruneridge Avenue, and on the west by San Tomas Expressway. The proposed multi-family project is planned on the southern and southwestern portion of the golf club, including Holes 1 and 2, a portion of Hole 3 and the existing maintenance yard as indicated on Figure 1. The site relatively flat in topography except for the gently rolling terrain of the fairways. This area is landscaped with grass and a number of mature trees. Two office and equipment buildings are present within the maintenance yard located west of the driving range.

The above description is based on a reconnaissance of the site by the Geotechnical Engineer, a review of a Google Earth aerial images and a Dimensioned Site Plan by Kenneth Rodrigues & Partners dated 3/13/24 showing the proposed multi-family development layout. A Google Earth

aerial image was used as the basis for our "Aerial Vicinity Map" included as Figure 1 in the Appendix. The Dimensioned Site Plan was used as our "Site Plan" included as Figure 2 in the Appendix.

Proposed Construction

Based on our review the architectural site plans, floor plans and building elevations, the proposed project is to include two multi-family buildings, a maintenance equipment storage building, a driveway and surface parking areas. The multi-family Building 1 on the southwest is planned to be five stories above grade with a central courtyard and swimming pool supported on a concrete podium deck with one or two levels of below grade parking garages. The multi-family Building 2 on the southwest is planned to be four stories above grade with a central courtyard supported on a concrete podium deck with one level of below grade parking garage. The multi-family buildings are expected to be wood-framed possibly with structural steel moment frames. A one-story 6,000 square feet equipment storage building is planned north of Building 1. The equipment building will either consist of wood-framing or a pre-engineered metal building with an interior slab-on-grade floor. Additional site improvements are expected to consist of installing underground utilities, golf course modifications, and landscaping.

Field Exploration

Our preliminary field exploration was performed on 4/29/24 and included a reconnaissance of site and the drilling of three preliminary exploratory test borings at the approximate locations shown on Figure 2. Additional exploratory borings and cone penetration tests (CPT) will be required to adequately characterize the subsurface conditions. Due to active golf course operations and access limitations, were not able to be perform the additional borings and CPTs at this time.

The borings were drilled to a maximum depth of 45 feet below the existing ground surface with sampling extending to 46 to 48 feet. The drilling was performed with a CME 55 drill rig using a power-driven, 8-inch diameter hollow stem augers. Visual classifications per ASTM D2488 were made from the auger cuttings and the samples recovered in the field. As the drilling proceeded, representative disturbed tube samples were obtained by driving a 3-inch O.D., California Modified split-tube sampler, containing thin brass liners, into the boring bottom in accordance with ASTM D3550. Disturbed samples were obtained by driving a 2-inch O.D., split-barrel SPT sampler into the boring bottom in accordance with ASTM D1586. The sampler was driven into the in-situ soils under the impact of a 140-pound hammer having a free fall of 30 inches. The number of blows required to advance the sampler 12 inches into the soil were adjusted to the standard penetration resistance (N-Value). The raw blow counts obtained using the California Modified sampler were corrected to equivalent N-Values using the rigs calibrated automatic trip hammer with an 80% energy

transfer ratio. When the sampler was withdrawn from the boring bottom, the liners containing the relatively undisturbed samples were removed, examined for identification purposes, labeled and sealed to preserve the natural or in-situ moisture content.

The samples were then transported to our laboratory for testing per ASTM D4220. Classifications made in the field were verified in the laboratory after further examination and testing. The stratification of the soils, descriptions, location of undisturbed soil samples and standard penetration resistance are shown on the respective "Log of Test Boring" contained within the Appendix.

Laboratory Testing

The laboratory testing program was directed towards providing sufficient information for the estimation of the engineering characteristics of the site soils so that the recommendations outlined in this report could be formulated. The laboratory test results are presented in the Appendix.

Moisture content and dry density tests (ASTM D2937) were performed on representative relatively undisturbed soil samples to determine the consistency of the soil and the moisture variation throughout the explored soil profile as well as estimate the compressibility of the underlying soils.

In order to assist in the identification and classification of the subsurface soils, sieve analysis tests (ASTM D6913) and Atterberg Limits test (ASTM D4318) were performed on selected soil samples. The Atterberg Limits test results were used to estimate the expansion potential of the near surface soils. The strength of the subsurface soils were evaluated by unconfined compression tests (ASTM D2166) and direct shear tests (ASTM D3080) performed on relatively undisturbed samples. To evaluate potential settlement due to estimated building loads, two consolidation tests were performed per ASTM D2435.

An R-Value test (Cal Test 301) was performed on a composite bulk sample representative of the proposed subgrade to assist in pavement section design. The bulk sample was obtained from the upper 3 feet across the site pavement areas.

A composite bulk sample of the upper 20 feet was obtained from the boring cuttings and tested to evaluate the presence and concentration of water-soluble sulfates in accordance with ASTM C1580. These test results were used to identify the corrosion potential of the soils to at or below grade concrete. Additional corrosivity indicator tests were performed including soil pH, minimum resistivity and chlorides.

Subsurface Conditions

Based on our findings from the field and laboratory results, the subsurface soil conditions across the project site consist of variably stratified alluvial fan deposits. At Boring 1, the upper 8 feet consists of highly expansive stiff sandy clay, underlain by very stiff sandy clay to 20 feet, underlain by medium dense clayey sand and dense gravelly sand to 35 feet, underlain by stiff sandy clay to 40 feet, underlain b ymedium dense sand with silt and gravel to 45 feet, further underlain by hard to very stiff sandy clay to the maximum depth explored of 48 feet. At Boring 2, the upper 10 feet consists of highly expansive very stiff sandy clay, underlain by very stiff sandy clay to 20 feet, further underlain by stiff to firm sandy clay and clay layers to a depth of 46 feet. At Boring 3, the upper 18 feet consists of highly expansive stiff to hard sandy clay layers, underlain by a medium dense to dense layer of sandy gravel to 23 feet, underlain by stiff sandy clay to 30 feet, underlain by loose to medium dense clayey sand layers to 40 feet, further underlain by firm to stiff sandy clay and clay to 46.5 feet.

Groundwater was initially encountered at depths ranging from 21 to 30 feet below the surface and at the time of our exploration. The water levels rose to depth of 13 to 25 feet at the completion of drilling prior to tremie grouting. Fluctuations in the groundwater level can occur with variations in seasonal rainfall, subsurface stratification, and irrigation on the site and vicinity.

A more thorough description and stratification of the soils encountered along with the results of the laboratory tests are presented on the respective “Log of Test Boring” in the Appendix. The approximate locations of the borings are shown on Figure 2.

Soil Corrosivity

A representative composite sample of the upper 20 feet from the three borings was collected and transported to Sunland Analytical in Rancho Cordova for testing of water-soluble sulfates, pH, minimum resistivity and chlorides per ASTM and California Test Methods.

The testing indicates a sulfate content of 177.1 ppm (mg/kg), a chloride content of 103.8 ppm, a minimum resistivity of 510 ohm-cm, and a soil pH value of 7.3 for the sample collected. It is noted that the sulfate test results indicate low or “S0” sulfate exposure to concrete as identified in the Durability Requirements, Section 1904 of the 2022 California Building Code, and Tables 19.3.1.1 of ACI 318-19 Building Code Requirements for Structural Concrete. Therefore, no cement type or minimum concrete strength requirements are applicable.

The Caltrans Corrosion Guidelines[1] defines a corrosive site as one where the soil and/or water has a sulfate concentration of 1,500 ppm or more, a chloride concentration of 500 ppm or more, a pH of 5.5 or less, and a minimum resistivity less than 1,500 ohm-cm. Based on these criteria, the low resistivity value indicates the presence of high quantities of soluble salts and a higher propensity for corrosion.

KC ENGINEERING CO. is not a corrosion engineering firm. Therefore, to further define the soil corrosion potential and interpret the above test results, or to design cathodic protection or grounding systems, a licensed Corrosion Engineer should be consulted.

Site Geology

According to Geologic Map of the Cupertino and San Jose West Quadrangles[2], the site is mapped within Holocene age alluvial fan deposits. These deposits are described as alluvial sand, fine-grained, silt and clay; where differentiated represents distal alluvial fan deposits at outer edge of fan areas. The materials encountered in our borings correlate with published geologic mapping. A geologic map showing the site and surrounding area is included as Figure 3, "Geologic Map".

Geo-Hazards

The subject site is located within a Seismic Hazard Zone for liquefaction potential as shown on the map San Jose West Quadrangle map by the CGS[3]. However, the site is not shown on the CGS map to be located within an Alquist-Priolo Earthquake Fault Zone or landslide hazard area. As noted in Seismic Hazard Zone Report 058 by the CGS, dated 2002, evaluation and mitigation of seismic hazards are to be conducted under the "Guidelines for Evaluating and Mitigating Seismic Hazards in California", Special Publication 117A, dated 2008. The following comments and subsections address these issues.

The project site is located in a seismically-active region and earthquake related ground shaking should be expected during the design life of structures constructed on the site. The project will be submitted to the City for design review per the 2022 California Building Code. Therefore, appropriate 2022 CBC Earthquake Loads per Section 1613 must be applied by the project Structural Engineer for the proposed structures.

[1] California Department of Transportation, Division of Engineering Services, Materials Engineering and Testing Services Corrosion Branch, *Corrosion Guidelines*, Version 3.2, May 2021.
[2] Dibblee, T. W., 2007, *Geologic Map of the Cupertino and San Jose West Quadrangles, Santa Clara and Santa Cruz Counties, California.*
[3] Davis, J.F., 2/7/02, *Earthquake Zones of Require Investigation San Jose West Quadrangle,* California Geological Survey.

Seismicity & Ground Motion Analysis

The site is not located within an Alquist-Priolo Earthquake Fault Zone[4]. There are no known active faults crossing the site as mapped and/or recognized by the State of California. However, Santa Clara is located in a seismically active region and earthquake related ground shaking should be expected during the design life of structures constructed on the site. The California Geological Survey has defined an active fault as one that has had surface displacement in the last 11,700 years or has experienced earthquakes in recorded history.

Based on our review of the Fault Activity Map of California[5] and the USGS National Seismic Hazard Maps-Source Parameters[6], the nearest major active faults are the Monte Vista-Shanon Fault, the N. San Andreas Fault, and the Hayward Fault located approximately 4.7 miles southwest, 8.7 miles southwest, and 10.6 miles northeast of the site, respectively. Numerous other active faults in the Bay Area may also produce significant seismic shaking at the site.

The 2022 CBC specifies that the potential for liquefaction and soil strength loss should be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with an adjustment for site class effects in accordance with American Society of Civil Engineer (ASCE 7-16)[7]. The MCE_G is peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. Based on ASCE 7-16, the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated to be 0.649g for the property using ASCE 7 Hazards seismic design tool web-based with a site coefficient (F_{PGA}) of 1.1 for Site Class D. Structures at the site should be designed to withstand the anticipated ground accelerations.

Based on the ASCE 7 Hazards Tool[8] website and ASCE 7-16, the 2022 CBC earthquake design values are as follows:

Site Class:	F*, D
Mapped Acceleration Parameters:	S_S = 1.5g; S_1 = 0.6g
Design Spectral Response Accelerations:	S_{DS} = 1.0g; S_{D1} = 1.02g

* A Site Class F is noted because liquefiable layers are present (ASCE 7-16, Section 20.3.1). However, it is our opinion that a stiff clay soil profile or Site Class D represents the average for the upper

[4] Parish, J.G., 2018 *Earthquake Fault Zones,* California Geological Survey, Special Publication 42, Revised 2018.
[5] Jennings, C.W. and Bryant, W.A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6, scale 1:750,000
[6] U.S. Geological Survey, 2008 National Seismic Hazards Maps – Source Parameters, accessed 6/1124, from USGS web site: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm
[7] American Society of Civil Engineer (ASCE), 2016, Minimum Design Loads for Buildings and Other Structures, Standard 7-16 and Supplements 1-3.
[8] https://asce7hazardtool.online, accessed 6/7/24

100 feet. In our opinion, a ground motion hazard analysis is not necessary per ASCE 7-16, Section 11.4.8-1. The MCE_R spectral response acceleration parameter SM_1 has been increased by 50 percent for the calculation of the design spectral response acceleration parameter SD_1. The modified ASCE seismic design report is included in the Appendix.

Fault Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on our review of geologic maps, no known active or inactive faults cross or project toward the subject site. Therefore, it is our opinion that there is no potential for fault-related surface rupture at the subject site.

Landsliding

The subject site and surrounding areas are essentially flat in elevation and therefore, not subject to seismically-induced landslide hazards.

Liquefaction

Soil liquefaction is a phenomenon in which loose and saturated cohesionless soils are subject to a temporary, but essentially total loss of shear strength, due to pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. Soils typically found most susceptible to liquefaction are saturated and loose, fine to medium grained sand having a uniform particle range and less than 35% fines passing the No. 200 sieve, and a corrected standard penetration blow count $(N_1)_{60}$ less than 30. According to Special Publication 117A by the California Geological Survey, the assessment of hazards associated with potential liquefaction of soil deposits at a site must consider translational site instability (i.e. lateral spreading, etc.) and more localized hazards such as bearing failure and settlement. The acceptable factor of safety against liquefaction is recommended in SP117 to be 1.3 or greater.

Based on our site exploration and laboratory test data, the soil material encountered beneath the site were found to be predominately cohesive sandy clays with a fines content ranging from 54 to 68 percent passing a No. 200 sieve. However, potential liquefiable layers were identified in Boring 1 between 20 to 28 and 40 to 45 feet, and in Boring 3 between 30 to 40 feet. These layers were evaluated for potential liquefaction.

A liquefaction analysis was performed for the layers in Boring 1 and 3 using the data from our field and lab exploration per the recommended analysis methods of the NCEER report[9] and Idriss and Boulanger (2008). The groundwater modeled in the analysis was 13 feet below the ground surface, based on our findings in Boring 1. A 10% exceedance in 50 years peak ground acceleration of o.55g was used for alluvium conditions as noted in the San Jose West Seismic Hazard Zone Report 058. A maximum magnitude of 7.2 was also used from the nearby San Andreas Fault. Based on our analysis, the layers in Boring 1 and 3 were found to have a factor of safety less than 1.3 indicating a potential for liquefaction.

Utilizing the volumetric strain relationship developed by Tokimatsu and Seed[10], total liquefaction settlement of 2.6 inches was determined for the layers in Boring 1, and 2.0 inches for the layers in Boring 3. Since liquefiable layers were not identified in Boring 2, differential settlement across the structure footprint for Building 1 may approach 2.6 inches and 2 inches for Building 2. According to Ishihara[11], the potential for surface manifestation (i.e. sand boils/ejecta, ground fissures, etc…) is unlikely considering the depth of the potentially liquefiable soil layers. Due to the lack of open slope faces, the potential for lateral spreading at the site is considered very low to nil.

Settlement Considerations

Our investigation of the site also included an evaluation of consolidation settlement of the firm to stiff subsurface sandy clay layers encountered in Boring 2 between 35 and 44 feet below grade, and for the loose to medium dense clayey sand layers in Boring 3 between 30 to 40 feet. In order to determine the compressibility and potential settlement of these soils, laboratory consolidation tests (ASTM D2435) were performed on relatively undisturbed soil samples. The results are presented in the Appendix. The samples in Boring 2 and 3 were found to be under-consolidated.

We performed a preliminary settlement analysis utilizing estimated structure loads considering the use of a thickened mat slab foundation system. We assumed floor loads of 150 psf and basement floor loads of 400 psf. Utilizing a uniform distributed bearing load of 2,250 psf for the basement mat slab foundation at Building 1, we determined total consolidation settlement of 5.5 inches at the corners and 7.25 inches at the center of ther mat. Utilizing a uniform distributed bearing load of 1,500 psf for the basement mat slab foundation at Building 2, we determined total consolidation settlement of 2.8 inches at the corners and 3 inches at the center of ther mat.

[9] Youd, T.L., et al., 2001 "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," in Journal of Geotechnical Geoenvironmental Engineering, October 2001

[10] Tokimatsu, K. and Seed, H.B., 1987, *Evaluation of Settlements in Sands Due to Earthquake Shaking*, Journal of the Geotechnical Engineering Division, ASCE, Volume 113, No. 8, August 1987.

[11] Ishihara, K., 1985, *Stability of Natural Deposits During Earthquakes,* Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, Volume 1, p. 321-376, August.

Once actual structure loads are determined by the Structural Engineer, additional analysis will be required.

The above consolidation settlement values exceed the typical acceptable total settlement value of 1 inch in 50 feet and 0.5 inch in 30 feet. As noted in the "Liquefaction" section above, total and differential liquefaction settlement of 2.6 inches was determined for the layers in Boring 1, and 2.0 inches for the layers in Boring 3.

In our opinion, the amount of anticipated total and differential settlement and/or angular distortion that may occur over the proposed building footprints from consolidation settlement and seismically induced liquefaction are considered excessive for a conventional mat slab foundation sytem. To mitigate these concerns and to minimize the anticipated differential settlement to acceptable levels, we recommend that an appropriate subsurface ground improvement technique such as drilled displacement columns or compacted aggregate piers be performed under the mat slabs. As an alternate to subsurface ground improvement, a conventional driven pile foundation system may be utilized.

It is also our opinion that additional exploratory borings, cone penetration tests, laboratory testing and settlement analysis be performed to adequately characterize the subsurface conditions.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical point of view, the proposed Pruneridge Golf Club Multi-Family project and associated facility improvements are considered to be feasible for construction on the subject site provided the recommendations presented in this report are incorporated into the project plans and specifications. We should note that additional subsurface exploration, lab testing and analysis will be required including updated recommendations.

All grading and foundation plans for the development must be reviewed by the Soil Engineer prior to contract bidding or submittal to governmental agencies to ensure that the geotechnical recommendations contained herein are properly incorporated and utilized in design.

KC ENGINEERING CO., should be notified at least two working days prior to site clearing, grading, and/or foundation operations on the property. This will give the Soil Engineer ample time to discuss the problems that may be encountered in the field and coordinate the work with the contractor.

Field observation and testing during the grading and/or foundation operations must be provided by representatives of ***KC ENGINEERING CO.***, to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to the grading and/or foundation operations performed without the full knowledge and under the direct observation of the Soil Engineer will render the recommendations of this report invalid.

Geotechnical Considerations

The primary geotechnical considerations for the multi-family structures and the equipment storage building are the presence of near surface highly expansive soils, the potential for total and differential settlements due to the anticipated building loads and seismically induced liquefaction, and the presence of shallow groundwater.

Laboratory testing of near surface samples obtained show that the surficial clay soils across the site are highly expansive. These soils are prone to heave and shrink movements with changes in moisture content. These highly expansive soils are expected to be removed by planned excavation of the parking garage basements for the multi-family buildings. However, at the equipment storage building, these expansive soils should be mitigated by lime treatment or placement of low to non-expansive materials under the building foundation.

To mitigate these concerns, we recommend that the upper 2 feet of existing grades be over-excavated, processed and compacted prior to placing any additional fills. Due to the on-site expansive soils, we recommend that low to non-expansive import materials or lime treated on-site soils be used to elevate the building pads and to allow the use of conventional spread footing foundations for the structures. Building pad grading recommendations are provided in the "Grading" section below.

As discussed in the "Settlement Considerations" section above, total settlement of up to 7.25 inches at Building 1 and 3 inches at Building 2 are expected due to structure loading and consolidation settlement. Additionally, 2 to 2.6 inches of seismically induced liquefaction settlement may occur across the site. In our opinion, the amount of anticipated total and differential settlement and/or angular distortion that may occur over the proposed building footprints is excessive for a conventional shallow spread footing and slab floor foundation and/or a thickened mat slab. To mitigate these concerns and to minimize the anticipated differential settlement, we recommend that the soil materials below the basement floors to a depth of 40 to 50 feet be mitigated by an appropriate ground improvement technique such as drilled displacement columns or compacted aggregate piers. The goal of ground improvement is to reduce the settlement potential to tolerable values and to provide an increased allowable bearing capacity for conventional mat slab or a spread footing design. As an alternate to subsurface ground improvement, a conventional driven pile and tie-beam foundation system may be utilized. In addition, flexible underground utility pipelines and connections, such as PVC and HDPE, with sand or gravel bedding and shading should be utilized to resist the effects of differential settlement. Increased slope gradients for storm drain and sewer pipelines should also be considered by the Civil Engineer.

Relatively shallow groundwater was encountered during our site exploration as shown on our boring logs. Groundwater as shallow as 13 feet was encountered in Boring 1. Shallow groundwater should be accounted for by the General Contractor during construction. Temporary dewatering and groundwater control during construction of the lower-level basements and underground utility construction is the responsibility of the Contractor. Temporary dewatering wells and sump pump systems will likely be required. With respect to long-term performance considerations, we recommend that underfloor subdrains and basement waterproofing measures be incorporated in the design of the multi-family basement parking garages by the project Architect, Civil Engineer and Structural Engineer. Groundwater control recommendations are provided below. A waterproofing consultant should be retained to aid in the design of the subterranean basement garages.

Considering the multi-family buildings will receive subsurface ground improvement, it is the opinion of **KC ENGINEERING COMPANY** that these structures be supported on a conventional thickened mat slab or spread footings with a thickened slab floor. Considering that the

equipment storage building pad will be lime treated or constructed with low to non-expansive soil, it is our opinion that this structure may be supported by a conventional spread footing and slab floor foundation system. Recommendations for these foundation systems are presented in the "Foundation" section herein.

Demolition

Prior to building pad grading, demolition of the existing buildings and foundations, asphalt pavements, underground utilities and other improvements should be performed under the proposed building footprints, plus a 5 feet lateral over-build. Demolition should include the complete removal of all surface and subsurface structures. Recycling of the concrete and asphalt pavements may be possible. Where any of the following are encountered: concrete, storm drain systems, foundations, asphalt, buried pipelines, tanks, etc.; these should be removed with the exception of items specified by the owner for salvage. In addition, all underground structures must be located on the grading plans so that proper removal may be carried out. It is vital that ***KC ENGINEERING CO.***, intermittently observe the demolition operations and be notified in ample time to ensure that subsurface structures are not covered.

Excavations made by the removal of any structure should be left open by the demolition contractor for backfill in accordance with the requirements for engineered fill. The removal of any underground structures or utility pipelines should be done under the observation of the Soil Engineer to assure adequacy of the removal and that subsoils are left in proper condition for placement of engineered fills. Any soil exposed by the demolition operations, which are deemed soft or unsuitable by the Soil Engineer, shall be excavated as uncompacted fill soil and be removed as required by the Soil Engineer during grading. The demolition operation should be approved by the Soil Engineer prior to commencing building pad grading operations. Any resulting excavations should be properly backfilled with engineered fill under the observation of the Soil Engineer. Should the location of any localized excavation be found to underlie any structure, backfill should be compacted to a minimum relative compaction of 92% or the excavation widened to extend 5 feet beyond the footprint of the structure and backfilled to the specifications for engineered fill as recommended in the "Grading" section below.

Groundwater Control & Dewatering

Relatively shallow groundwater was encountered during our site exploration as shown on our boring logs. Shallow groundwater should be accounted for by the General Contractor during construction. Temporary dewatering and groundwater control during construction of the lower-level basements and undergound utility construction are the responsibility of the Contractor. Temporary dewatering wells and sump pump systems will likely be required. Temporary dewatering well points and/or sump pits should be installed to facilitate pumping of collected

seepage water. The system should be designed to prevent migration and pumping of soil fines with discharge water.

With respect to long-term performance considerations, we recommend that underfloor subdrains and basement waterproofing measures be incorporated in the design of the multi-family basement level parking garages by the project Architect and Civil Engineer. We recommend that subdrains should be placed at approximate 25 feet horizontal intervals and consist of a 2 feet deep trench by 18 inches wide filled with Caltrans Class 2 Permeable drainrock with a 6 inch diameter SDR 35 perforated pipe placed 3 to 4 inches above the bottom of trench. Subdrain trenches and pipes should slope a minimum of 1% to a automatic trip sump ejector well pit or pits that discharge to the facility storm drain system as designed by the Architect and/or Civil Engineer. Retaining wall drainage recommendations are provided in the "Retaining Wall" section below. We recommend that basement and elevator pit waterproofing measures be incorporated in the design of all subterranean levels by the project Architect and a waterproofing consultant.

Should groundwater be encountered during underground utility construction, the utility excavation should begin at its lowest point and proceed uphill. The utility trench should be over-excavated 6 to 12 inches below the City of County required pipe bedding material. Open-graded 1.5-inch crushed aggregate should be placed in the bottom of the trench followed by the City or County standard bedding material. A sump pit should be excavated at the lowest point of the open excavation/trench to facilitate pumping of collected water. The collected water should be pumped to an approved discharge facility or dispersed on-site.

Temporary Excavations

Applicable safety standards require that excavations in excess of 4 feet must be properly shored or that the walls of the excavation be sloped back to provide safety for construction of the basements and pipeline trenches. We expect that some of the structure excavations or retaining walls will be open cut with no shoring. All temporary excavations should be designed, planned, constructed and maintained by the Contractor and should conform to all state and federal safety regulations and requirements.

Based on the subsurface soils encountered during our field exploration, it is our opinion that the soils may be considered to be Type B above the groundwater and Type C below per OSHA Standards. Based on our findings, it is our opinion that the proposed temporary excavations will perform adequately at a maximum 1H:1V slope with a 3 feet vertical at the toe. This is provided that the following comments and recommendations are included in the Contractor work plan:

1) The excavations must be monitored by competent personnel for stability and safety. Should any tension cracking or displacement be observed, the excavation must be backfilled immediately, or the upper slope laid back and flattened and our office notified for further recommendations.
2) The cut slope and adjacent surface should be covered with visqueen during rain events and to minimize rainfall infiltration.
3) Dewatering must be provided to maintain the groundwater level at 2 feet below the base of the excavation. Our Geotechnical Engineer should be contacted to evaluate the excavations and provide supplemental recommendations as necessary.
4) Stockpiling of spoils and equipment should be setback 10 feet minimum from the top of temporary back cuts.
5) Excavation stability and dewatering is the responsibility of the Contractor.

Temporary Shoring

Prior to excavation for the basement, temporary shoring will need to be installed to support the surrounding improvements and adjacent properties. Temporary shoring design and construction is the responsibility of the General shoring contractor. We expect the shoring system to consist of cantilevered soldier piers and timber lagging.

For temporary shoring design, we recommend the following parameters: a maximum friction angle of 28 degrees and a total unit weight of 120 pcf. A minimum shoring wall active pressure of 45 pcf representing an equivalent fluid weight is recommended. An allowable passive pressure of 220 pcf acting over 3 pier diameters is recommended.

The above design recommendations assume that there will not be any surcharge pressures from adjacent buildings on the shoring system. Vehicle and equipment surcharge loads should be applied. Applicable surcharge loads should be applied for any adjacent footing and/or other surcharge loading conditions where located within a 1:1 plane extending up from the base of the new excavation. We do recommend that monitoring and a pre-construction survey of adjacent improvements be performed prior to excavation and shoring installation. A monitoring and survey program should be developed and provided by the Contractor.

Drainage strips, such as Miradrain, should be placed behind temporary shoring walls. Typical drainage strips of 12 inch wide spaced every 5 feet on center is recommended. The Miradrain strips should connect to a collection pipe system at the toe of wall to be connected to a permanent discharge system as designed by the Civil or Structural Engineers.

Grading

Grading activities may be performed during the rainy season, however, achieving proper compaction may be difficult due to excessive moisture and delays will occur. Use of lime treatment or geogrids and geotextiles to stabilize soft areas and street subgrades may be required depending on actual moisture conditions at the time of grading. Grading performed during the dry months will minimize the occurrence of the above problems.

The surface of the site in areas to be graded should be stripped to remove all existing vegetation and/or other deleterious materials. Where any loose or soft soils are encountered, they must be over-excavated to undisturbed native ground. Excavated soil materials may be used as engineered fill with the approval of the Soils Engineer provided they do not contain organics.

Due to the planned subterranean basement level parking garages for the multi-family buildings, grading for these structures is primarily planned to consist of excavation and off-haul of materials. Temporary shoring will need to be installed to support the surrounding improvements and adjacent properties as the excavation proceeds to design depth. Temporary shoring design and construction is the responsibility of the General shoring contractor. Temporary excavation slopes and shoring recommendations are presented above. Access ramps into the basement may be facilitated by limited temporary open cut excavations or placement of fill.

After shoring installation and excavation of materials, we recommend that the exposed building subgrade soils be processed and compacted prior to ground improvement, subdrainage installation and foundation construction operations. We recommend that the upper 12 inches of the exposed subgrade soils be scarified, mixed and uniformly moisture conditioned to optimum moisture content or above and compacted to a minimum degree of relative compaction of 92% as determined by ASTM D1557 Laboratory Test Procedure. After processing the lower 12 inches and compacting the native subgrade, the site may be brought to the desired finished grades, if required, by placing engineered fill in lifts of 8 inches in un-compacted thickness and compacting to a relative compaction of 92% at or above optimum in accordance with the aforementioned test procedure. All soils encountered during our exploration are suitable for use as engineered fill when placed and compacted at the recommended moisture content.

Retaining wall backfill utilizing on-site soils or select import should be placed and compacted as recommended above. Utility trench backfill should be compacted to a minimum of 90% or as required by the City of Santa Clara Standard Specifications.

As mitigation to minimize differential heave and shrink movements from the highly expansive near surface clays at the equipment storage building, we recommend that the upper 1.5 feet of the building pad and concrete flatwork be processed and compacted as well compacted

structural fill by lime treatment. We recommend that the structural fill pad be a minimum of 1.5 feet thick, extend 5 feet beyond the building footprint and across concrete flatwork areas, and consist of the existing materials treated with high-calcium quicklime as described below. Alternatively, select import may be used. The upper 12 inches of the driveways and parking lots may also be lime treated, if desired, as described herein and in the "Pavement" section below.

It is recommended that the upper 1.5 feet of the equipment building pad and concrete flatwork comprise the on-site materials modified with high calcium quicklime meeting ASTM C977. It is noted that the structural fill should extend at least 5 feet beyond the building footprints and to the edge of surrounding flatwork, whichever is greater. The lime treatment should consist of a 5% mixture by dry weight. Based on a unit weight of 120 p.c.f., a spread rate of 9.0 p.s.f. applies to the 18-inch mixing depth. In the pavement areas, if desired, the upper 12 inches of subgrade should be lime treated with a minimum spread rate of 6.0 p.s.f. The lime treated soils should be compacted to at least 95% relative compaction of the maximum wet density per ASTM D1557 at a moisture content at least 4% above optimum. The lime treatment must be performed by a qualified soil stabilization contractor in general conformance with Caltrans Standard Specification Section 24. The product specification and quality control test results must be provided to us by the contractor for review and acceptance prior to the treatment operations. The lime should be spread and mixed with equipment capable of providing relatively uniform conditions. The lime treated section must be mixed and allowed to mellow over-night followed by a final mixing and compaction. After compaction, it is important to moist cure the lime treated soils until placement of the subsequent slab subbase materials (i.e. do not let pad dry out and desiccate).

Should select import material be used to establish the upper 18 inches of the structural fill pad or be required for general fill, the import material should be approved by the Soil Engineer before it is brought to the site. Where select import soil is used within the upper 18 inches of the pad, it should meet the following requirements:

a. Have a Plasticity Index not higher than 15;
b. Not more than 15% passing the No. 200 sieve;
c. No rocks larger than 3 inches in maximum size;
d. Caltrans Class 2 aggregate base may be used.

The fill materials shall be placed in uniform lifts of not more than 8 to 12 inches in uncompacted thickness depending on size and weight of equipment used. Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to obtain uniformity of material in each layer. Before compaction begins, the fill shall be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry.

Compaction shall be by footed rollers or other types of acceptable compacting rollers. Rollers shall be of such design that they will be able to compact the fill to the specified density. Rolling shall be accomplished while the fill material is within the specified moisture content range. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to ensure that the required density has been obtained. No ponding or jetting shall be permitted.

The standard test used to define maximum densities and optimum moisture content of all compaction work shall be the Laboratory Test procedure ASTM D1557 and field tests shall be expressed as a relative compaction in terms of the maximum dry density and optimum moisture content obtained in the laboratory by the foregoing standard procedure. Field density and moisture tests shall be made in each compacted layer by the Soil Engineer in accordance with ASTM D6938, respectively. When footed rollers are used for compaction, the density and moisture tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements for any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

Surface & Subsurface Drainage

A very important factor affecting the performance of structures is the proper design, implementation, and maintenance of surface and subsurface drainage, as well as maintaining uniform moisture conditions around the structures. Ponded water will cause swelling and/or loss of soil strength and may also seep under structures. Should surface water be allowed to seep under the structures, differential foundation movement resulting in structural damage and/or standing water under the slab will occur. This may cause dampness to the floor which may result in mildew, staining, and/or warping of floor coverings. To minimize the potential for the above problems, dampproofing and waterproofing should be provided as required by Section 1805 of the 2022 CBC. In addition, the following surface drainage measures are recommended and must be maintained by the property owner in perpetuity:

a) Positive building pad slopes and drainage must be provided by the project Civil Engineer to remove all storm water from the pad and to prevent storm and/or irrigation water from ponding adjacent to the structure foundations. The finished pad grade around the structures should be compacted and sloped 5% away from the exterior foundations and as required in Section 1804.4 of the 2022 CBC and be directed to yard swales and drainage outlets. Earth swales should slope a minimum of 2% to a suitable outlet.

b) Enclosed or trapped planter areas adjacent to the structure foundation should be avoided if possible. Where enclosed planter areas are constructed, these areas must

be provided with adequate measures to drain surface water (irrigation and rainfall) away from the foundation. Positive surface gradients and/or controlled drainage area inlets should be provided. Care should be taken to adequately slope surface grades away from the structure foundation and into area inlets. Drainage area inlets should be piped to a suitable discharge facility.

c) Adequate measures for storm water discharge from the roof gutter downspouts must be provided by the project Civil Engineer and maintained by the property owners at all times, such that no water is allowed to pond next to the structure. Closed pipe discharge lines should be connected to downspouts and discharged into a suitable drainage facility. It is important not to allow concentrated discharge on the surface of any slope so as to prevent erosion.

d) Site drainage should be designed by the project Civil Engineer. Civil engineering, hydraulic engineering, and surveying expertise is necessary to design proper surface drainage to assure that the flow of water is directed away from the foundations.

e) Over-irrigation of plants is a common source of water migrating beneath a structure. Consequently, the amount of irrigation should not be any more than the amount necessary to support growth of the plants. Foliage requiring little irrigation (drip system) is recommended for the areas immediately adjacent to the structure.

f) Landscape mounds or concrete flatwork should not be constructed to block or obstruct the surface drainage paths. The Landscape Architect or other landscaper should be made aware of these landscaping recommendations and should implement them as designed. The surface drainage facilities should be constructed by the contractor as designed by the Civil Engineer.

With respect to any proposed bio-retention swales or basins, we anticipate that bio-swales will be located relatively close to the proposed structures. We recommend a minimum separation of 10 horizontal feet where possible. The bottom of the swales and/or treatment materials should be sloped away from the structure foundation a minimum of 5%. In addition, we recommend that a subsurface drain be provided below the select treatment soils and drainrock at the low side of the swale/basin. The subdrain should be connected to the nearest storm drain catch basin. A 4 inch SDR35 perforated pipe surrounded by Caltrans Class 2 Permeable Material should be provided to discharge collected water into the nearest catch basin. An impermeable liner may also be required in the bottom of the swales where located closer than 10 feet from a building foundation. Structure foundations where located adjacent to bio-treatment swales should be deepened 1 foot below the bottom of the treatment section. Additional details can be provided when plans are available.

With respect to long-term performance considerations, we recommend that underfloor subdrains be incorporated in the design of the subterranean basements as designed by the by the Architect and Civil Engineer. Subdrains should be placed at approximate 25 feet horizontal intervals and consist of a 2 feet deep trench by 18 inches wide filled with Caltrans Class 2 Permeable drainrock with a 6 inch diameter SDR 35 perforated pipe placed 3 to 4 inches above the bottom of trench. Subdrain trenches and pipes should slope a minimum of 1% to a sump ejector pit or storm drain catch basin.

Foundations

As discussed in the "Geotechnical Considerations" section above, the amount of anticipated total and differential settlement and/or angular distortion that may occur over the proposed multi-family building footprints are excessive for a conventional shallow spread footing and slab floor foundation and/or a thickened mat slab. To mitigate these concerns and to minimize the anticipated differential settlement, we recommend that the soil materials below the basement floors by an appropriate ground improvement technique such as drilled displacement columns (DDC) with pressurized grout or compacted aggregate piers (CAP). The goal of ground improvement is to reduce the settlement potential to tolerable values and to provide an increased allowable bearing capacity for a conventional mat slab or a spread footing design. Ground improvement should extend to a depth of 40 to 50 feet below grade and be designed and constructed by a qualified design-build contractor such as Geopier or Farrell Design-Build. The ground improvement contractor should review the geotechnical data and work with the project Structural Engineer to provide a design that will limit the settlement and provide support to the mat slab. Pre-production pile load testing should be performed on either the DDC or CAP systems.

After installation of the DDC or CAP ground improvement inclusions, the multi-family structures may be supported on either a thickened mat slab or a spread footing and thickened slab floor system. Considering that the equipment storage building pad will be lime treated or constructed with low to non-expansive soil, it is our opinion that this structure may be supported by a conventional spread footing and slab floor foundation system. Recommendations for both systems are provided below.

Mat Slab

As discussed above, a thickened mat slab foundation system may be used for the multi-family structures. We recommend that the mat slabs be a minimum of 18 inches thick and reinforced as required by the Structural Engineer. We recommend a subgrade modulus value of 125 psi/in and an allowable bearing capacity of 3,000 psf for cases where DDC or CAP ground improvement inclusions are used. This bearing value may be increased by one-third to account for transient

wind and seismic loading conditions. Increased modulus and bearing values may be utilized provided ground improvement by DDC and CAP are used and coordinated with the Ground Improvement Engineer and Structural Engineer. Foundations designed in accordance with the above criteria are expected to experience a total settlement of less than 1 inch in 50 feet and less than ½ of an inch of differential settlement across a span of 30 feet.

The subgrade soils under mat slabs should be prepared as recommended in the "Grading" section above. Mat slabs should be underlain by a minimum of 6 inches of Caltrans Class 2 Aggregate Base compacted to a minimum relative compaction of 95%. A waterproofing membrane should be placed between the compacted aggregate base and concrete mat slab as designed by a waterproofing consultant. Under floor subdrainage is also recommended as presented above.

We recommend that appropriate provisions be provided by the Structural Engineer and Contractor to minimize mat slab cracking, such as curing measures and/or admixtures to minimize concrete shrinkage and curling. American Concrete Institute methods and guidelines of curing, such as wet curing or membrane curing, are recommended to minimize drying shrinkage cracking.

Spread Footings

Continuous spread footings should be utilized around the perimeter of the structure and for all interior bearing and shear walls and be a minimum of 2 feet wide. The continuous footings should extend a minimum depth of 24 inches below the interior slab subgrade soil elevation or lowest adjacent grade. The recommended design allowable bearing pressure for footings where used for the multi-family buildings is 3,000 psf and 2,000 psf for the equipment storage building due to dead plus live loads. This value may be increased by one-third for transient wind and seismic loads.

All foundations must be adequately reinforced to provide structural continuity and resist the anticipated loads as determined by the project Structural Engineer. The final footing design and reinforcement should be determined by the project Structural Engineer. However, continuous footings and tie-beams are recommended to be reinforced with a minimum of six No. 5 bars, three at the top and three near the bottom of the footing. Additional reinforcement will be as required by the Structural Engineer and in accordance with structural building code requirements. Foundations designed in accordance with the above criteria are expected to experience a total settlement of less than 1 inch with less than ½ of an inch of differential settlement across a span of 30 feet.

To accommodate lateral building loads, the passive resistance of the foundation soil can be utilized. The passive soil pressures can be assumed to act against the front face of the footing

below a depth of 1 foot below the ground surface. It is recommended that an allowable passive pressure equivalent to that of a fluid weighing 220 p.c.f. be used. For design purposes, an allowable friction coefficient of 0.35 can be assumed at the base of the spread footings. These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since the mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

Slab-on-Grade Construction

The interior floor slabs of the equipment storage building and exterior concrete flatwork and driveways will likely experience some cracking due to finishing, curing methods, drying shrinkage, as well as moisture variations and related soil movements within the underlying clay soils. We should note that City or County maintained curbs, gutters, sidewalks and driveway aprons should be designed and constructed per the City or County Standards, Specifications and Plans. To reduce the potential cracking of the slabs-on-grade, the following recommendations are made:

a) All areas to receive slabs should be thoroughly wetted and soaked in the upper 8 inches to seal any desiccation or shrinkage cracks prior to placing concrete. This work should be done under the observation of the Soil Engineer.

b) Slabs should be underlain by a minimum of 4 inches of angular gravel or clean crushed rock material placed between the finished subgrade and the slabs to serve as a capillary break between the subsoil and the slab. The gravel should not have more that 10% passing the No. 4 sieve per CBC Section 1805.4.1. Caltrans Class 2 aggregate base may also be used provided it is compacted to a minimum of 90%.

c) Interior floor slabs and exterior slabs for attached patios, structure entries, outdoor areas, and auto parking stalls should be a minimum of 5 inches thick and reinforced with a minimum of No. 4 rebar spaced 18 inches center to center, each way. Additional PCC pavement recommendations are presented under the "Pavement" section of this report. The actual slab thickness and reinforcement should be determined by the project Structural Engineer in accordance with the structural requirements and the anticipated loading conditions. The reinforcement shall be placed in the center of the slab unless otherwise designated by the design engineer.

d) Where moisture vapor is a concern, such as for all interior floor slabs, a vapor retarder membrane should be installed between the prepared building pad aggregate base and the interior slabs to minimize moisture condensation under the floor coverings and/or upward vapor transmission. The vapor barrier membrane should be a minimum 15-mil extruded polyolefin plastic that complies

with ASTM E1745 Class A and have a permeance of less than 0.01 perms per ASTM E96 or ASTM F1249. It is noted that polyethylene films (visqueen) do not meet these specifications. The vapor barrier must be adequately lapped and taped/sealed at penetrations and seems in accordance with ASTM E1643 and the manufacturer's specifications. The vapor retarder must be placed continuously across the slab area.

e) Water vapor migrating to the surface of the concrete can adversely affect floor covering adhesives. Provisions should be provided in the concrete mix design to minimize moisture emissions. This should include the selection of a water-cement ratio which inhibits water permeation (0.45 max) and/or the addition of suitable admixtures to limit water transmission.

f) Slabs for driveways, entries, attached patios and exterior flatwork should be placed structurally independent of the foundations. Driveway/pavement slab recommendations are presented in the "Pavement" section of the report. A 30-pound felt strip, expansion joint material, or other positive separator should be provided around the edge of all floating slabs to prevent bonding to the foundation. However, rebar doweling to the foundation is recommended to minimize vertical movements between exterior slabs and building foundations. Doweling details should be determined by the Structural Engineer.

g) To minimize moisture infiltration under exterior slabs and to add edge rigidity, we recommend that slabs be thickened at the edges to extend below the aggregate base layer to the soil subgrade for a minimum width of 6 inches.

h) Slabs should be provided with crack control saw cut joints or tool joints to allow for expansion and contraction of the concrete. In general, contraction joints should be spaced no more than 20 times the slab thickness in each direction. The layout of the joints should be determined by the project Structural Engineer and/or Architect.

i) We recommend that appropriate provisions be provided by the Structural Engineer and Contractor to minimize slab cracking, such as curing measures and/or admixtures to minimize concrete drying-shrinkage and curling. American Concrete Institute methods and guidelines of curing, such as wet curing or membrane curing, are recommended to minimize drying shrinkage cracking.

Retaining Walls

The basement walls for the multi-family buildings and elevator pits, along with other site walls that are incorporated into the project should be designed to resist lateral pressures exerted from a media having an equivalent fluid weight as noted the following table.

Gradient of Back Slope	Equivalent Fluid Weight (p.c.f.)			Coefficient of Friction
	Unrestrained Condition (Active)	Restrained Condition (At Rest)	Passive Resistance	
Horizontal	45	65	220	0.35

It should be noted that the effects of any surcharge or compaction loads behind the walls must be accounted for in the design of the walls. We recommend that the project Structural Engineer use the formula $P_Q = QHKa$ where Q = uniform surcharge load in psf, Ka = 0.37 for soil backfill and Ka = 0.26 for gravel backfill, and H = wall height. Because the surcharge pressure acting on the retaining wall is considered relatively uniform, the resultant force P_Q should be applied at mid-height of the wall.

Per Section 1803.5.12 of the 2022 California Building Code, dynamic lateral earth pressures on retaining walls supporting more than 6 feet of backfill in height are required. Based on the Mononobe-Okabe & Seed-Whitman equations, a total unit weight of 120 pcf and a Kh of ½ PGAm, an earthquake load of $14.5H^2$ should be applied at 1/3H where H = wall height, from the bottom of the wall is applicable.

We recommend that the basement and elevator pit walls be supported by mat slab or footing foundation as presented above. Other low height retaining walls (less than 5 feet) may be supported by spread footings.

The above criteria are based on fully drained conditions. In order to achieve fully-drained conditions, a gravel drainage filter blanket should be placed behind the wall. Basement walls should also be waterproofed as designed by a waterproofing consultant. The gravel blanket should be a minimum of 12 inches thick and should extend to the bottom of the mat slab and within 12 inches of the surface and capped with compacted soil. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket should consist of compacted engineered fill or blanket material. The gravel drainage blanket material may consist of either granular crushed rock or drainpipe fully encapsulated in geotextile filter fabric (Mirafi 140N or equivalent) or Class II permeable material that meets CalTrans Specification, Section 68. A 4-inch diameter SDR35 perforated drainpipe should be installed in the bottom of the drainage blanket and should be underlain by 4 inches of filter type material. Piping with a minimum gradient of 2% shall be provided to discharge water that collects behind the walls to

an adequately controlled discharge system away from the structure foundations. Perimeter basement wall drains should extend a minimum depth of 2 feet below the bottom of the mat/floor slab.

Pavement Areas

The driveways and parking areas are expected to be paved with either asphalt concrete (AC) or Portland cement concrete (PCC) surfaces. Recommendations for these pavement surfaces are presented below. We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, as well as engineered fill and utility trench backfill within the limits of pavements. Pavements will typically have poor performance and shorter life where water is allowed to migrate into the aggregate base and subgrade soils. The main sources of water into pavement materials are landscape planters constructed within or adjacent to pavement areas. Where this is planned, it is suggested to extend the curbs into the soil subgrade at least 2 inches. The construction of all pavements should conform to the requirements set forth by the latest Standard Specifications of the Department of Transportation of the State of California (Caltrans) and/or City or County of Santa Clara.

R-Value: A bulk sample was obtained of the near surface soils within the planned street areas that are representative of the anticipated subgrade soils. The sample was tested in accordance with the California Test Method 301 to determine the R-Value for the site soils. R-Value of 10 was determined.

Preparation of Subgrade: After underground utilities have been placed in the areas to receive pavement and removal of excess material has been completed, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned and compacted to a minimum relative compaction of 95% at a moisture content at 3% or more above optimum in accordance with the grading recommendations specified in this report. The upper 12 inches of the subgrade may alternatively be lime treated. Prior to placement of aggregate baserock, it is recommended that the subgrade be proof rolled and observed for deflection by the Soils Engineer. Should deflection and/or pumping conditions be encountered, stabilization recommendations will be provided based on field conditions.

Aggregate Base: All aggregate base material placed subsequently should also be compacted to a minimum relative compaction of 95% based on the ASTM Test Procedure D1557. Aggregate base should meet the minimum requirements of Caltrans ¾" Class 2 per Section 26 and be crushed and angular. The recommended aggregate base thicknesses for asphalt concrete pavements are noted in the table below. The minimum aggregate base thickness for Portland cement concrete PCC roadway pavements is 6 compacted inches.

Asphalt Concrete: Asphalt concrete shall conform with Section 39 of Caltrans Standard Specifications and shall be per the City or County of Santa Clara Standards. Based on an R-Value of 10, and traffic indices typical for commercial/farming projects, the recommended pavement sections for aggregate and asphalt concrete surfaces are summarized in the table below. Should the driveway and parking lot soils be lime treated, we are providing an alternate section based on a minimum R-value of 30. The appropriate traffic index (TI) and any minimum pavement sections should be determined by the Civil Engineer in conformance with the City or County of Santa Clara.

Traffic Condition	Traffic Index (TI)	Asphalt Concrete (inches)	Class II Aggregate Base[1] (inches)
Auto Parking Stalls	4.5	3.0 3.0	7.5 2.0*
Drive Isles/Lanes	6.5	4.0 4.0	12.5 6.5*

NOTES:
(1) Minimum R-Value = 78
(2) All layers in compacted thickness to CalTrans Standard Specifications.
* 12" Lime Treated Subgrade (R-Value = 30 min)
** AB underlain with Tensar NX650 Geogrid.

Portland Cement Concrete: Where PCC pavement areas are utilized, such as for drive isles and truck areas or trash enclosures, the concrete should be poured on the compacted aggregate base layer described above of 6 inches. We recommend a minimum of 6 inches thick PCC reinforced with a minimum of No. 4 rebar spaced at 16 inches on center, each way, underlain by 6 inches of compacted Class 2 aggregate base. Pavement joints shall be per the HDM and City/County Standards.

Underground Utility and Excavations

Groundwater was encountered at depths as shallow as 1.5 feet at the time of our exploration. Depending on the time of year of underground construction, higher groundwater may be encountered especially in deeper utilities. Temporary dewatering and shoring are the responsibility of the Contractor.

Should groundwater be encountered, the utility construction should begin at its lowest point and proceed uphill. The utility trench should be over-excavated 6 to 12 inches below the City/County required pipe bedding material. Open-graded 1.5-inch crushed aggregate should be placed in the bottom of the trench followed by the City/County standard bedding material. A sump pit should be excavated at the lowest point of the open excavation/trench to facilitate pumping of

collected water. The collected water should be pumped to a City/County approved discharge facility.

Utility trenches extending underneath all traffic areas must be backfilled with native or import soil materials and compacted to relative compaction of 90% to within 12 inches of the subgrade. The upper 12 inches should be compacted to 95% relative compaction in accordance with Laboratory Test Procedure ASTM D1557. Backfilling and compaction of these trenches must also meet the requirements set forth by the City or County of Santa Clara, Department of Public Works.

Applicable safety standards require that excavations in excess of 5 feet must be properly shored or that the walls of the excavation slope back to provide safety for installation of lines. If trench wall sloping is performed, the inclination should vary with the soil type and applicable OSHA Safety Standards.

With respect to state-of-the-art construction or local requirements, utility lines are generally bedded with granular materials. These materials can convey surface or subsurface water beneath the structures. It is, therefore, recommended that all utility trenches which possess the potential to transport water be sealed with a compacted impervious cohesive soil material or lean concrete where the trench enters/exits the building perimeter. This impervious seal should extend a minimum of 2 feet away from the building perimeter.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his representative to notify ***KC ENGINEERING CO.***, in writing, a minimum of two working days before any clearing, grading, or foundation excavation operations can commence at the site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings and from a reconnaissance of the site. Should any variations or undesirable conditions be encountered during the development of the site, ***KC ENGINEERING CO.***, will provide supplemental recommendations as dictated by the field conditions.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

4. At the present date, the findings of this report are valid for the property investigated. With the passage of time, significant changes in the conditions of a property can occur due to natural processes or works of man on this or adjacent properties. In addition, legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may render this report invalid, wholly or partially. Therefore, this report should not be considered valid after a period of two (2) years without our review, nor should it be used, or is it applicable, for any properties other than those investigated.

5. Notwithstanding, all the foregoing applicable codes must be adhered to at all times.

APPENDIX

Aerial Vicinity Map

Site Plans

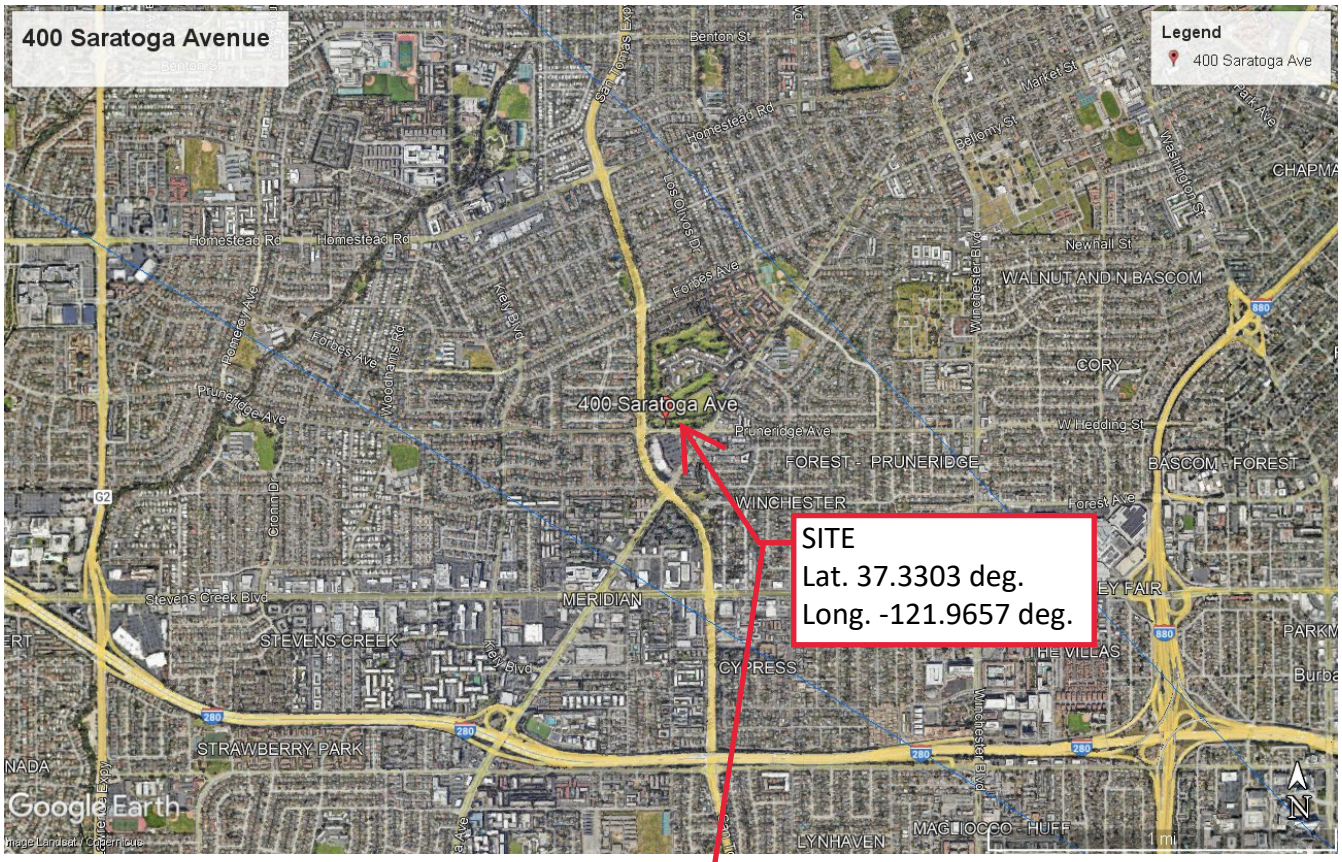
Geologic Map

Log of Test Borings

Subsurface Exploration Legend

Laboratory Test Results

ASCE 7 Hazards Report





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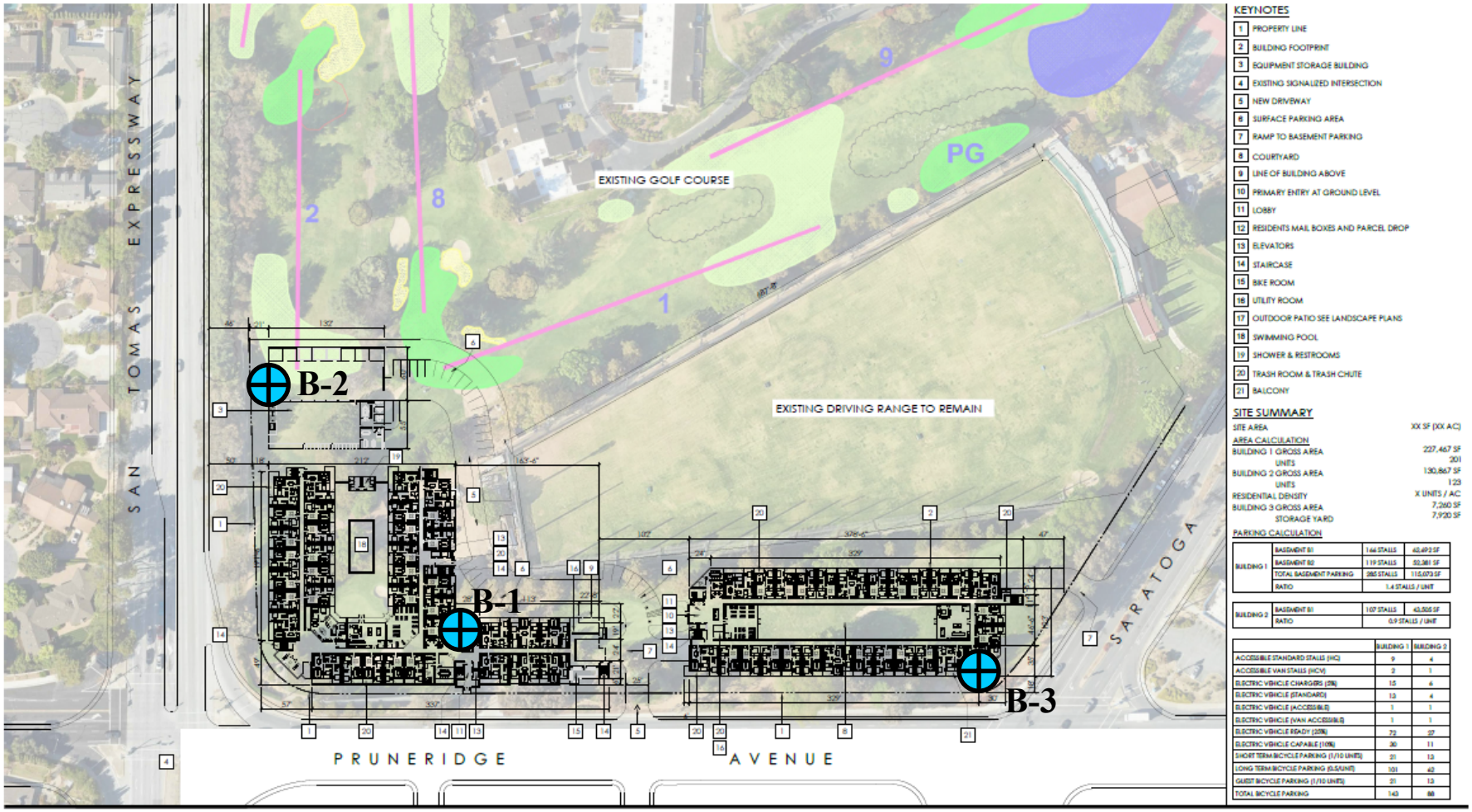
Project No. VV5781
Proposed Pruneridge Golf Club Multi-Family
400 Saratoga Avenue, Santa Clara, CA
Figure 1 – AERIAL VICINITY MAP



BUILDING 1	BASEMENT B1	166 STALLS	60,692 SF
	BASEMENT B2	119 STALLS	52,381 SF
	TOTAL BASEMENT PARKING	285 STALLS	113,073 SF
	RATIO	1.4 STALLS / UNIT	

BUILDING 2	BASEMENT B1	107 STALLS	43,506 SF
	RATIO	0.9 STALLS / UNIT	

	BUILDING 1	BUILDING 2
ACCESSIBLE STANDARD STALLS (HC)	9	4
ACCESSIBLE VAN STALLS (HCV)	2	1
ELECTRIC VEHICLE CHARGERS (5%)	15	6
ELECTRIC VEHICLE (STANDARD)	13	4
ELECTRIC VEHICLE (ACCESSIBLE)	1	1
ELECTRIC VEHICLE (VAN ACCESSIBLE)	1	1
ELECTRIC VEHICLE READY (20%)	72	27
ELECTRIC VEHICLE CAPABLE (10%)	30	11
SHORT TERM BICYCLE PARKING (1/10 UNITS)	21	13
LONG TERM BICYCLE PARKING (0.5/UNIT)	101	62
GUEST BICYCLE PARKING (1/10 UNITS)	21	13
TOTAL BICYCLE PARKING	143	88

NEW HOUSING DEVELOPMENT AT PRUNERIDGE GOLF CLUB

400 SARATOGA AVENUE
SANTA CLARA, CALIFORNIA
DEMMON PARTNERS
KENNETH RODRIGUES & PARTNERS, INC.

37.666 03.13.2024

DIMENSIONED SITE PLAN

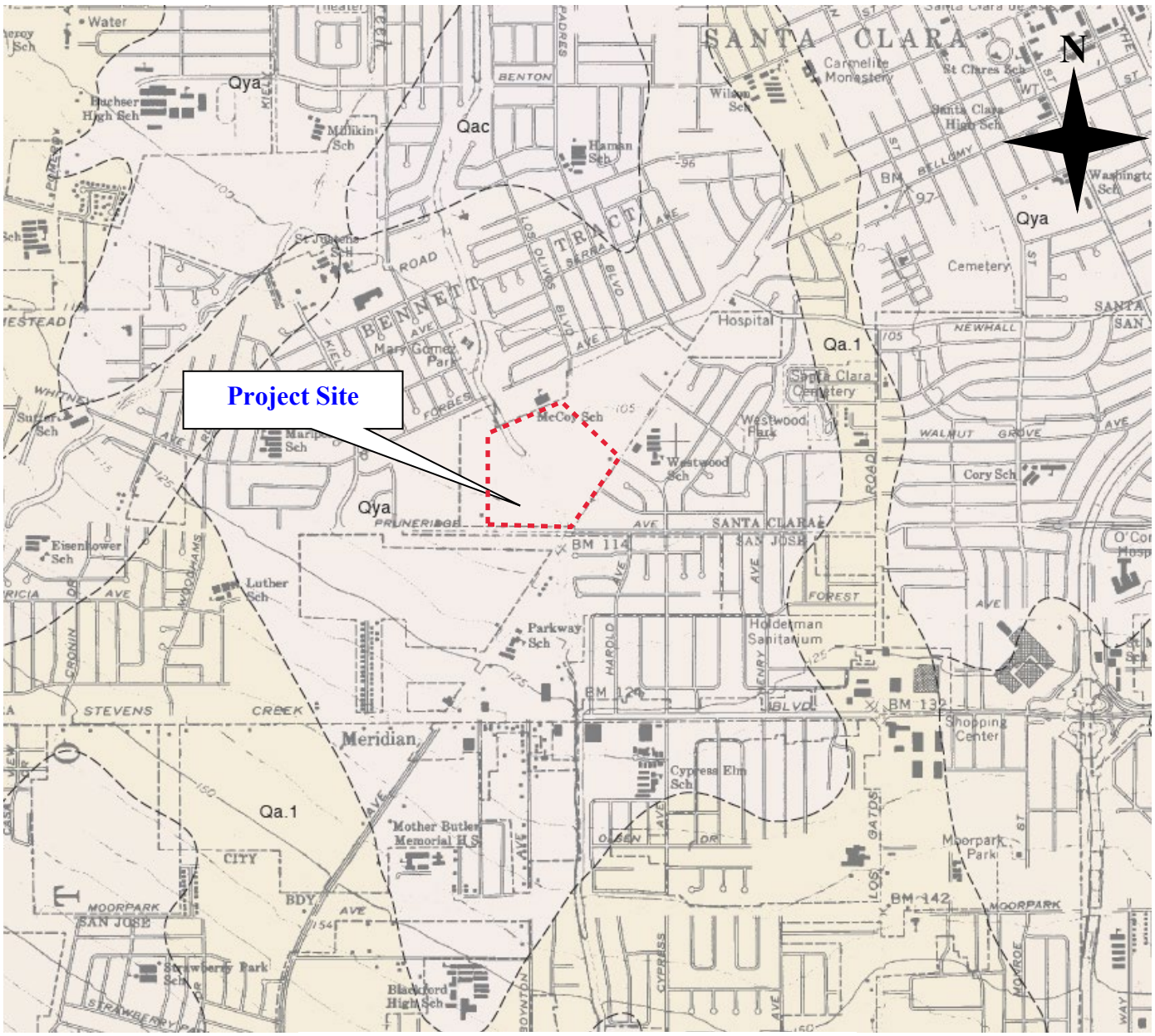
A1.1

LEGEND

Approximate Boring Locations

KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688
707.447.4025

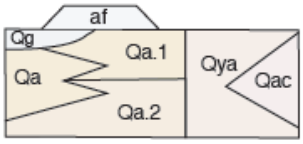
Project No. VV5781
Proposed Pruneridge Golf Multi-Family
400 Saratoga Avenue, Santa Clara, California
Figure 2 - SITE PLAN



GEOLOGIC MAP OF THE CUPERTINO and SAN JOSE WEST QUADRANGLES

SANTA CLARA AND SANTA CRUZ COUNTIES, CALIFORNIA

BY THOMAS W. DIBBLEE, JR., 2007



SURFICIAL SEDIMENTS

Qa *Alluvial gravel, sand, silt, and clay; represents undifferentiated stream alluvium in drainages and younger alluvial fan deposits at base of slopes and on fan areas*

Qg *Alluvial gravel, sand, silt, and clay; represents undifferentiated stream alluvium in drainages*

af *Artificial fill*

Qa.1 *Alluvial sand, fine-grained, silt, and gravel; where differentiated represents alluvial fan deposits at base of slopes and upper fan areas; mapped as part of undivided* **Qa** *where not differentiated*

Qa.2 *Alluvial gravel, sand, silt, and clay; represents younger stream alluvium in fan deposits; mapped as part of undivided* **Qa** *where not differentiated*

Qya *Alluvial sand, fine-grained, silt, and clay; where differentiated represents distal alluvial fan deposits at outer edge of fan areas; mapped as part of undivided* **Qa** *where not differentiated*

Qac *Silty clay and organic clay, fossiliferous; represents intra-fan areas; mapped as part of undivided* **Qa** *where not differentiated*

KC ENGINEERING COMPANY
865 Cotting Lane, Suite A
Vacaville, CA 95688
707.447.4025

Project No. VV5781
Proposed Pruneridge Golf Club Multi-Family
400 Saratoga Avenue, Santa Clara, CA
Figure 3 – GEOLOGIC MAP

LOG OF TEST BORING

BORING NO.: 1

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 21'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 13' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Dark Brown Sandy CLAY; moist, stiff.	CL/CH					
	1-1					13	89.2	27.8	2.75	
5										
	1-2			As Above; very stiff.		16	100.2	24.6	3.25	
				Dark Olive Brown Silty CLAY; moist, very stiff.	CL					
10										
	1-3					26	106.2	20.3	4.5	
				Light Olive Brown Sandy CLAY; moist, very stiff.	CL					
15										
	1-4					18	104.3	17.5	4.5	ϕ=25.5° c=726 psf
20										
				Light Olive Brown Clayey SAND; very moist to wet, medium dense. ▽	SC					
	1-5					10	107.3	19.2		%<200=38
25										
	1-6			As Above.		14	103.8	18.1		%<200=30

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 4

LOG OF TEST BORING

BORING NO.: 1

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 21'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 13' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
30	1-7			Gray & Brown Gravelly SAND; wet, dense	SP	42	117.6	12.1		
35	1-8			Gray Sandy CLAY; wet, stiff.	CL	12	99.7	24.4	2.5	%<200=65
40	1-9			Gray SAND with Silt & some Pea Gravels; wet, medium dense.	SP-SM	22	101.8	23.6		%<200=6.3
45	1-10			Light Olive Brown Sandy CLAY; wet, hard to very stiff.	CL	37	116.5	16.6	4.5	
	1-11					16				%<200=56
50				Boring Terminated @ 48'. Groundwater Encountered @ 21' then rose to 13'						
55										

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 4

LOG OF TEST BORING
BORING NO.: 2

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 22'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 19' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Dark Brown Sandy CLAY; moist, very stiff w/ some roots.	CH					
	2-1					17	92.7	22.6	4.0	ϕ=28.3° c=503 psf
5	2-2			As Above; very stiff.		22			4.25	LL=62 PI=41
10				Mottled Dark Olive Brown Sandy CLAY; moist, very stiff.	CL					
	2-3					27	104.3	18.8	4.0	
15	2-4			As Above.		26	107.2	23.1	4.5+	
20				Olive & Gray Sandy CLAY; very moist, stiff.	CL					
	2-5					10	89.8	24.7	1.25	ϕ=28° c=425 psf
25	2-6			As Above.		14	97.0	28.2	2.25	UCC=1,894 psf

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 5

LOG OF TEST BORING
BORING NO.: 2

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 22'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 19' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
30	2-7			Olive Brown Sandy CLAY; wet, stiff.	CL	9	103.8	20.8		%<200=54
35	2-8			Black Sandy CLAY; wet, stiff.	CL	9	100.7	23.1	1.5	%<200=59 Pc=591 psf
40	2-9			Gray CLAY; wet, firm to stiff. As Above.	CL	8	106.6	21.6	1.5	
45	2-10			Olive Brown CLAY with Sand & Few Fine Gravels; wet, stiff.	CL	12	108.4	20.1	2.75	
50				Boring Terminated @ 46'. Groundwater Encountered @ 22' then rose to 19'.						
55										

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 5

LOG OF TEST BORING
BORING NO.: 3

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 30'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 25' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
0				Dark Brown to Black Sandy CLAY; moist, very stiff.	CL					
	3-1					18	103.7	19.2	4.5	LL=45 PI=26
5	3-2			As Above.		16	100.8	24.6	2.75	
				Mottled Olive Brown Sandy CLAY; moist, stiff.	CL					
10	3-3					14	103.9	22.0	2.75	UCC=4,182 psf
				Olive & Gray Sandy CLAY w/ Gravels; moist, hard.	CL					
15	3-4					36	116.6	5.8		
				Olive & Gray Sandy GRAVEL; wet, meduim dense to dense.	GW					
20	3-5			As Above.		30	114.4	8.8		
				Olive Brown Fine Sandy CLAY; wet, stiff.	CL					
25	3-6			▼		13	101.8	22.3	1.25	UCC=2,284 psf %<200=68

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 6

LOG OF TEST BORING
BORING NO.: 3

PROJECT: Pruneridge Golf Club Mulit-Family
CLIENT: Demmon Partners
LOCATION: 400 N. Saratoga Ave., Santa Clara, CA
DRILLER: Britton
DRILL RIG: CME55
DEPTH TO WATER: INITIAL ▽ : 30'

PROJECT NO.: VV5781
DATE: 04/29/24
ELEVATION: n/a
LOGGED BY: DVC
BORING DIAMETER: 8"
FINAL: ▼ : 25' AFTER: 2 HRS

DEPTH	SAMPLE NO.	SAMPLER	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	SOIL CLASSIFICATION	CONVERTED SPT BLOW COUNT (BLOWS/FT.)	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	Qp (t.s.f.) Penetrometer	ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, ø&c, Gradation)
30	3-7			Gray Clayey SAND; wet, loose.	SC	6	105.7	21.2		%<200=44 Pc=923 psf
35	3-8			Gray Clayey SAND with Few Gravels; wet, medium dense.	SC	14	104.6	21.6		%<200=43
40	3-9			Gray Sandy CLAY; wet, firm.	CL	8			1.25	%<200=56
45	3-10			Gray CLAY; wet, stiff.	CL	12	102.8	24.6	1.75	
				Boring Terminated @ 46.5'. Groundwater Encountered @ 30' then rose to 25'.						
50										
55										

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.

Figure 6

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is retained on the No. 200 Sieve	GRAVEL More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (<5% fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines (Cu>4 & 1≤Cc≤3)
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines (Cu < 4 and/or 1>Cc>3)
		Gravel with fines (>12% fines)	GM		Silty gravels and gravel-sand-silt mixtures (PI<4 or below "A" line)
			GC		Clayey gravels and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
	SAND Half or more of the coarse fraction is smaller than No. 4 sieve	Clean sands (<5% fines)	SW		Well graded sands, gravelly sands, little or no fines (Cu>6 & 1≤Cc≤3)
			SP		Poorly graded sands, gravelly sands, little or no fines (Cu<6 and/or 1>Cc>3)
		Sand with fines (>12% fines)	SM		Silty sands and gravel-sand-silt mixtures (PI<4 or below "A" line)
			SC		Clayey sands and gravel-sand-clay mixtures (PI>7 & on or above "A" line)
FINE GRAINED SOILS Half or more of the material passes the No. 200 Sieve	SILTS AND CLAYS Liquid Limit is less than 50%		ML		Inorganic silts with gravel and sand having slight plasticity (PI<4 or below "A" line)
			CL		Inorganic clays of low to med. plasticity with gravel and sand (PI>7 & on or above "A" line)
			OL		Organic silts and clays of low plasticity
	SILTS AND CLAYS Liquid Limit is 50% or more		MH		Inorganic elastic silts (PI below "A" line)
			CH		Inorganic clays of high plasticity, fat clays (PI on or above "A" line)
			OH		Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils

MTI-KC ENGINEERING COMPANY
865 Cotting Lane, Ste A, Vacaville, CA 95688
8798 Airport Road, Redding, CA 96002

SAMPLER AND LAB TESTING LEGEND

- Auger
- Bulk Sample, taken from auger cuttings
- California Sampler
- Bulk/Grab Sample
- Pitcher
- Standard Penetration Test
- Shelby Tube
- No Recovery

LL=Liquid Limit (%)
PI=Plasticity Index
¦ =Friction Angle
C=Cohesion
UCC=Unconfined Compression
R value=Resistance Value
Consol=Consolidation Test

SOIL GRAIN SIZE

U.S. STANDARD SIEVE OPENINGS

		#200		#40		#10		#4	¾"		3"		12"
CLAY	SILT	SAND							GRAVEL			COBBLES	BOULDERS
		FINE		MEDIUM		COARSE			FINE		COARSE		
	0.002		0.075		0.425		2.00	4.75		19.0		75	300

SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY (Coarse-grained soils)

SANDS & GRAVELS	BLOWS/FOOT[1]
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	> 50

CONSISTENCY (Fine-grained soils)

SILTS & CLAYS	STRENGTH[2]	BLOWS/FOOT[1]
Very Soft	< 500	0 – 2
Soft	500 – 1,000	2 – 4
Firm	1,000 – 2,000	4 – 8
Stiff	2,000 – 4,000	8 – 15
Very Stiff	4,000 – 8,000	15 – 30
Hard	> 8,000	>30

1 – Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. split spoon sampler (ASTM D1586)
2 – Unconfined compressive strength in lb/ft^2 as determined by lab testing or approximated by the standard penetration test (ASTM D1586) or pocket penetrometer.

WEATHERING (Bedrock)

Fresh	No visible sign of decomposition or discoloration; rings under hammer impact
Slightly weathered	Slight discoloration inwards from open fractures; little or no effect on normal cementation; otherwise similar to Fresh
Moderately weathered	Discoloration throughout; weaker minerals decomposed; strength somewhat less than fresh rock but cores can not be broken by hand or scraped with knife; texture preserved; cementation little to not affected; fractures may contain filling
Highly weathered	Most minerals somewhat decomposed; specimens can be broken by hand with effort or shaved with knife; texture becoming indistinct but fabric preserved; faint fractures
Completely weathered	Minerals decomposed to soil but fabric and structure preserved; specimens can be easily crumbled or penetrated

STRENGTH (Bedrock)

Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen will crumble under light hammer blows
Moderately strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments
Very strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

BEDDING (Bedrock) SPACING (inches)

Very thickly bedded	> 48
Thickly bedded	24 to 48
Thin bedded	2.5 to 24
Very thin bedded	5/8 to 2.5
Laminated	1/8 to 5/8
Thinly laminated	<1/8

FRACTURING (Bedrock) SPACING (inches)

Very little fractured	> 48
Occasionally fractured	12 to 48
Moderately fractured	6 to 12
Closely fractured	1 to 6
Intensely fractured	5/8 to 1
Crushed	<5/8

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client: Demmon Partners
601 University Avenue, Suite 110
Sacramento, CA 95825

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara

Date: 05/21/2024
Client No.: VV5781
Report No.: 0300-001
Submitted By: KC Engineering
Date Submitted: 04/30/2024

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
1-1 @ 3.0’	Dark Brown Sandy Clay (visual)	89.2	27.8	---	---	---
1-2 @ 6.0’	Dark Brown Sandy Clay (visual)	100.2	24.6	---	---	---
1-3 @ 11.0’	Dark Olive Brown Silty Clay (visual)	106.2	20.3	---	---	---
1-4 @ 16.0’	Light Olive Brown Sandy Clay (visual)	104.3	17.5	---	---	---
1-5 @ 21.0’	Light Olive Brown Clayey Sand (visual)	107.3	19.2	---	---	---
1-6 @ 26.0’	Light Olive Brown Clayey Sand (visual)	103.8	18.1	---	---	---
1-7 @ 31.0’	Gray and Brown Gravelly Sand (visual)	117.6	12.1	---	---	---
1-8 @ 36.0’	Gray Sandy Clay (visual)	99.7	24.4	---	---	---
1-9 @ 41.0’	Gray Sand with Silt (visual)	101.8	23.6	---	---	---
1-10 @ 46.0’	Light Olive Brown Sandy Clay (visual)	116.5	16.6	---	---	---
2-1 @ 3.0’	Dark Brown Sandy Clay with Organics (visual)	92.7	22.6	---	---	---
2-3 @ 11.0’	Mottled Dark Olive Brown Sandy Clay (visual)	104.3	18.8	---	---	---

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.

Materials Testing, Inc.

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Client: Demmon Partners
601 University Avenue, Suite 110
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400 Saratoga Ave., Santa Clara

Date: 05/21/2024
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Submitted By: KC Engineering
Date Submitted: 04/30/2024

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
2-4 @ 16.0’	Mottled Dark Olive Brown Sandy Clay (visual)	107.2	15.6	---	---	---
2-5 @ 21.0’	Olive Gray Sandy Clay (visual)	89.8	24.7	---	---	---
2-6 @ 26.0’	Olive Gray Sandy Clay (visual)	97.0	28.2	---	---	---
2-7 @ 31.0’	Olive Brown Sandy Clay (visual)	103.8	20.8	---	---	---
2-8 @ 36.0’	Black Sandy Clay (visual)	100.7	23.1	---	---	---
2-9 @ 41.0’	Gray Clay (visual)	106.6	21.6	---	---	---
2-10 @ 46.0’	Olive Brown Clay with Sand (visual)	108.4	20.1	---	---	---
3-1 @ 3.0’	Dark Brown Sandy Clay with Gravel (visual)	103.7	19.2	45	19	26
3-2 @ 6.0’	Dark Brown Clay (visual)	100.8	24.6	---	---	---
3-3 @ 11.0’	Mottled Olive Brown Sandy Clay (visual)	103.9	22.0	---	---	---
3-4 @ 16.0’	Olive and Gray Sandy Clay (visual)	116.6	5.8	---	---	---
3-5 @ 21.0’	Olive and Gray Sandy Gravel (visual)	114.4	8.8	---	---	---

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
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Vacaville, California 95688
(707) 447-4025, fax 447-4143

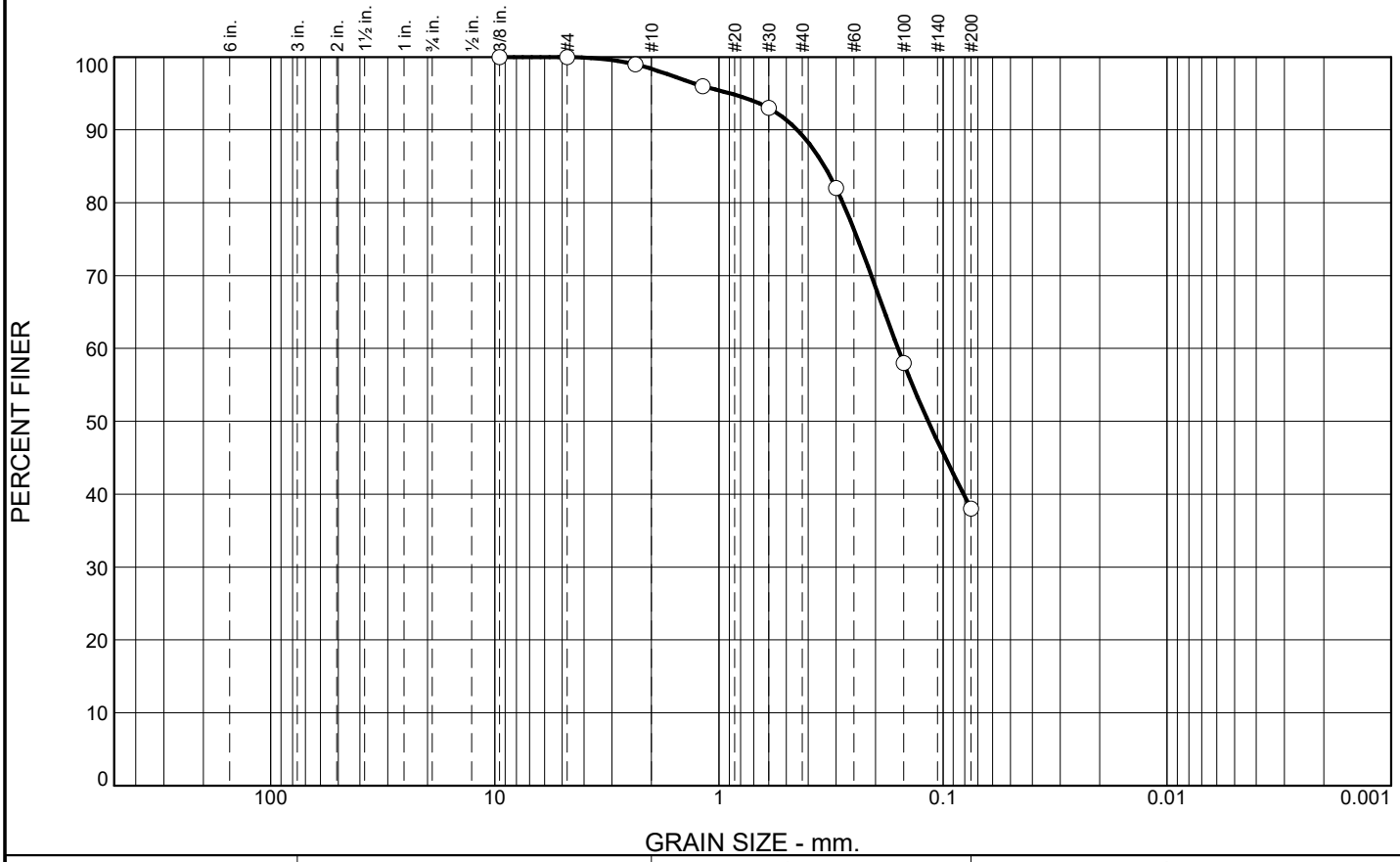
Client:	Demmon Partners 601 University Avenue, Suite 110 Sacramento, CA 95825	Date: Client No.: Report No.: Submitted By:	05/21/2024 VV5781 0300-001 KC Engineering
Project:	Pruneridge Golf Club Multi-Family 400 Saratoga Ave., Santa Clara	Date Submitted: Page No.:	04/30/2024 3 of 3

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937) and Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
3-6 @ 26.0’	Olive Brown Sandy Clay (visual)	101.8	22.3	---	---	---
3-7 @ 31.0’	Gray Clayey Sand (visual)	105.7	21.2	---	---	---
3-8 @ 36.0’	Gray Clayey Sand (visual)	104.6	21.6	---	---	---
3-10 @ 46.0’	Gray Clay (visual)	102.8	24.6	---	---	---

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	2	9	51	38	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	99		
#16	96		
#30	93		
#50	82		
#100	58		
#200	38		

* (no specification provided)

Material Description

Light Olive Brown Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4472 D_{85}= 0.3382 D_{60}= 0.1589
D_{50}= 0.1165 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

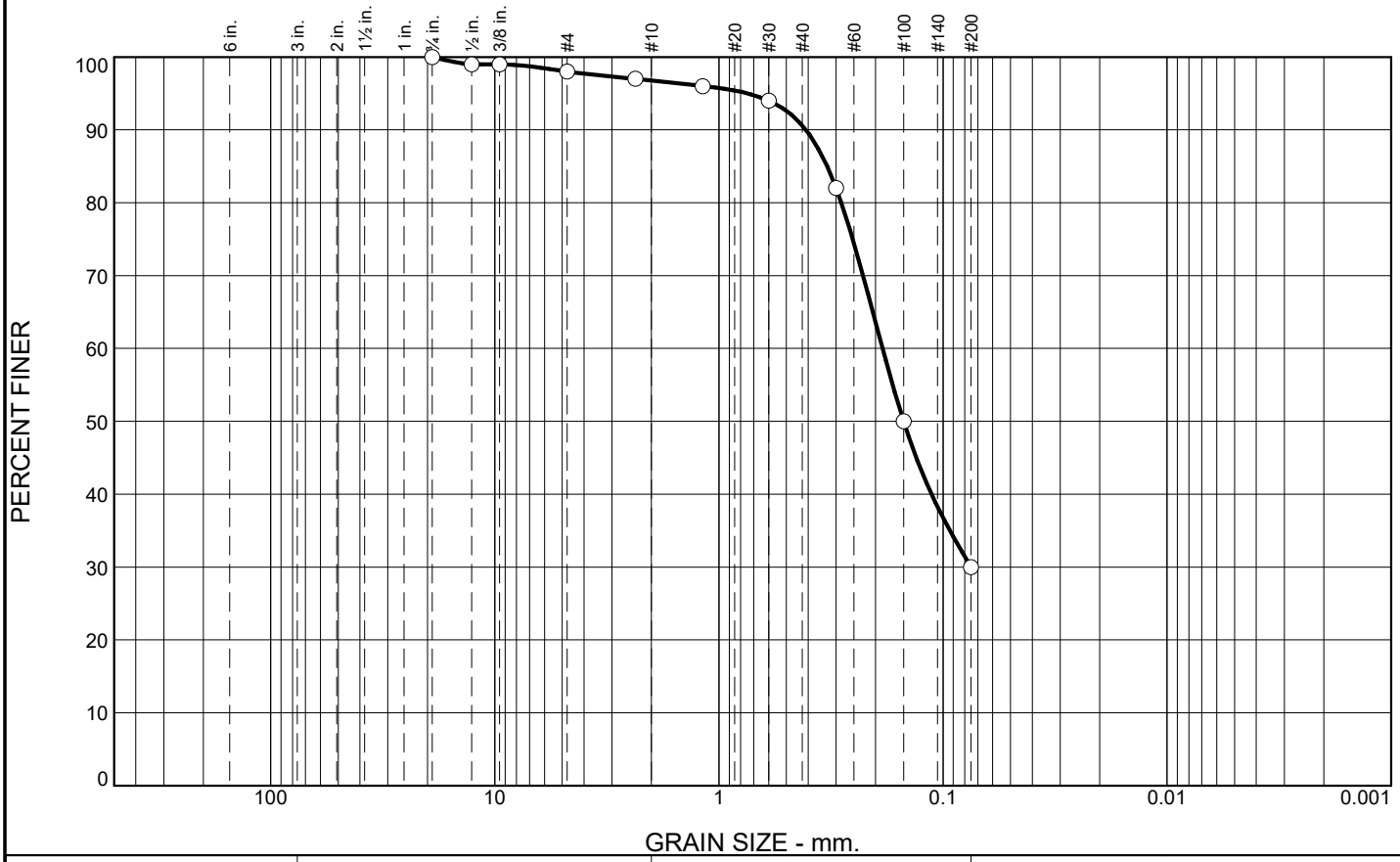
Location: 1-5
Sample Number: 5 **Depth:** 21.0' **Date:** 05/21/2024

MATERIALS TESTING M T I INCORPORATED

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-002

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	3	6	61	30	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	99		
3/8"	99		
#4	98		
#8	97		
#16	96		
#30	94		
#50	82		
#100	50		
#200	30		

* (no specification provided)

Material Description

Light Olive Brown Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4094 D_{85}= 0.3291 D_{60}= 0.1862
D_{50}= 0.1500 D_{30}= 0.0750 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

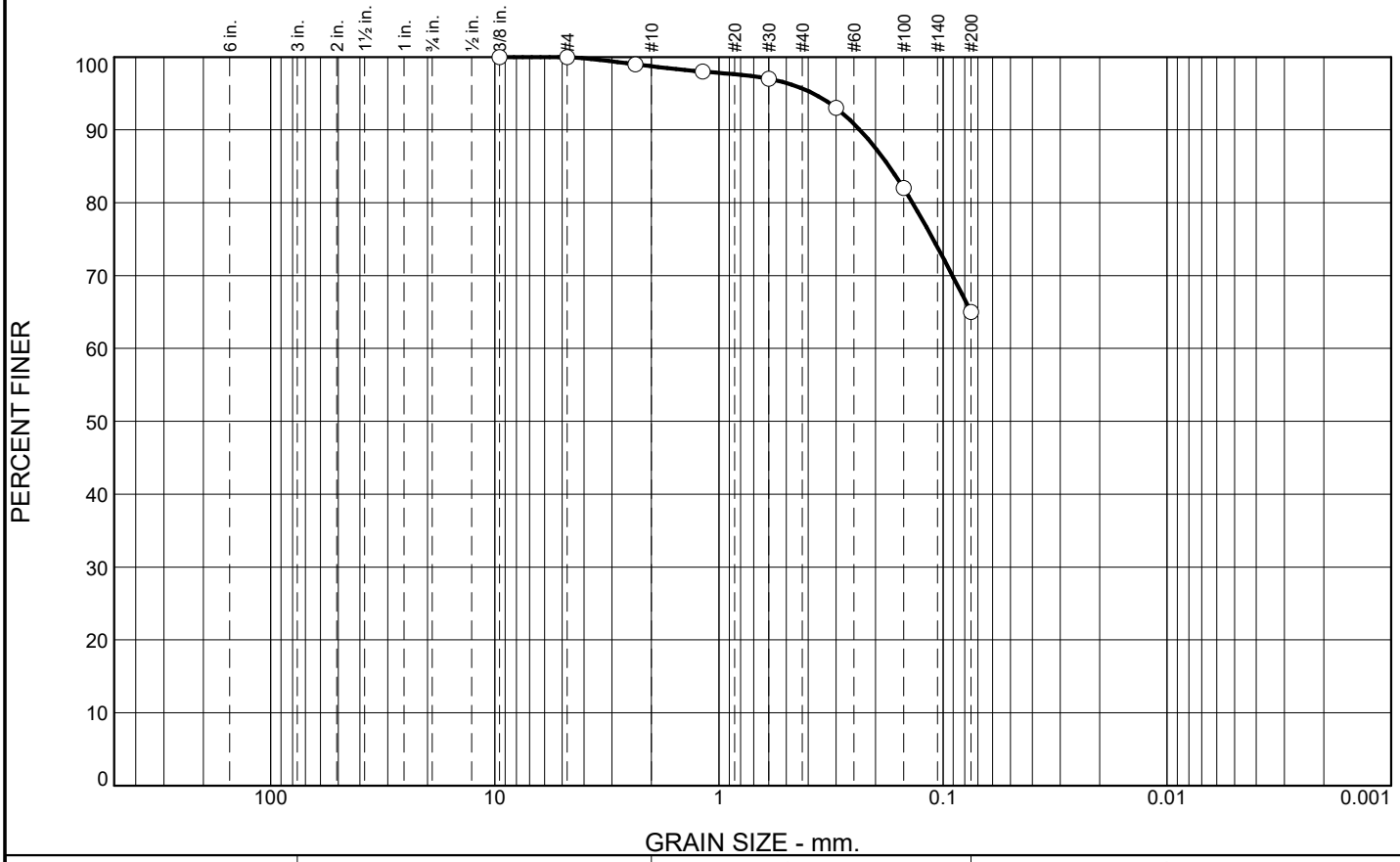
Material tested in accordance with ASTM D6913.

Location: 1-6
Sample Number: 6 **Depth:** 26.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-003

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	1	3	31	65	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	99		
#16	98		
#30	97		
#50	93		
#100	82		
#200	65		

* (no specification provided)

Material Description

Gray Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2351 D_{85}= 0.1742 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

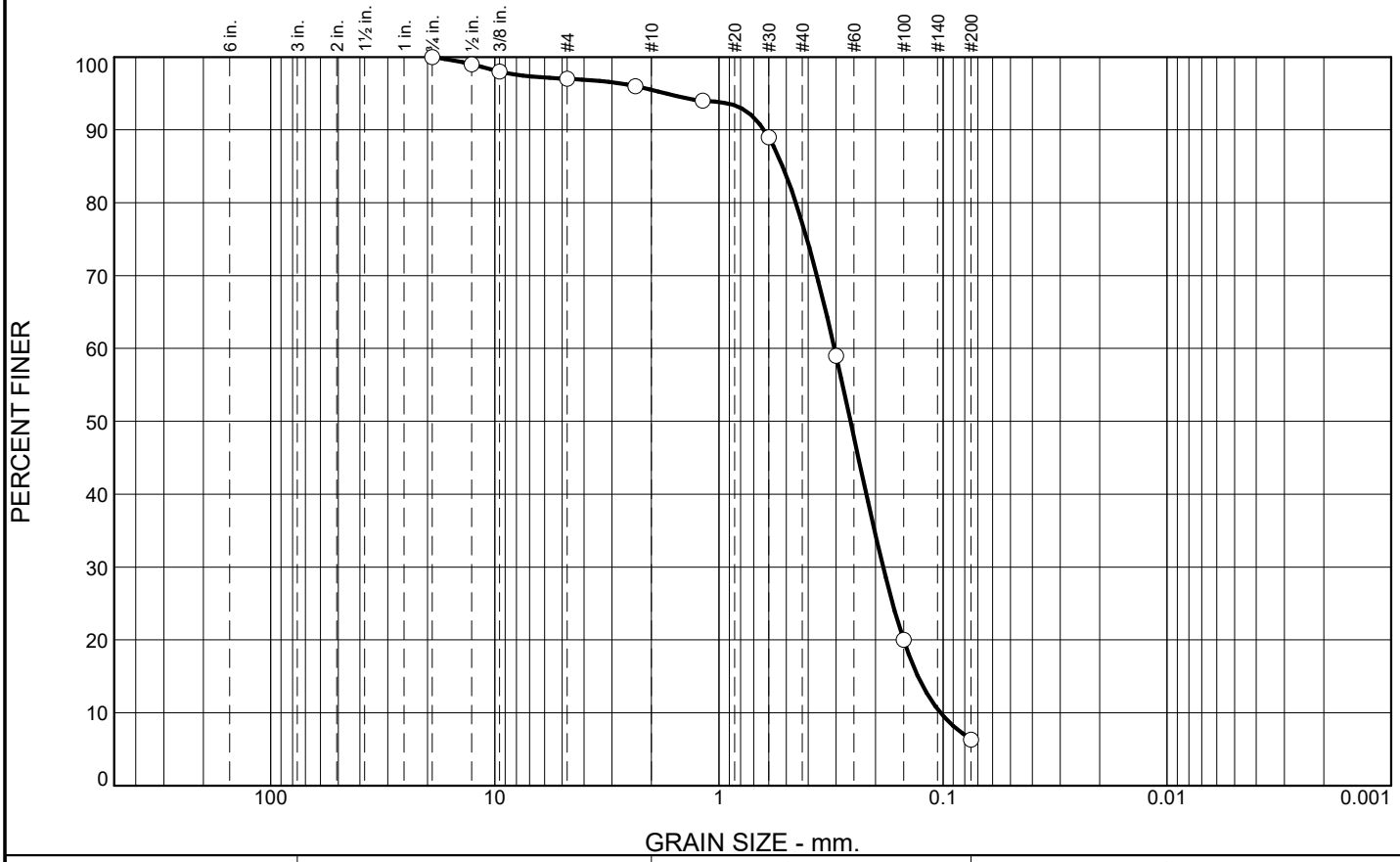
Material tested in accordance with ASTM D6913.

Location: 1-8
Sample Number: 8 **Depth:** 36.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-004

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	5	18	71	6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	99		
3/8"	98		
#4	97		
#8	96		
#16	94		
#30	89		
#50	59		
#100	20		
#200	6.3		

* (no specification provided)

Material Description

Gray Sand with Silt (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6291 D_{85}= 0.5196 D_{60}= 0.3052
D_{50}= 0.2587 D_{30}= 0.1850 D_{15}= 0.1294
D_{10}= 0.1027 C_u= 2.97 C_c= 1.09

Classification

USCS= SP-SM AASHTO=

Remarks

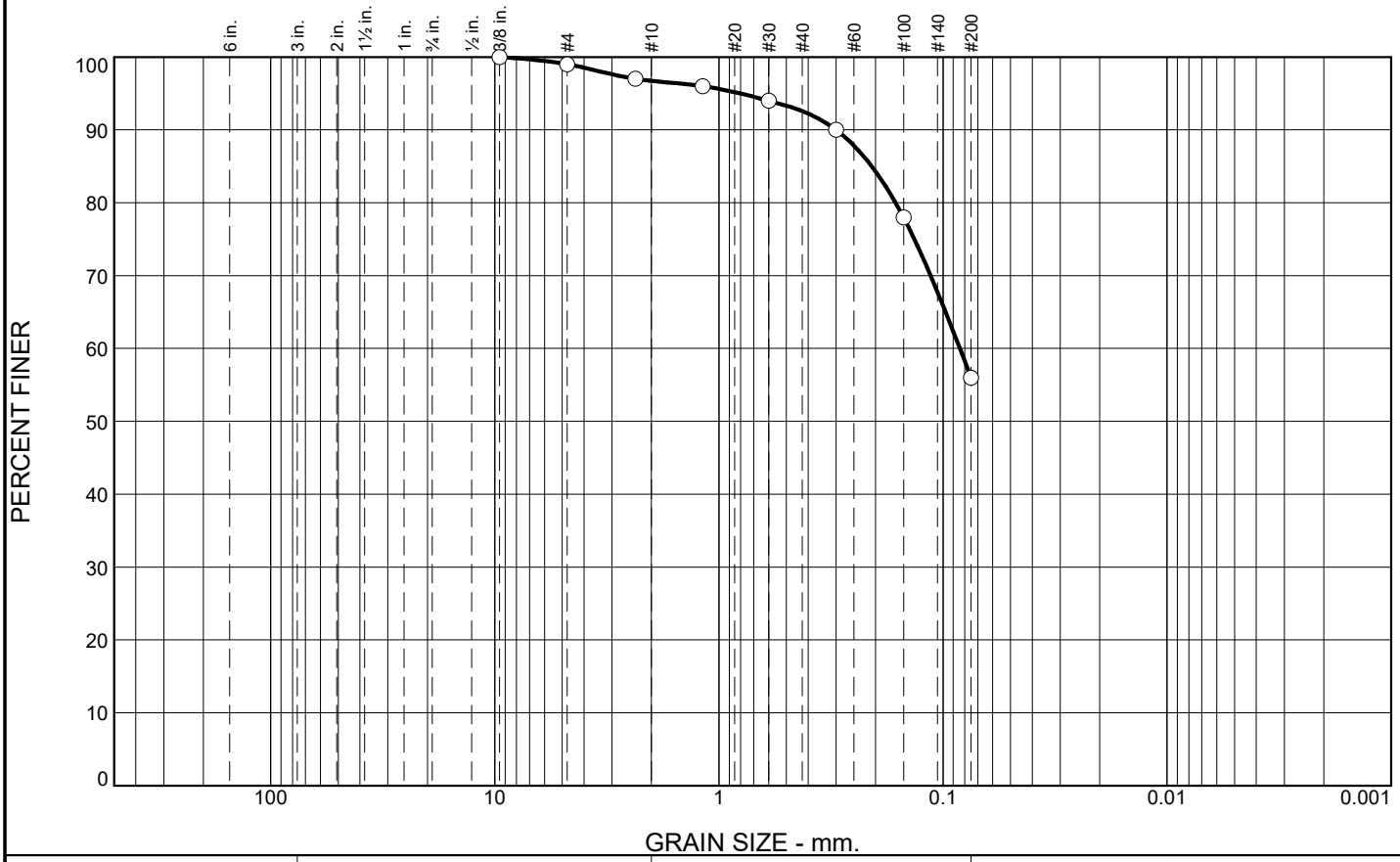
Material tested in accordance with ASTM D6913.

Location: 1-9
Sample Number: 9 **Depth:** 41.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-005

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	3	4	37	56	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	99		
#8	97		
#16	96		
#30	94		
#50	90		
#100	78		
#200	56		

* (no specification provided)

Material Description

Light Olive Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3000 D_{85}= 0.2079 D_{60}= 0.0842
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

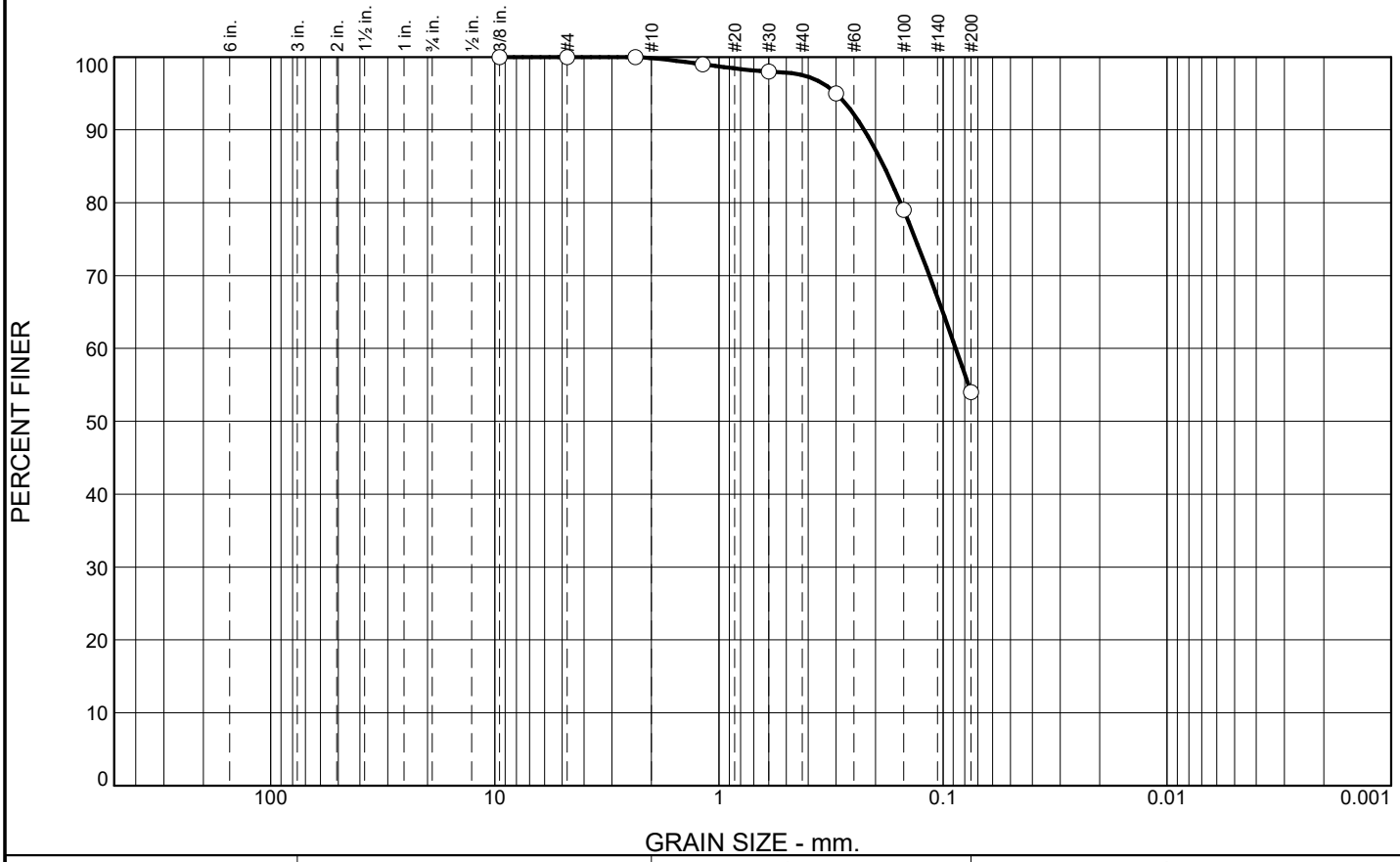
Location: 1-11
Sample Number: 11 **Depth:** 46.5-48.0' **Date:** 05/21/2024

MATERIALS TESTING M T I INCORPORATED

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-006

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	0	2	44	54	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	100		
#16	99		
#30	98		
#50	95		
#100	79		
#200	54		

* (no specification provided)

Material Description

Olive Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2249 D_{85}= 0.1837 D_{60}= 0.0878
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

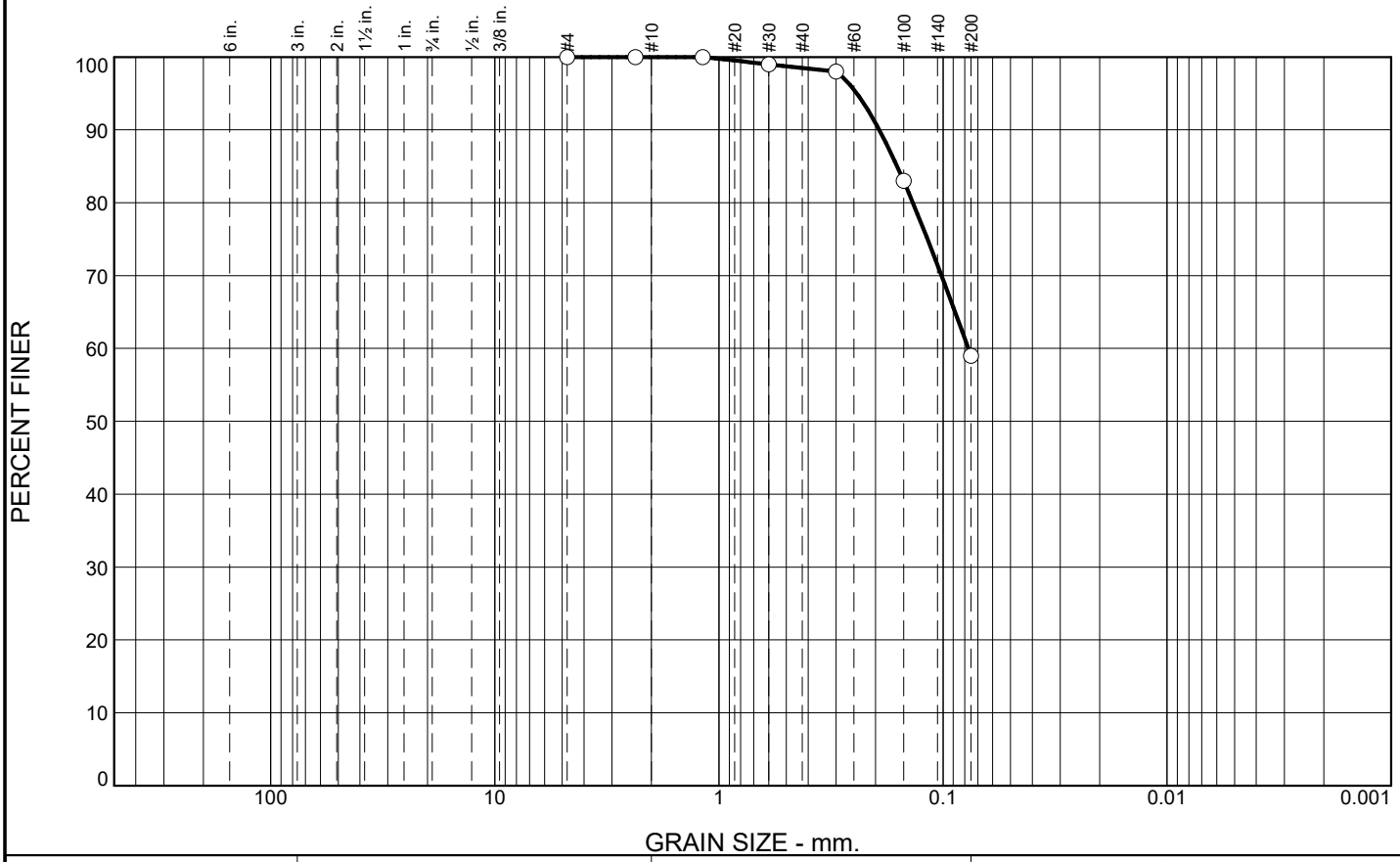
Location: 2-7
Sample Number: 19 **Depth:** 31.0' **Date:** 05/21/2024

MATERIALS TESTING M T I INCORPORATED

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-007

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	0	2	39	59	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#8	100		
#16	100		
#30	99		
#50	98		
#100	83		
#200	59		

* (no specification provided)

Material Description

Black Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1926 D_{85}= 0.1603 D_{60}= 0.0771
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

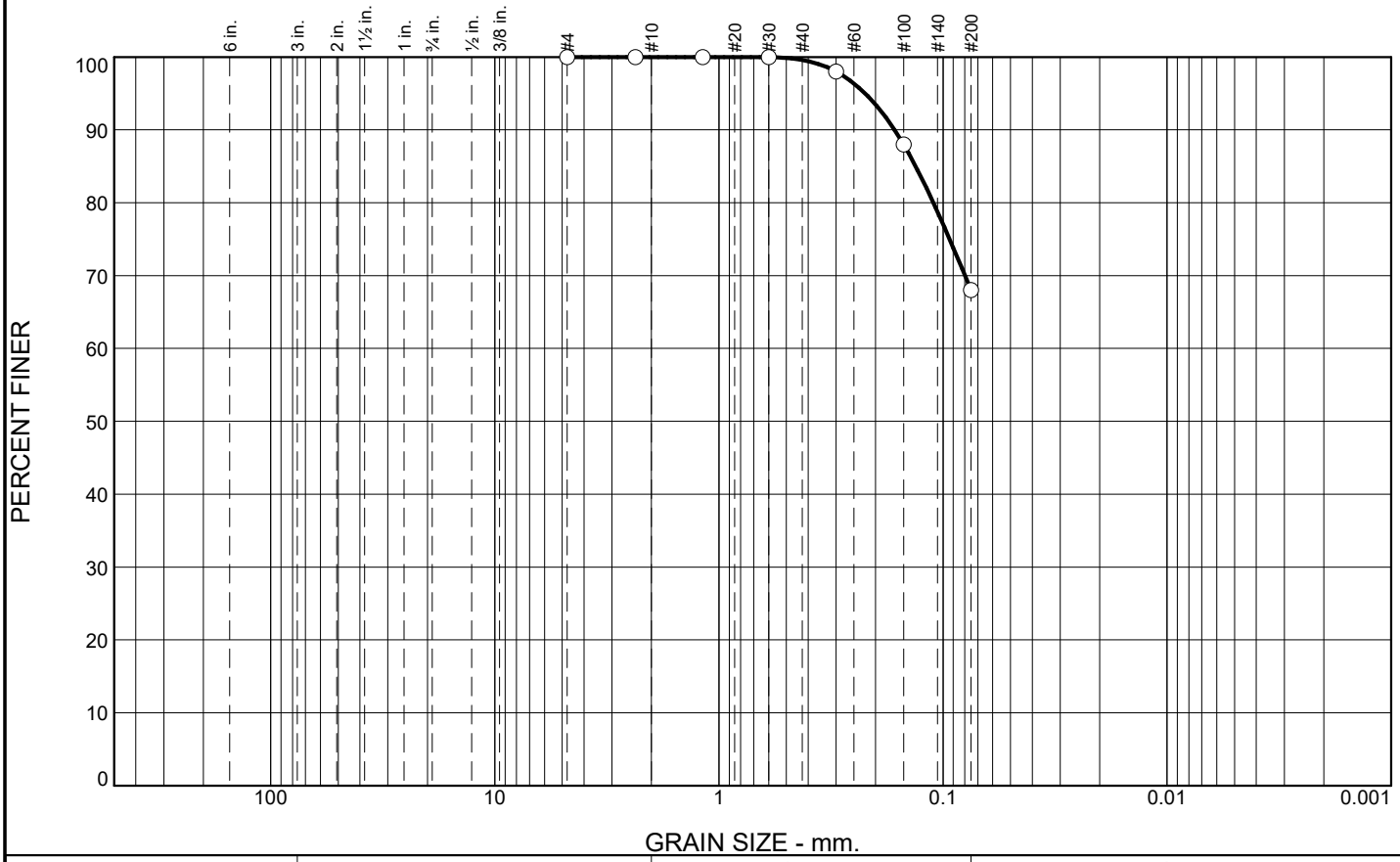
Material tested in accordance with ASTM D6913.

Location: 2-8
Sample Number: 20 **Depth:** 36.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-008

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	0	0	32	68	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100		
#8	100		
#16	100		
#30	100		
#50	98		
#100	88		
#200	68		

* (no specification provided)

Material Description

Olive Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1647 D_{85}= 0.1325 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

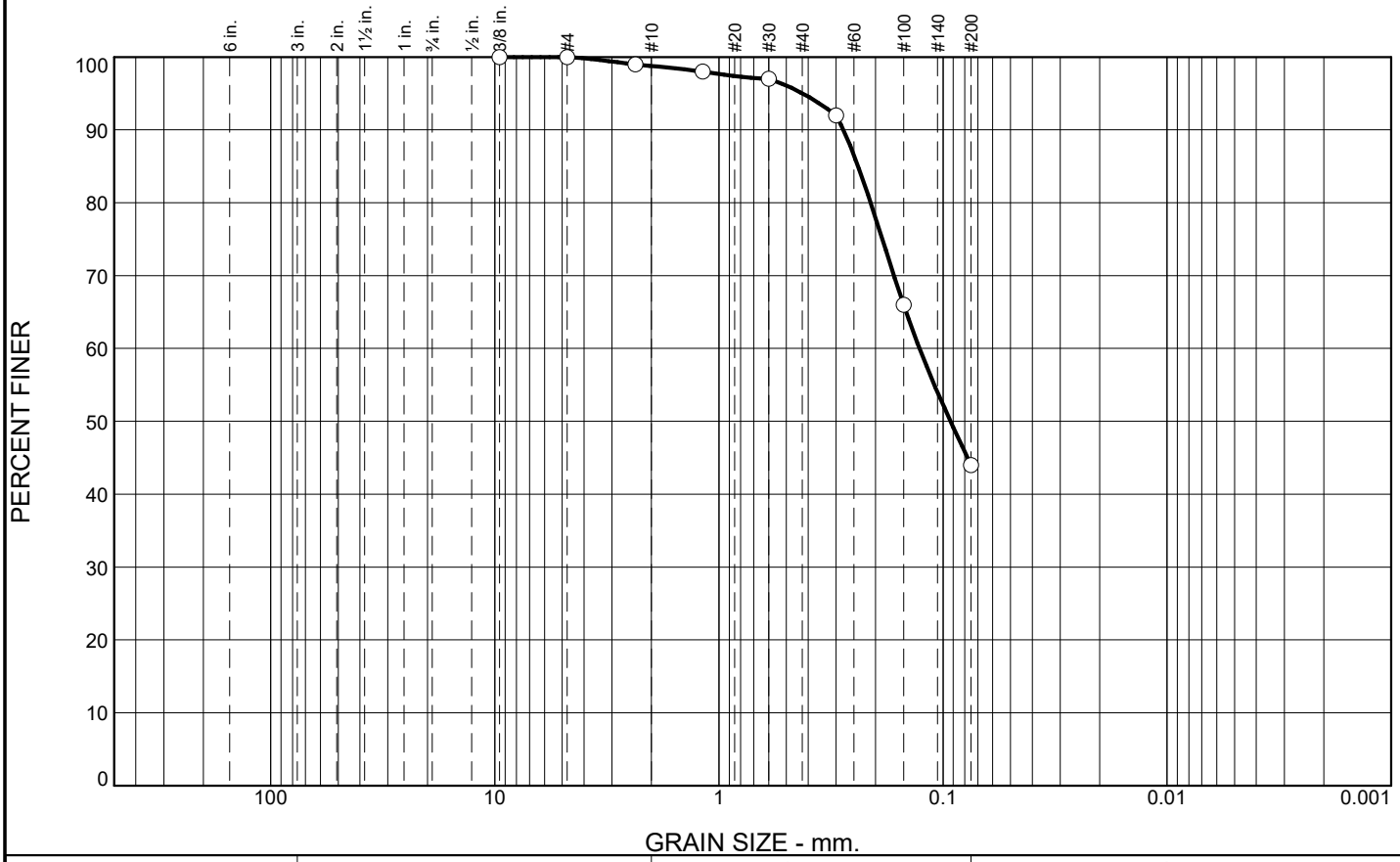
Material tested in accordance with ASTM D6913.

Location: 3-6
Sample Number: 28 **Depth:** 26.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-009

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	1	4	51	44	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	99		
#16	98		
#30	97		
#50	92		
#100	66		
#200	44		

* (no specification provided)

Material Description

Gray Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2782 D_{85}= 0.2393 D_{60}= 0.1273
D_{50}= 0.0926 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

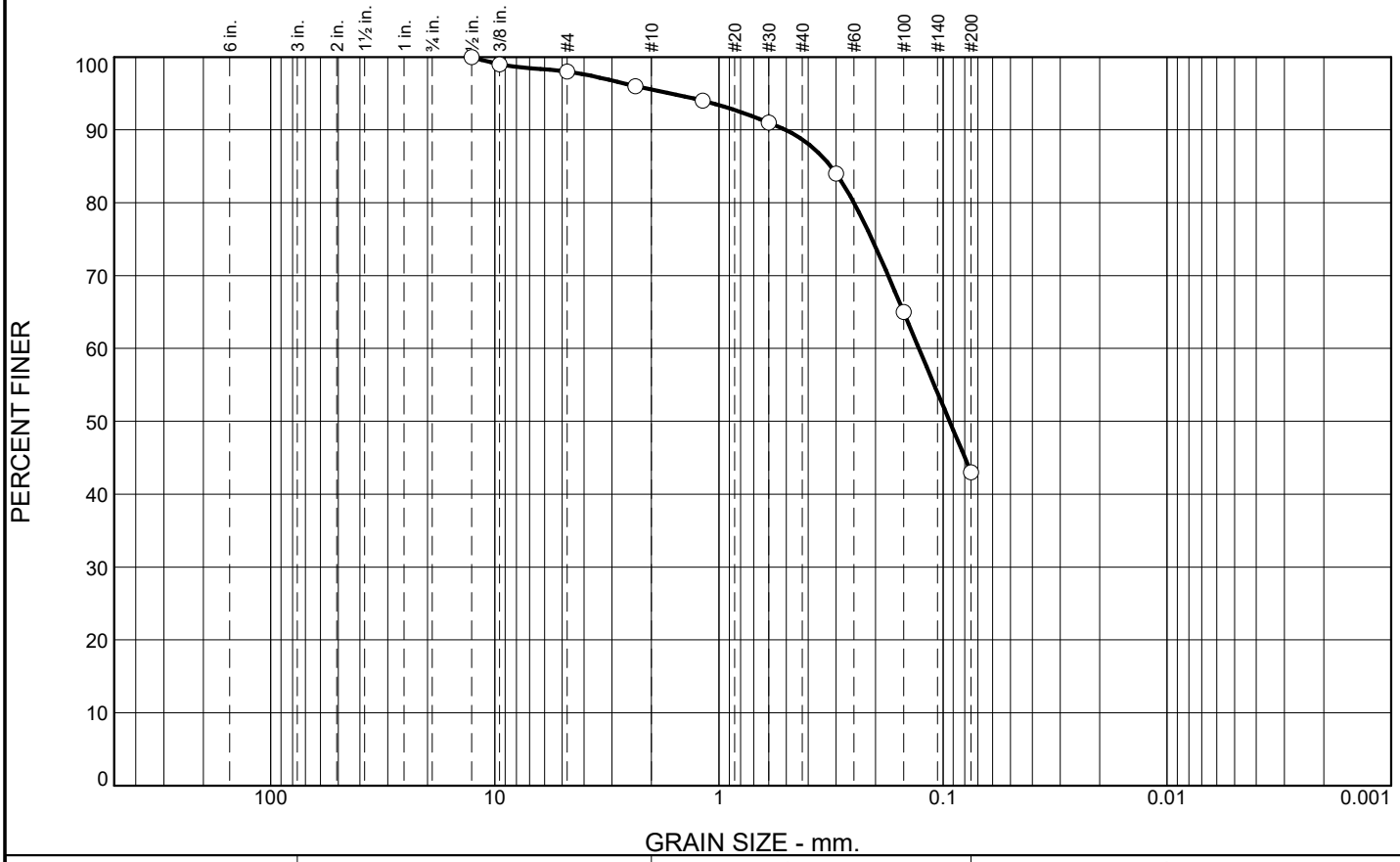
Material tested in accordance with ASTM D6913.

Location: 3-7
Sample Number: 29 **Depth:** 31.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-010

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	4	7	46	43	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	99		
#4	98		
#8	96		
#16	94		
#30	91		
#50	84		
#100	65		
#200	43		

* (no specification provided)

Material Description

Gray Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5045 D_{85}= 0.3173 D_{60}= 0.1281
D_{50}= 0.0935 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

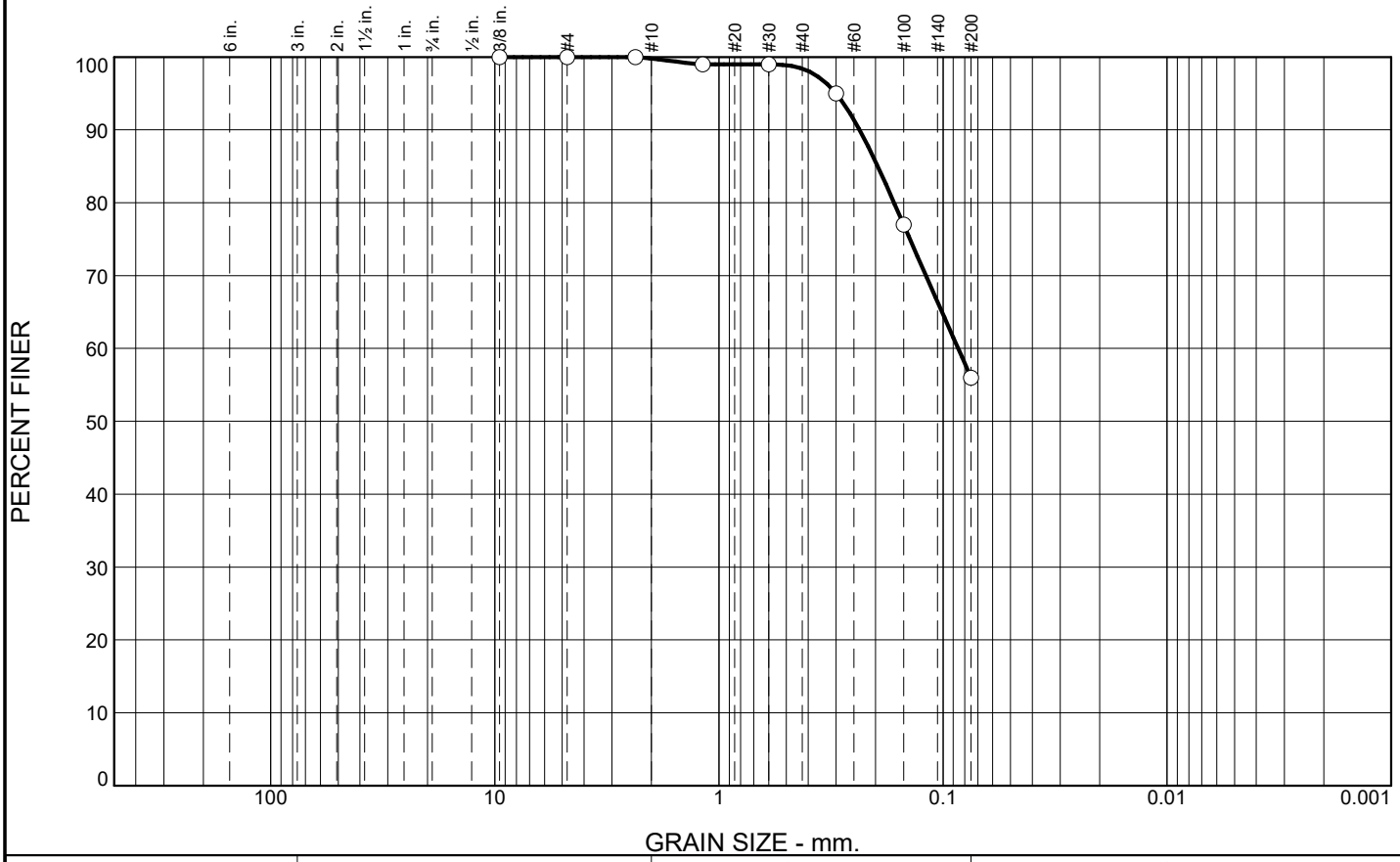
Material tested in accordance with ASTM D6913.

Location: 3-8
Sample Number: 30 **Depth:** 36.0' **Date:** 05/21/2024

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-011

Tested By: Jack Bianchin

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	0	2	42	56	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	100		
#16	99		
#30	99		
#50	95		
#100	77		
#200	56		

* (no specification provided)

Material Description

Gray Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2357 D_{85}= 0.1957 D_{60}= 0.0856
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

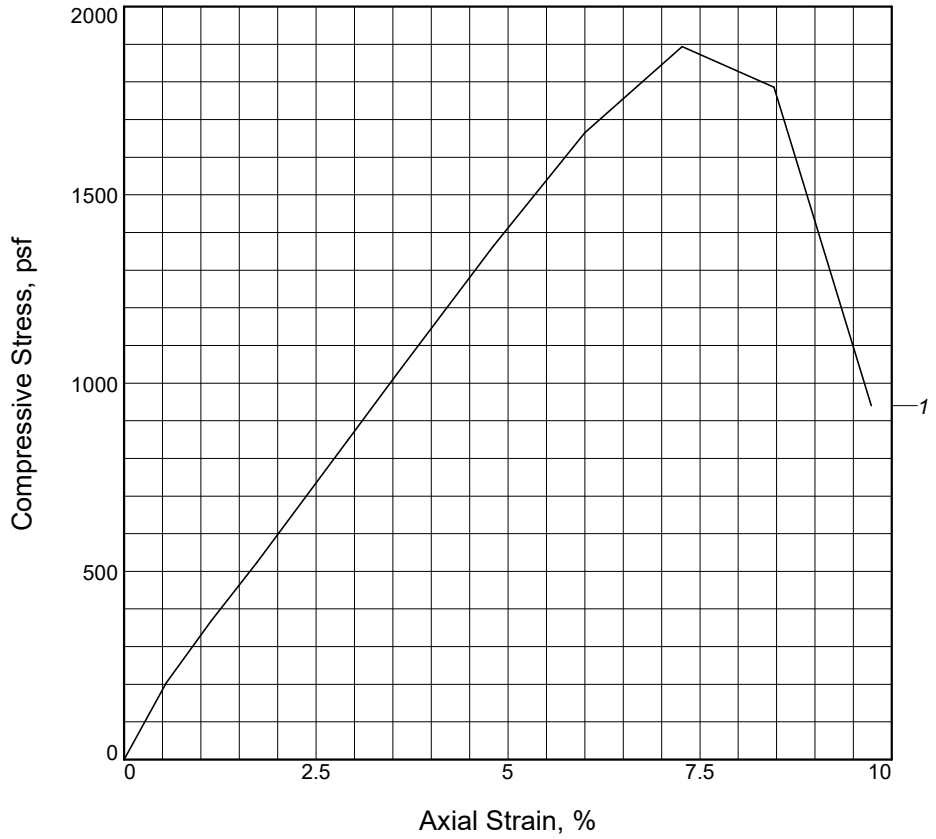
Location: 3-9
Sample Number: 31 **Depth:** 40.0-41.5' **Date:** 05/21/2024

MATERIALS TESTING M T I INCORPORATED

Client: Demmon Partners
Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Project No: VV5781 **Figure** 0300-012

Tested By: Jack Bianchin

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1894			
Undrained shear strength, psf	947			
Failure strain, %	7.3			
Strain rate, in./min.	0.075			
Water content, %	28.2			
Wet density, pcf	124.4			
Dry density, pcf	97.0			
Saturation, %	94.0			
Void ratio	0.8720			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Olive Gray Sandy Clay (visual)

LL =	**PL =**	**PI =**	**GS=** 2.91	**Type:** Tube

Project No.: VV5781
Date Sampled: 05/21/2024

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Columnar

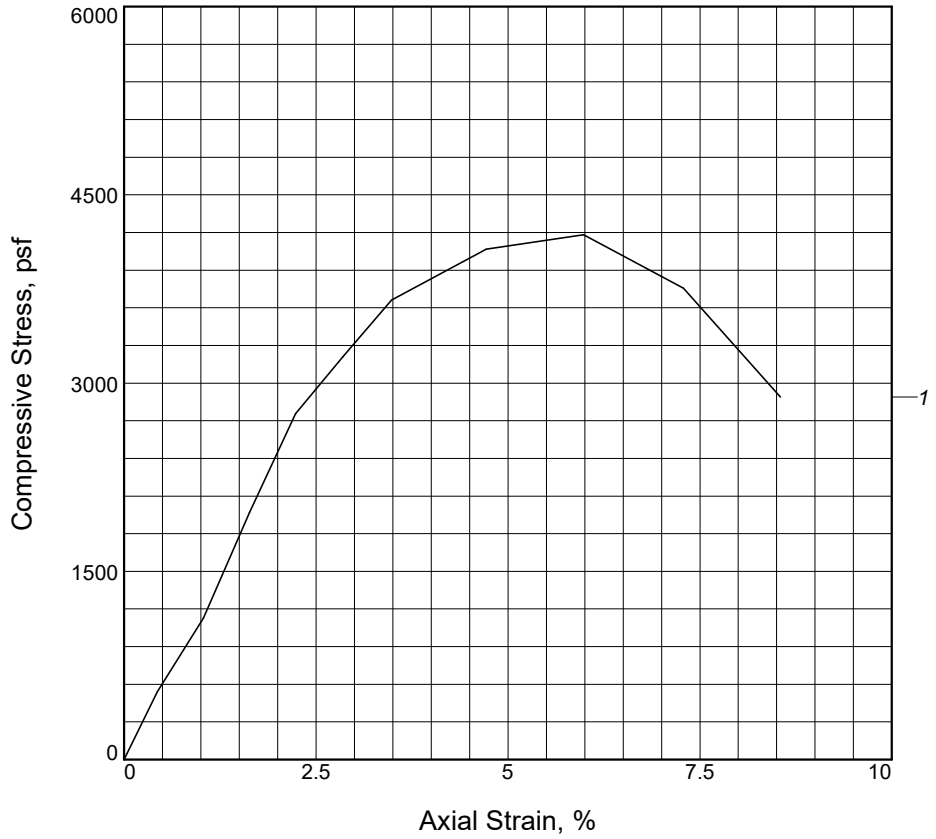
Figure 0300-013

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 2-6
Sample Number: 18 **Depth:** 26.0'

Tested By: Jack Bianchin

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	4182			
Undrained shear strength, psf	2091			
Failure strain, %	6.0			
Strain rate, in./min.	0.075			
Water content, %	22.0			
Wet density, pcf	126.7			
Dry density, pcf	103.9			
Saturation, %	100.9			
Void ratio	0.5683			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Mottled Olive Brown Sandy Clay (visual)

LL =	**PL =**	**PI =**	**GS=** 2.61	**Type:** Tube

Project No.: VV5781
Date Sampled: 05/22/2024

Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Bulge and Columnar

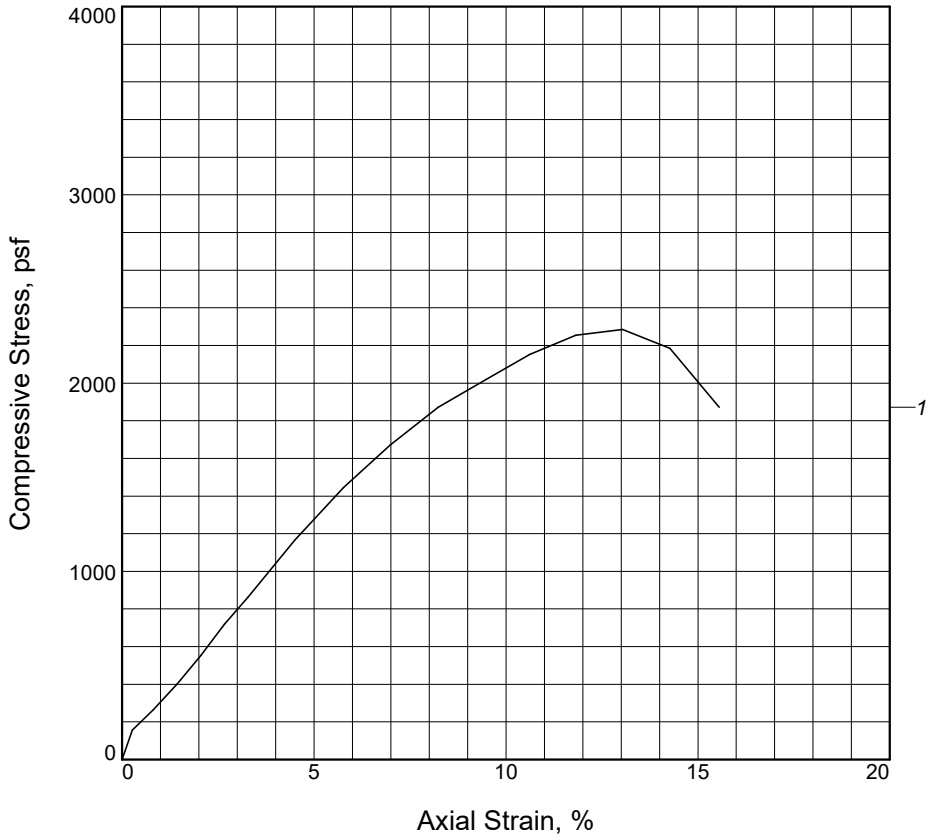
Figure 0300-014

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 3-3
Sample Number: 25 **Depth:** 11.0'

Tested By: John Hubbard

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	2284			
Undrained shear strength, psf	1142			
Failure strain, %	13.0			
Strain rate, in./min.	0.075			
Water content, %	22.3			
Wet density, pcf	124.5			
Dry density, pcf	101.8			
Saturation, %	97.6			
Void ratio	0.5950			
Specimen diameter, in.	2.41			
Specimen height, in.	6.00			
Height/diameter ratio	2.49			

Description: Olive Brown Sandy Clay (visual)

LL =	**PL =**	**PI =**	**GS=** 2.60	**Type:** Tube

Project No.: VV5781
Date Sampled: 05/22/2024

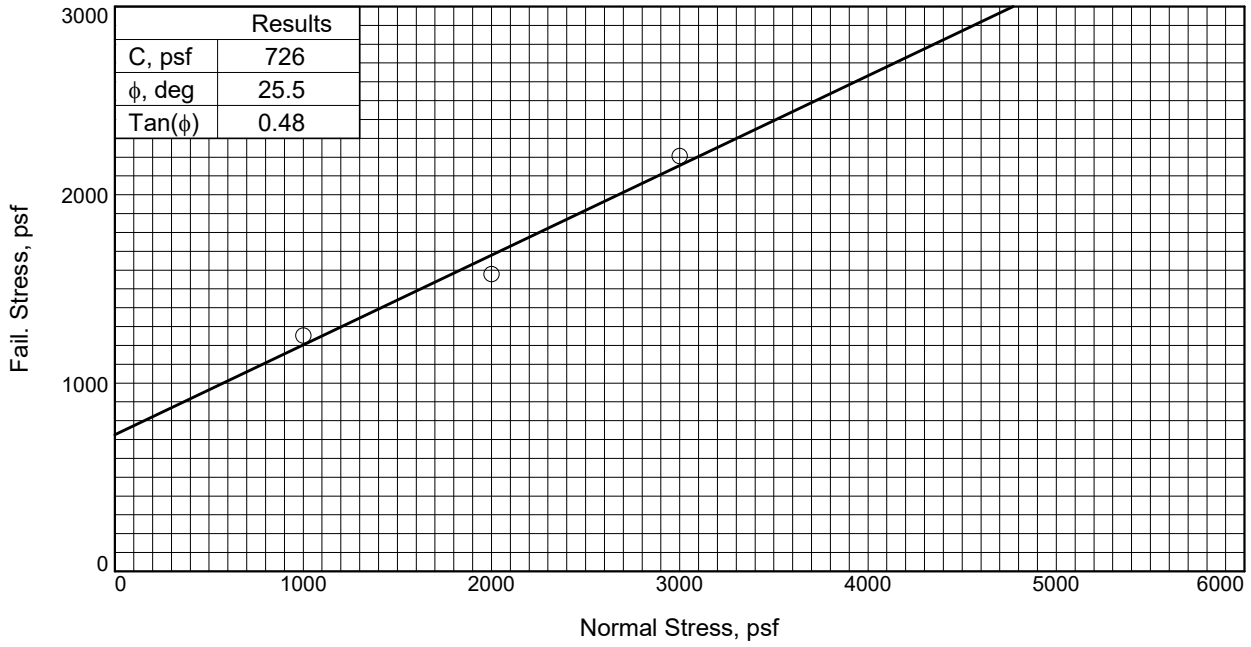
Remarks:
Material tested in accordance with ASTM D2166.
Type of Failure - Bulge

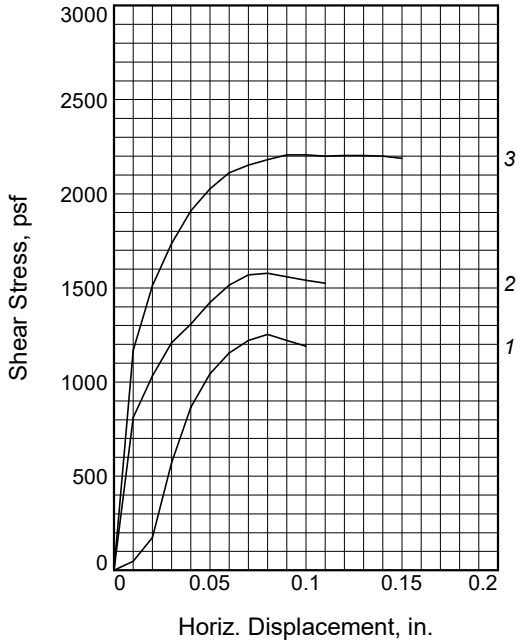
Figure 0300-015

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 3-6
Sample Number: 28 **Depth:** 26.0'

Tested By: John Hubbard





Sample No.		1	2	3
Initial	Water Content, %	17.5	17.5	17.5
	Dry Density, pcf	111.2	101.5	106.4
	Saturation, %	91.1	71.3	80.5
	Void Ratio	0.5216	0.6665	0.5908
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.0	21.9	22.3
	Dry Density, pcf	113.3	108.3	107.0
	Saturation, %	132.0	105.7	103.9
	Void Ratio	0.4927	0.5619	0.5815
	Diameter, in.	2.41	2.41	2.41
	Height, in.	0.98	0.94	0.99
Normal Stress, psf		1000	2000	3000
Fail. Stress, psf		1253	1578	2207
Displacement, in.		0.08	0.08	0.09
Ult. Stress, psf				
Displacement, in.				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Tube
Description: Light Olive Brown Sandy Clay (visual)

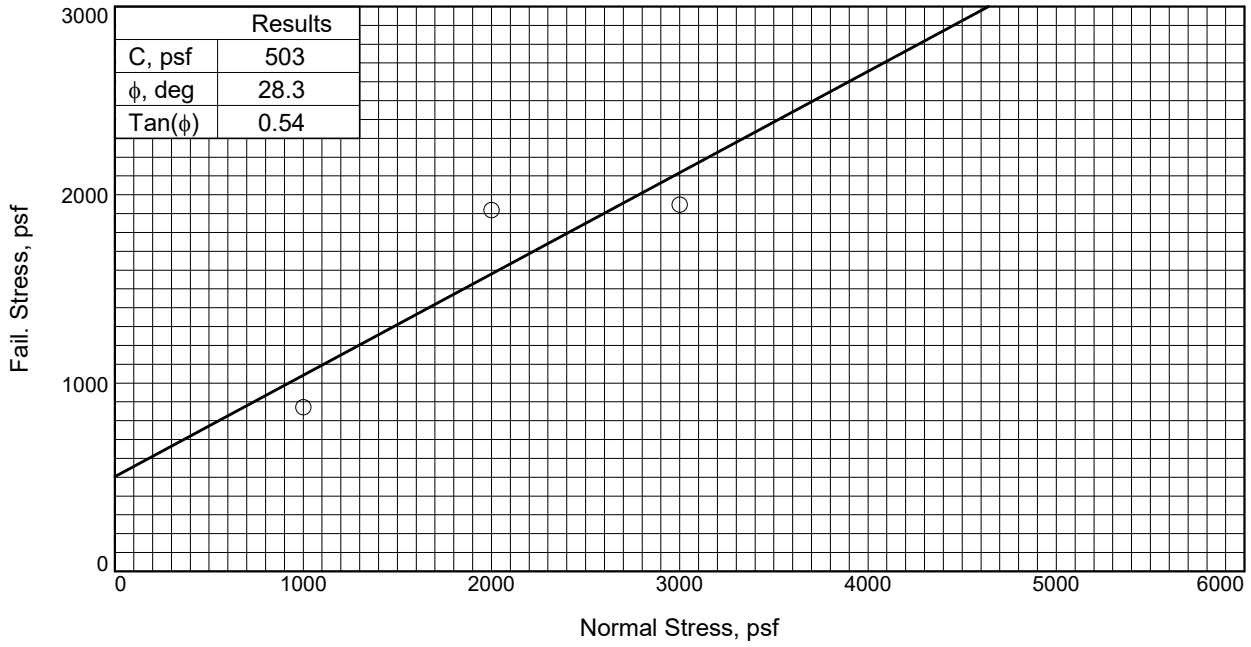
Specific Gravity= 2.71
Remarks: Material tested in accordance with ASTM D3080.

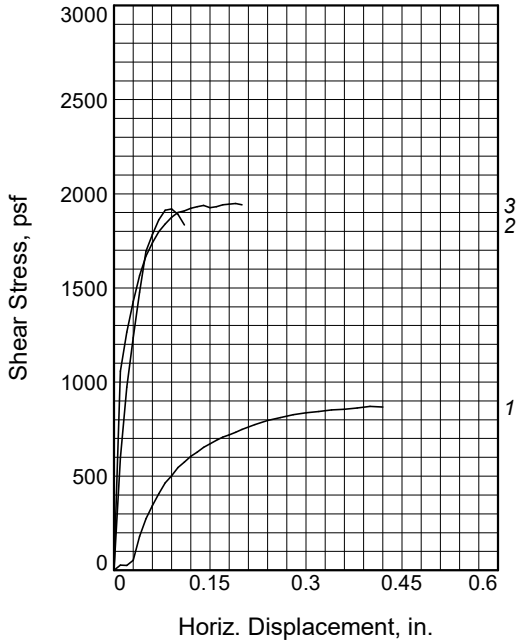
Figure 0300-016

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 1-4
Sample Number: 4 **Depth:** 16.0'
Proj. No.: VV5781 **Date Sampled:** 05/21/2024

Tested By: John Hubbard





Sample No.		1	2	3
Initial	Water Content, %	22.6	22.6	22.6
	Dry Density, pcf	71.1	97.6	87.1
	Saturation, %	44.8	85.3	66.1
	Void Ratio	1.3455	0.7072	0.9130
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	36.3	27.8	30.9
	Dry Density, pcf	78.9	99.2	90.5
	Saturation, %	87.0	109.1	98.1
	Void Ratio	1.1135	0.6809	0.8413
	Diameter, in.	2.41	2.41	2.41
	Height, in.	0.90	0.98	0.96
Normal Stress, psf		1000	2000	3000
Fail. Stress, psf		871	1919	1948
Displacement, in.		0.40	0.09	0.19
Ult. Stress, psf				
Displacement, in.				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Tube
Description: Dark Brown Sandy Clay with Organics (visual)

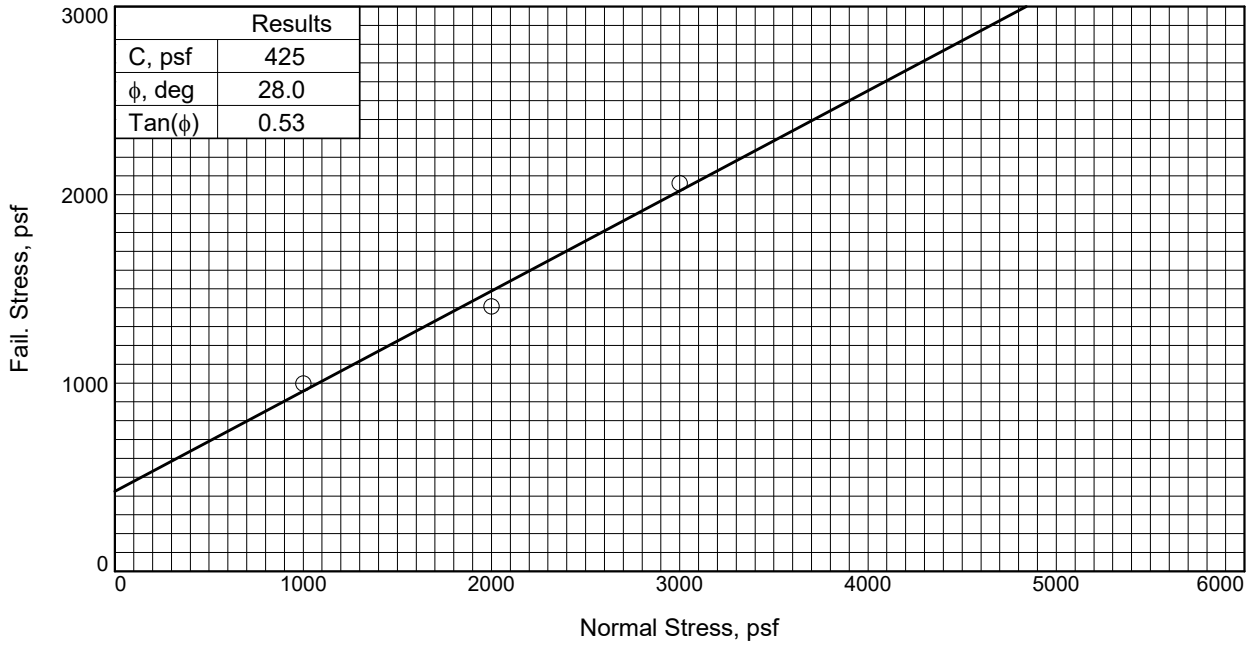
Specific Gravity= 2.67
Remarks: Material tested in accordance with ASTM D3080.

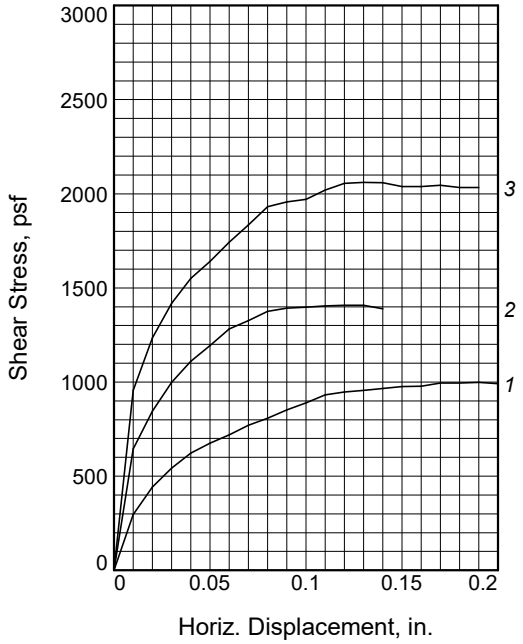
Figure 0300-017

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 2-1
Sample Number: 12 **Depth:** 3.0'
Proj. No.: VV5781 **Date Sampled:** 05/21/2024

Tested By: John Hubbard





Sample No.		1	2	3
Initial	Water Content, %	24.7	24.7	24.7
	Dry Density, pcf	96.0	97.2	96.0
	Saturation, %	80.9	83.1	80.9
	Void Ratio	0.8860	0.8625	0.8860
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	25.0	25.6	27.6
	Dry Density, pcf	99.1	101.6	99.0
	Saturation, %	87.8	94.7	96.4
	Void Ratio	0.8262	0.7827	0.8291
	Diameter, in.	2.41	2.41	2.41
	Height, in.	0.97	0.96	0.97
Normal Stress, psf		1000	2000	3000
Fail. Stress, psf		998	1408	2061
Displacement, in.		0.19	0.12	0.13
Ult. Stress, psf				
Displacement, in.				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Tube
Description: Olive Gray Sandy Clay (visual)

Specific Gravity= 2.90
Remarks: Material tested in accordance with ASTM D3080.

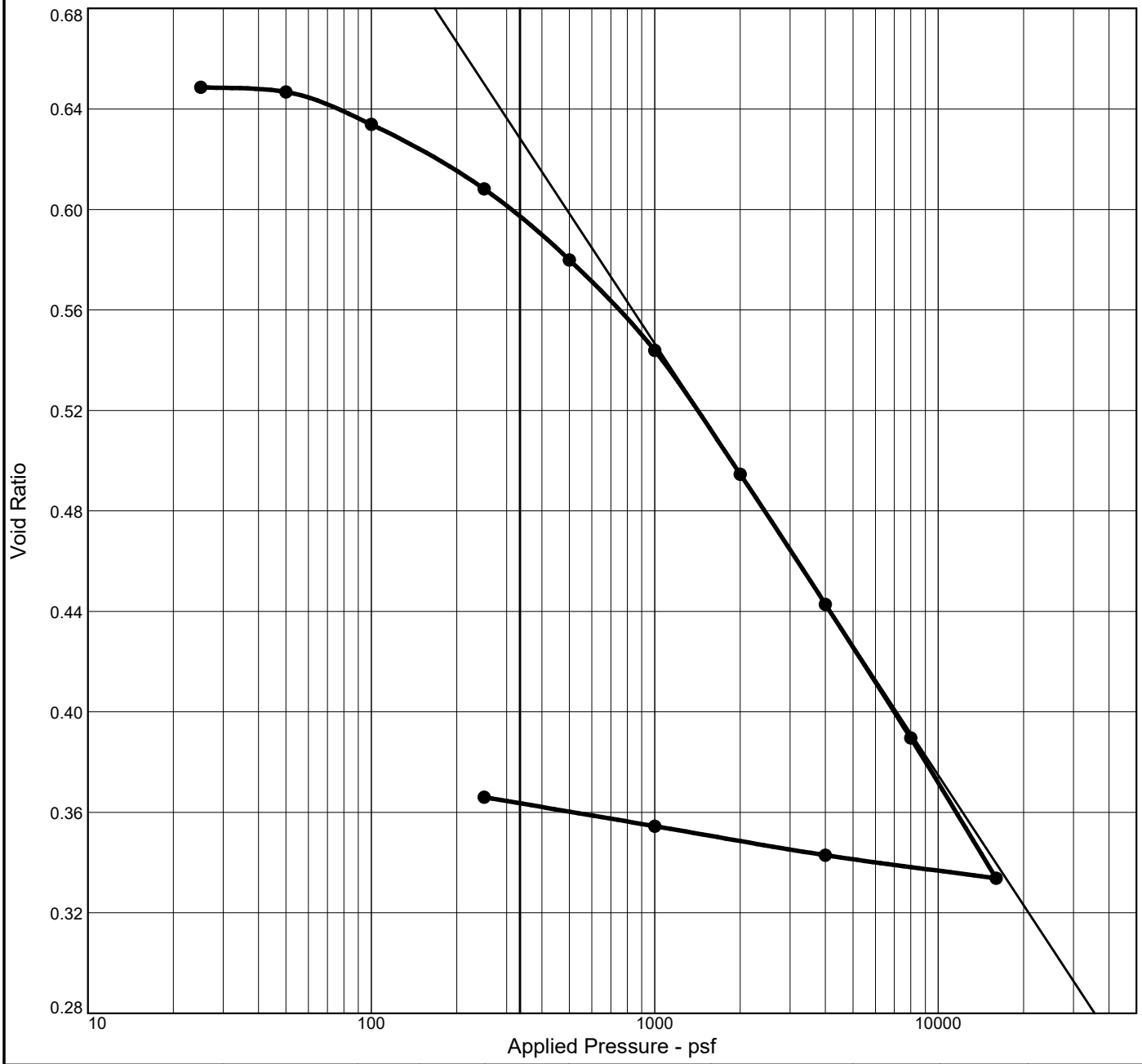
Figure 0300-018

Client: Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA
Location: 2-5
Sample Number: 17 **Depth:** 21.0'
Proj. No.: VV5781 **Date Sampled:** 05/21/2024

Tested By: John Hubbard

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
94.7 %	23.1 %	100.7			2.66	3515	591	0.17	0.02	0.649

MATERIAL DESCRIPTION	USCS	AASHTO
Black Sandy Clay (visual)		

Project No. VV5781 **Client:** Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA

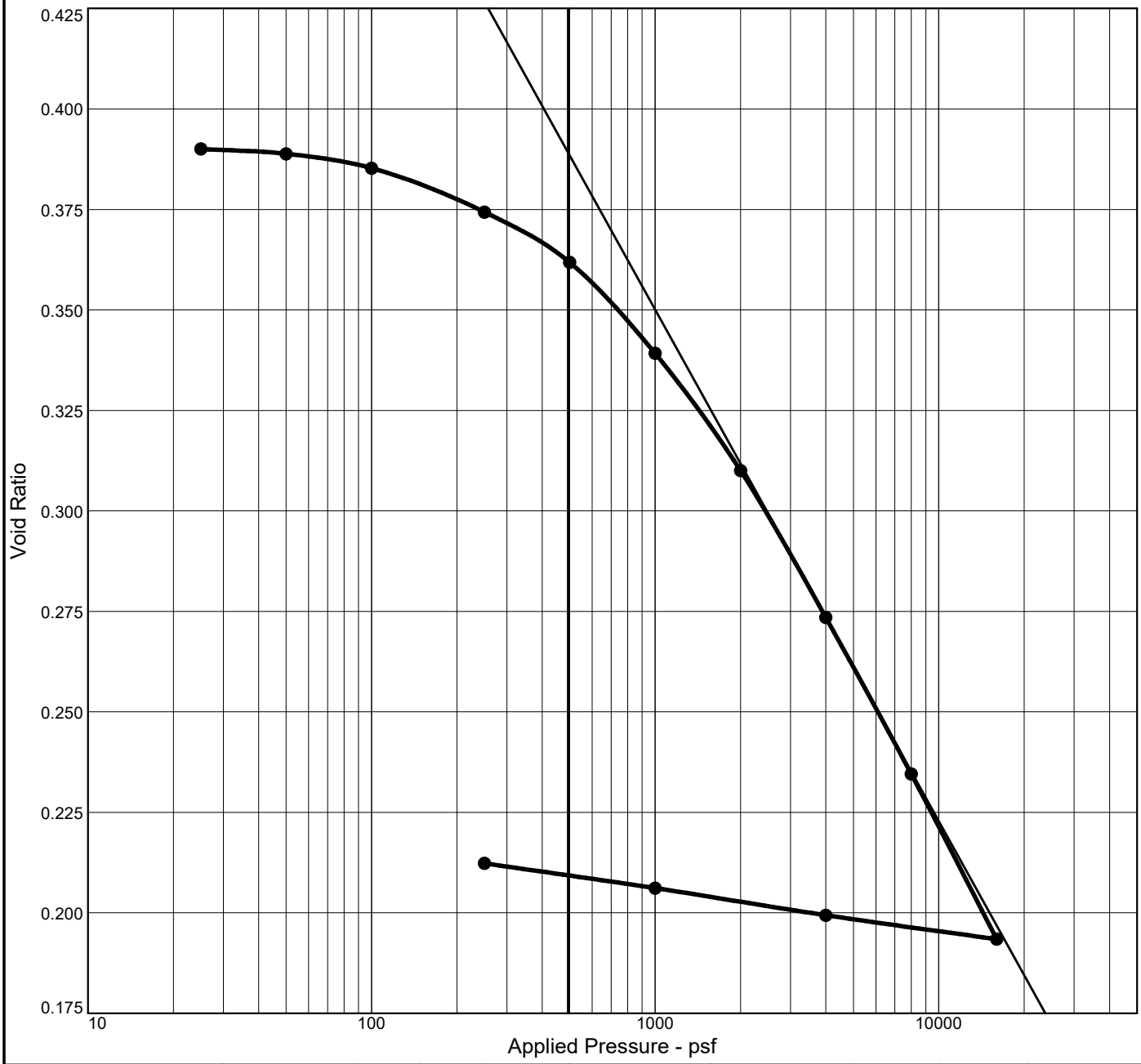
Location: 2-8 **Depth:** 36.0' **Sample Number:** 20

Remarks:
Material tested in accordance with ASTM D2435.

Figure 0300-019

Tested By: John Hubbard

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
					2.74	3495	923	0.13	0.01	0.390

MATERIAL DESCRIPTION	USCS	AASHTO
Gray Clayey Sand (visual)		

Project No. VV5781 **Client:** Demmon Partners

Project: Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara,CA

Location: 3-7 **Depth:** 31.0' **Sample Number:** 29

Remarks:
Material tested in accordance with ASTM D2435.

Figure 0300-020

Tested By: John Hubbard

Materials Testing, Inc.

8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611

865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client:	Demmon Partners 601 University Avenue, Suite 110 Sacramento, CA 95825	Pages:	1 of 1
		Client No:	VV5781
		Report No:	0300-021
		Date:	05/22/2024
Project:	Pruneridge Golf Club Multi-Family 400 Saratoga Ave., Santa Clara	Submitted by:	KC Engineering
		Date Received:	04/30/2024

"R" VALUE TEST REPORT
(AASHTO T 190)

Sample: 13
Description: Dark Brown Sandy Clay (visual)
Location: Site 0-3.0'

SIEVE ANALYSIS

Sieve Size	1-1/2"	1"	3/4"	1/2"	3/8"	#4
As Received (% Pass)	---	---	---	---	---	100
As Used (% Pass)	---	---	---	---	---	100

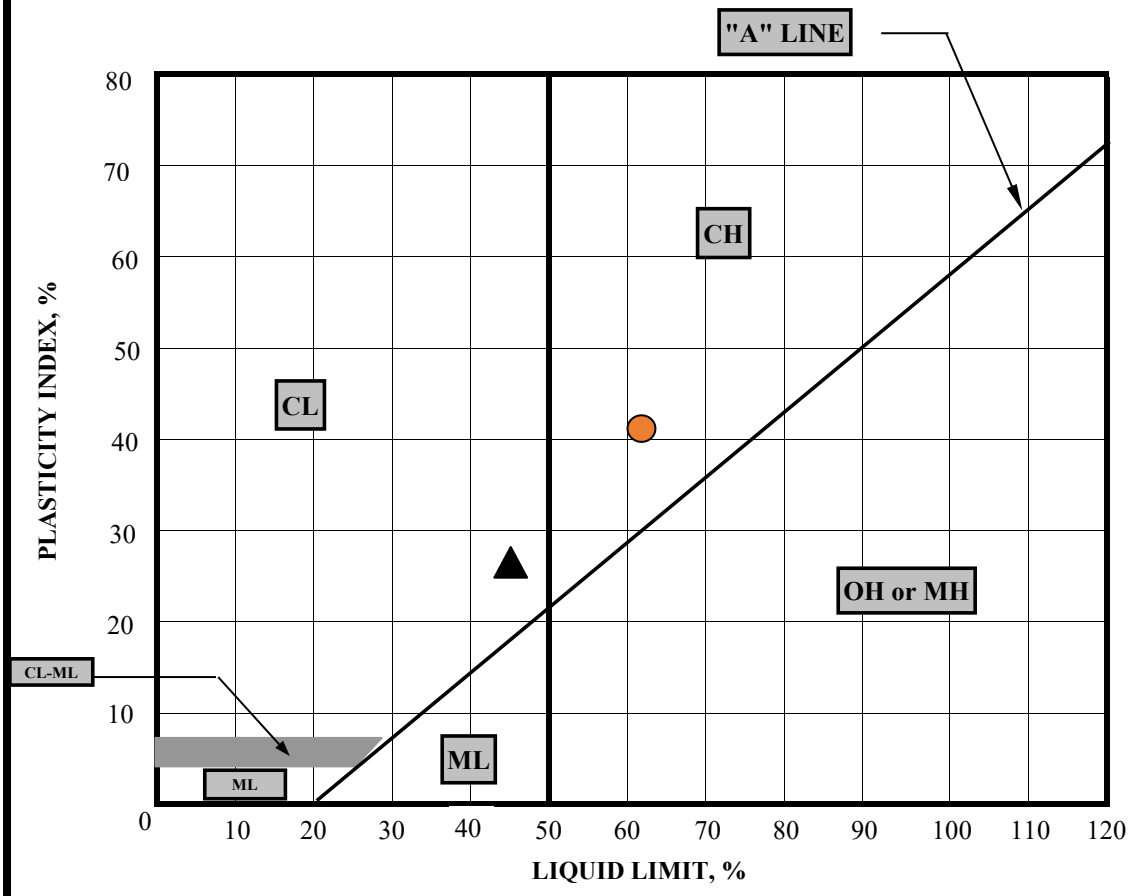
RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	106.3	20.8	669	0	0	29
2	98.8	24.7	325	0	0	12
3	92.6	28.5	228	0	0	6

R-Value @ 300 PSI Exudation Pressure = **10**

Notes:

Tested by Don Kelley.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
▲	3-1	3.0'	N/A	45	19	26	N/A	CL
●	2-2	6.0'	N/A	62	21	41	N/A	CH

Note: Atterberg Limits tested in accordance with ASTM D4318.

Materials Testing, Inc.

PLASTICITY CHART AND DATA

Pruneridge Golf Club Multi-Family
400 Saratoga Ave., Santa Clara

Client No.	Date:	Report No:
VV5781	5/21/2024	0300-022

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 05/08/2024
Date Submitted 05/02/2024

To: David Cymanski
K.C. Engineering
865 Cotting Lane Suite A
Vacaville, CA 95688

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : VV5781 Site ID : SITE 0-20FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 92135-190919.

EVALUATION FOR SOIL CORROSION

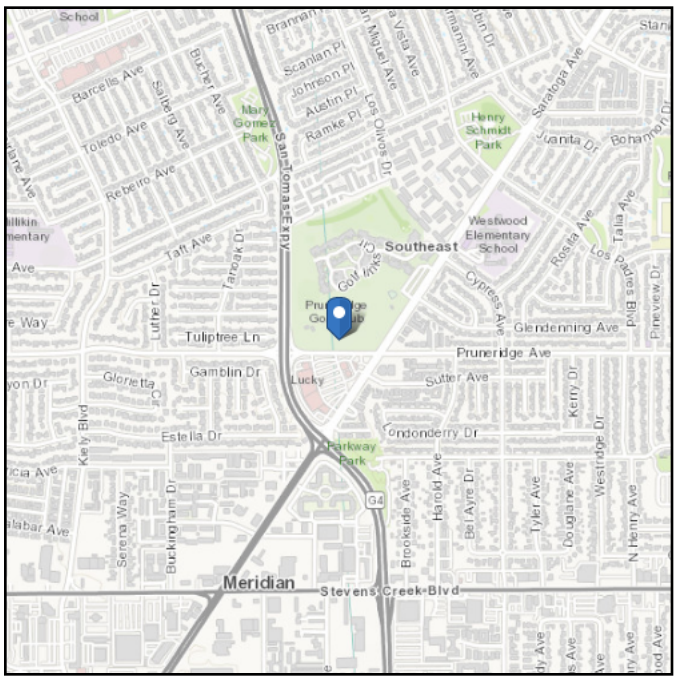
Soil pH	7.30	
Minimum Resistivity	0.51 ohm-cm (x1000)	
Chloride	103.8 ppm	0.01038 %
Sulfate-SO4	177.1ppm	0.01771 %

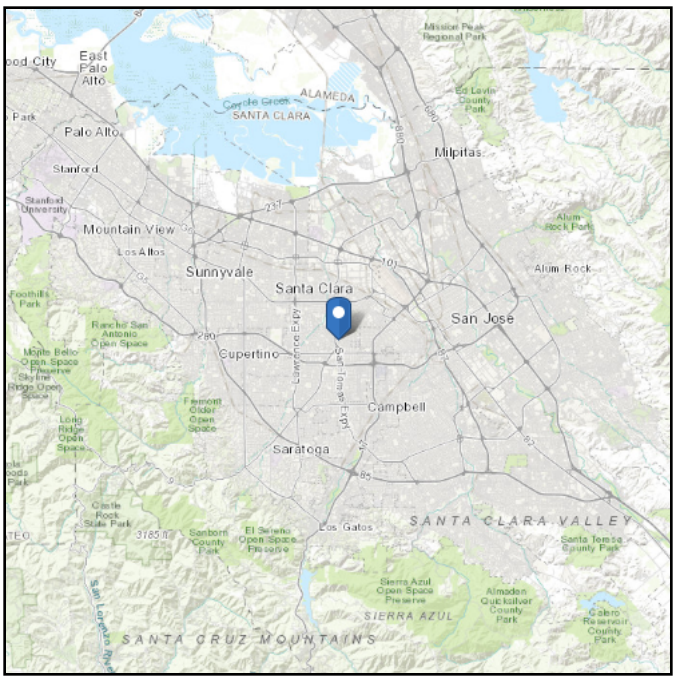
METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate-SO4 ASTM C1580, Chloride CA DOT Test #422m

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16	**Latitude:**	37.3303
Risk Category:	II	**Longitude:**	-121.9657
Soil Class:	D - Stiff Soil	**Elevation:**	113.35723719042359 ft (NAVD 88)





Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.5	S_{D1} :	1.02
S_1 :	0.6	T_L :	12
F_a :	1	PGA :	0.59
F_v :	1.7	PGA_M :	0.649
S_{MS} :	1.5	F_{PGA} :	1.1
S_{M1} :	1.53	I_e :	1
S_{DS} :	1	C_v :	1.4

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Fri Jun 07 2024

Date Source: USGS Seismic Design Maps

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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