Appendix F

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

WAREHOUSE BUILDING
459 AND 469 PIERCY ROAD
SAN JOSE, CALIFORNIA 95138

Prepared for
Xebec Realty Partners
3030 Old Ranch Road, Suite 470
Seal Beach, California 90740

March 2021
Project No. 5341-1

ROMIG ENGINEERS

March 10, 2021
5341-1

Xebec Realty Partners
3010 Old Ranch Road, Suite 470
Seal Beach, California 90470

RE: GEOTECHNICAL INVESTIGATION
WAREHOUSE BUILDING
459 AND 469 PIERCY ROAD
SAN JOSE, CALIFORNIA

Attention: Mr. Sam Salim

Gentlemen:

As requested, we have performed a geotechnical investigation for the proposed warehouse building to be constructed at 459 and 469 Piercy Road in San Jose, California. The accompanying report summarizes the results of our field exploration, laboratory testing, and engineering analysis, and presents geotechnical recommendations for the proposed project.

We refer you to the text of our report for specific recommendations.

Thank you for the opportunity to work with you on this project. If you have any questions or comments about our findings or recommendations for the project, please call.

Very truly yours,

ROMIG ENGINEERS, INC.

Tom W. Porter

Tom W. Porter, P.E.

REGISTERED PROFESSIONAL ENGINEER THOMAS W. PORTER No. 77883 CIVIL STATE OF CALIFORNIA

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GEOTECHNICAL INVESTIGATION
WAREHOUSE BUILDING
459 AND 469 PIERCY ROAD
SAN JOSE, CALIFORNIA 95138

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MARCH 2021

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GEOTECHNICAL INVESTIGATION
FOR
WAREHOUSE BUILDING
459 AND 469 PIERCY ROAD
SAN JOSE, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed warehouse building to be constructed at 459 and 469 Piercy Road in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of this investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for design and construction of the proposed project.

Project Description

The project consists of constructing an approximately 119,840 square-foot warehouse building across both 459 and 469 Piercy Road in San Jose. The building is expected to be a single-story concrete tilt up building with a concrete slab-on-grade floor. We expect that paved parking and drive aisles will extend along the perimeter of the building. The existing residential structures at 469 Piercy Road will be demolished prior to construction. 459 Piercy Road is currently an undeveloped lot. Structural loads are expected to be moderate as is typical for this type of construction

Scope of Work

Our scope of work for this investigation was presented in our agreement with Xebec Realty Partners dated July 15, 2020 and supplemental agreement dated February 17, 2021. In order to complete our investigation, we performed the following work.

- Review of geologic and geotechnical literature in our files pertinent to the general area of the site.
- Subsurface exploration consisting of drilling, sampling, and logging seven exploratory borings in the area of the proposed building.
- Laboratory testing of selected soil samples to aid in soil classification and to help evaluate the engineering properties of the soils encountered at the site.
- Engineering analysis and evaluation of the surface and subsurface data to develop earthwork guidelines and foundation design criteria for the proposed building.

- Preparation of this report presenting our findings and geotechnical recommendations for the proposed construction.

Limitations

This report has been prepared for the exclusive use of Xebec Realty Partners for specific application to developing geotechnical design criteria for the proposed warehouse building to be constructed at 459 and 469 Piercy Road in San Jose, California. We make no warranty, expressed or implied, except that our services are performed in accordance with the geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and recommendations only. In the event there are any changes in the nature, design, or location of the project, or if any future improvements are planned, the conclusions and recommendations presented in this report should not be considered valid unless 1) the project changes are reviewed by us, and 2) the conclusions and recommendations presented in this report are modified or verified in writing.

The analysis, conclusions, and recommendations presented in this report are based on site conditions as they existed at the time of our investigation; the currently planned improvements; review of previous reports relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE EXPLORATION AND RECONNAISSANCE

To familiarize ourselves with the current site conditions, we performed a site reconnaissance on December 1, 2020. As you know, we previously performed a subsurface exploration at the 469 Piercy Road on August 30, 2017 and a subsurface exploration at the 459 Piercy Road on December 20, 2017. Subsurface exploration was performed using a Mobile B-61 truck-mounted drill equipped with 8-inch diameter hollow-stem augers. A total of seven exploratory borings were advanced to depths ranging between 30 to 50 feet. The approximate locations of the borings are presented on the Site Plan, Figure 2. The boring logs and the results of our laboratory tests are attached in Appendices A and B, respectively.

Surface Conditions

The site is located in a commercial area at the north corner of the intersection of Piercy Road and Hellyer Avenue. At the time of our recent reconnaissance, 469 Piercy Road was occupied by a two-story, wood-framed residence which had a wood siding exterior. A detached three-car garage was located to the northeast of the residence with an asphaltic concrete driveway providing access to Piercy Road. Concrete walkways extended along the perimeter of the residence and garage. A covered wood deck (porch) extended along the perimeter of the residence. Two above ground water storage tanks were located at the south corner of the site.

The depth and width of the existing building foundation is unknown. The perimeter stem walls were generally covered by the exterior siding and not visible. The driveway had hairline to $^1/_8$-inch wide cracks. The concrete flatwork had a few up to ¼-inch wide cracks. Roof downspouts discharged adjacent to the perimeter foundations.

459 Piercy Road was a relatively flat, undeveloped lot partially surrounded by wood fencing. The northwest portion of the property appeared to be underlain by several feet of fill possibly placed during grading work during development of the adjacent site. The site was vegetated with native grass and a few small shrubs.

The relatively flat properties were vegetated with native grass, small shrubs, and medium to large trees. Both properties appeared to be in similar condition as observed during our field explorations in 2017.

Subsurface Conditions

469 Piercy Road

At the location of Boring EB-1, we encountered approximately 5 feet of very stiff sandy lean clay of low to moderate plasticity underlain by approximately 6 feet of medium dense clayey gravel. We then encountered approximately 5 feet of very stiff sandy lean clay of low plasticity underlain by approximately 8 feet of dense poorly graded sand, underlain by hard sandy lean clay of moderate plasticity which extended to the maximum depth explored of 35 feet.

In Borings EB-2 and EB-4, we encountered stiff to hard sandy lean clay of low to moderate plasticity which extended to the maximum depths explored of 30 to 50 feet.

In Boring EB-3, we encountered approximately 12 feet of hard sandy lean clay of low to moderate plasticity underlain by approximately 15 feet of dense to very dense clayey gravel. We then encountered very stiff to hard sandy lean clay of moderate plasticity which extended to the maximum depth explored of 50 feet.

459 Piercy Road

At the location of Borings EB-1 and EB-2, we encountered approximately 2 to 2.5 feet of fill which consisted of very stiff to hard sandy lean clay of low plasticity. These surface soils appeared to be a possible surface fill or disturbed native soil. The surface soils were underlain by stiff to hard sandy lean clay of low to high plasticity which extended to the maximum depths explored of 44.9 and 49.9 feet.

In Boring EB-3, we encountered approximately 23 feet of stiff to hard sandy lean clay of low to moderate plasticity underlain by medium dense to very dense poorly graded sand which extended to the maximum depth explored of 35 feet.

A Liquid Limits of 40 and 49 and a Plasticity Indices of 19 and 25, respectively were measured on a sample of near surface native soil obtained from our borings. These test results indicate that the near surface soil generally has moderate to high plasticity and a low to moderate to high potential for expansion.

Ground Water

At 469 Piercy Road, ground water was measured at a depth of about 31 feet in Boring EB-1, at a depth of about 21 feet in Borings EB-2 and EB-3, and at a depth of about 25 feet in Boring EB-4, shortly after drilling and sampling was completed. At 459 Piercy Road, ground water was measured at a depth of about 42 feet in Boring EB-1, at a depth of about 24 feet in Boring EB-2, and at a depth of about 23 feet in Boring EB-3, shortly after drilling and sampling was completed. The borings were backfilled with grout shortly after drilling, therefore a stabilized ground water level may not have been obtained.

Information presented in Seismic Hazard Zone Report 044 for the San Jose East Quadrangle (California Geological Survey, 2000) indicates the historical high ground water level in the area of the site is expected to be present at an average depth of approximately 20 to 30 feet below the ground surface. Please be cautioned that fluctuations in the level of ground water can occur due to variations in rainfall, landscaping, surface and subsurface drainage patterns, and other factors.

GEOLOGIC SETTING

As part of our investigation, we briefly reviewed our local experience and geologic information in our files pertinent to the general area of the site. The information reviewed indicates that the site is underlain by Holocene age older alluvial fan deposits, Qhf2 (Blake, Graymer, McLaughlin and Wentworth, 1999). The unit is generally described as brown or tan, medium dense gravelly sand or sandy gravel that transitions upward to sandy or silty clay. The geology within the site vicinity is shown on the Vicinity Geologic Map, Figure 3.

The property and the immediate vicinity are located in an area that slopes very gently toward the southwest (approximately 10 feet vertically per 3,000 feet laterally, although locally the topography may be steeper). The site is located at an elevation of approximately 200 feet above sea level.

The Geologic Hazard Zone Map (2012) prepared by the County of Santa Clara and the State Seismic Hazard Zones Map of the San Jose East Quadrangle (California Geological Survey, 2001) indicates the site is located in an area that may be underlain by soils that have the potential to liquefy during a major earthquake. The potential for liquefaction of the soils encountered at the site is discussed later in this report.

Faulting and Seismicity

The County Hazard map indicates the site is located in a fault rupture hazard zone possibly related to the Silver Creek fault located to the northeast. The City of San Jose map (1983) indicates a splay fault, shown as the Evergreen fault splay mapped immediately to the northeast of the site. However, we understand that recent fault trenching work conducted between the site and the mapped faults did not encounter traces of faulting and that the City has indicated that a fault study is not required for project approval.

The site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probable. The closest active fault is the Hayward fault, located approximately 3.5 miles northeast of the property. Thus, the likelihood of surface rupture occurring from active faulting at the site is low.

The San Francisco Bay Area is an active seismic region. Earthquakes in the region result from strain energy constantly accumulating because of the northwestward movement of the Pacific Plate relative to the North American Plate. On average about 1.6-inches of movement occur per year. Historically, the Bay Area has experienced large, destructive earthquakes in 1838, 1868, 1906, and 1989. The faults considered most likely to produce large earthquakes in the area include the Hayward, Calaveras, San Andreas, and San Gregorio faults. The San Andreas and Calaveras faults are located approximately 12 miles southwest and 13 miles northeast of the site, respectively. The San Gregorio fault is located approximately 30 miles southwest of the site. These faults and significant earthquakes that have been documented in the Bay Area are listed in Table 1 and are shown on the Regional Fault and Seismicity Map, Figure 4.

Table 1. Earthquake Magnitudes and Historical Earthquakes
Warehouse Building
San Jose California

Fault	Maximum Magnitude (Mw)	Historical Earthquakes	Estimated Magnitude
San Andreas	7.9	1989 Loma Prieta	6.9
		1906 San Francisco	7.9
		1865 N. of 1989 Loma Prieta Earthquake	6.5
		1838 San Francisco-Peninsula Segment	6.8
		1836 East of Monterey	6.5
Hayward	7.1	1868 Hayward	6.8
		1858 Hayward	6.8
Calaveras	6.8	1984 Morgan Hill	6.2
		1911 Morgan Hill	6.2
		1897 Gilroy	6.3
San Gregorio	7.3	1926 Monterey Bay	6.1

In the future, the subject property will undoubtedly experience severe ground shaking during moderate and large magnitude earthquakes produced along the San Andreas fault or other active Bay Area fault zones. Using information from recent earthquakes, improved mapping of active faults, ground motion prediction modeling, and a new model for estimating earthquake probabilities, a panel of experts convened by the U.S.G.S. have concluded there is a 72 percent chance for at least one earthquake of Magnitude 6.7 or larger in the Bay Area before 2043. The Hayward fault has the highest likelihood of an earthquake greater than or equal to magnitude 6.7 in the Bay Area, estimated at 33 percent, while the likelihood on the San Andreas and Calaveras faults is estimated at approximately 22 and 26 percent, respectively (Aagaard et al., 2016).

Earthquake Design Parameters

The State of California currently requires that buildings and structures be designed in accordance with the seismic design provisions presented in the 2019 California Building Code and in ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures." Based on site geologic conditions and on information from our subsurface exploration at the site, the site may be classified as Site Class D, stiff soil, in accordance with Chapter 20 of ASCE 7-16. Spectral Response Acceleration parameters and site coefficients may be taken directly from the U.S.G.S. website based on the longitude and latitude of the site. For site latitude (37.2594), longitude (-121.7817) and Site Class D, design parameters are presented on Table 2.

Table 2. 2019 CBC Seismic Design Criteria
Warehouse Building
San Jose, California

Spectral Response Acceleration Parameters			Design Value
Mapped Value for Short Period	-	S_S	1.653
Mapped Value for 1-sec Period	-	S_1	0.625
Site Coefficient	-	F_a	1.0
Site Coefficient	-	F_v	1.7*
Adjusted for Site Class	-	S_{MS}	1.654
Adjusted for Site Class	-	S_{M1}	1.062*
Value for Design Earthquake	-	S_{DS}	1.102
Value for Design Earthquake	-	S_{D1}	0.708*

* The values of F_v, S_{M1} and S_{D1} above are provided for calculation of Ts. A site-specific ground motion hazard analysis may be required unless the exceptions in ASCE 7-16 Section 11.4.8 apply to the project.

Geologic Hazards

As part of our investigation, we reviewed the potential for geologic hazards to impact the site and the proposed building, considering the geologic setting and the soils encountered during our investigation. The results of our review are presented below and in the following sections of our report.

- Fault Rupture - The site is not located in a State of California Earthquake Fault Zone or area where fault rupture is considered likely. Therefore, active faults are not believed to exist beneath the site and the potential for fault rupture at the site is considered low.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the life of the building, as is typical for sites throughout the Bay Area. The building should be designed in accordance with current earthquake resistance standards.

Liquefaction and Differential Compaction

Severe ground shaking during an earthquake can cause loose to medium dense granular soils to densify. If the granular soils are below ground water, their densification can cause increases in pore water pressure, which can lead to soil softening, liquefaction, and ground deformation. Soils most prone to liquefaction are saturated, loose to medium dense, silty sands and sandy silts with limited drainage, and in some cases, sands and gravels that are interbedded with or that contain seams or layers of impermeable soil.

The clayey sand encountered at the site below the highest projected ground water depth, which is estimated to be about 21 feet below the ground surface, was considered in our liquefaction analysis. Soils with normalized standard penetration test, $(N_1)_{60}$, greater than 30 blows per feet were considered too dense to liquefy.

To evaluate the potential for earthquake-induced liquefaction of the sandy soils at the site within the depth of exploration, we performed a liquefaction analysis of the data from our borings generally following the methods described in the 2008 publication by Idriss and Boulanger titled "Soil Liquefaction During Earthquakes".

In addition to liquefaction, we analyzed the potential for differential compaction of the medium dense sandy soils above the water table during periods of time when a deeper ground water condition is present, such as encountered in our borings. Differential compaction occurs during moderate and large earthquakes when soft or loose, natural or fill soils densify and settle, often unevenly across a site. To evaluate the potential for earthquake-induced differential compaction, we performed a settlement analysis of the data from our borings following the methods presented at the US Army Corps of Engineers EM1110-1-1904.

Potentially liquefiable soils and/or soils prone to differential compaction were encountered in Boring EB-3 advanced at 459 Piercy Road between depths of approximately 27 to 32 feet and in Boring EB-1 at 469 Piercy Road between depths of about 5 to 11 feet. These clayey and gravelly sands and gravelly sands are potentially prone to liquefaction when subjected to the maximum considered earthquake acceleration (PGA_M) of 0.76g based on the Probabilistic Seismic Hazards Mapping Ground Motion Page (CGS, 2021).

Based on the results of our analysis, we estimate that total settlement of about ½-inch could occur within this clayey gravel strata due to severe ground shaking caused by a major earthquake. In our opinion, differential settlement of about ¼- to ½-inch over a horizontal distance of about 50 feet is possible at the ground surface from this amount of total settlement.

CONCLUSIONS

From a geotechnical viewpoint, the site is suitable for the proposed warehouse building provided the recommendations presented in this report are followed during design and construction. Specific geotechnical recommendations for the project are presented in the following sections of this report.

The primary geotechnical concerns for the proposed improvements are the presence of up to 3 feet of existing fill material encountered along the northwest portion of the site and the potential for variable support conditions across the building foundation depending on the final grading scheme. We expect that the proposed building will be bearing in very stiff to hard native soils below the existing fill. In our opinion, the proposed warehouse building may be supported on conventional spread footing foundations bearing in stiff native soils. During design, our office should be retained to finalize the foundation design and building settlement criteria presented in this report.

In our opinion, any existing fill not removed during grading for the building pad should be excavated and recompacted below the building, exterior flatwork, pavements, and any other site improvements during site preparation. The reworking of the fill and subgrade preparation should proceed as recommended in the section of this report titled "Earthwork." If documentation regarding the compaction of the existing fill can be obtained, it may be possible to utilize some of the existing fill, provided that the fill was compacted to current engineering standards.

Because subsurface conditions may vary from those encountered at the location of our borings, and to observe that our recommendations are properly implemented, we recommend that we be retained to: 1) review the grading and foundation plans for conformance with the recommendations presented in this report and; 2) observe and test during earthwork, foundation, shoring, drainage and slab construction.

FOUNDATIONS

Spread Footing Foundations

In our opinion, the building, loading dock ramps, site retaining walls, and other minor site improvements may be supported on a conventional spread footing foundation system bearing on stiff native soil. All continuous footings should have a width of at least 15 inches and should extend at least 30 inches below exterior grade and at least 24 inches below the bottom of concrete slabs-on-grade, whichever is deeper. Continuous footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 3,000 pounds per square foot for dead loads, 4,000 pounds per square foot for dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

All footings located adjacent to utility lines should bear below a 1:1 plane extending up from the bottom edge of the utility trench. We recommend that continuous foundations be designed with sufficient depth and reinforcing to tolerate the estimated differential settlement.

Our representative should observe all footing excavations prior to placement of reinforcing steel to confirm that they expose suitable material and have been properly cleaned. If soft or loose soils are encountered in the foundation excavations, our field representative may require overexcavation and/or compactive effort or a deeper footing depth before the reinforcing steel is placed.

Lateral Loads

Lateral loads may be resisted by friction between the bottom of the footings and the supporting subgrade. A coefficient of friction of 0.30 may be assumed for footing design. In addition, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in footing excavations or backfilled with compacted structural fill. We recommend assuming an equivalent fluid pressure of 350 pounds per cubic foot for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing or mat will be landscaped or subject to softening from rainfall and/or surface water runoff.

Settlement

On a preliminary basis, the 30-year post-construction differential settlement due to static loads is not expected to exceed about 1-inch across the proposed building, provided the building foundations are designed and constructed as recommended. As discussed earlier, additional differential settlement of about ¼- to ½-inch is possible across the foundation from dynamic densification during seismic shaking.

SLABS-ON-GRADE

General Slab Considerations

Portions of the surface and near surface soils at this site have a moderate to high potential for expansion. Expansive soils expand due to increases in moisture content and shrink as they dry. This can result in some slab lifting and cracking regardless of the geotechnical measures that are implemented. The recommendations presented below will help mitigate the influence of the expansive soils on the overlying concrete slabs-on-grade but will not eliminate the risks entirely.

To reduce the potential for expansion of the soil subgrades below at-grade concrete slabs-on-grade, at least the upper 6-inches of the surface soil should be scarified, moisture conditioned, and compacted at a moisture content at least 3 percent above the laboratory optimum. The native soil subgrade should be kept moist up until the time the non-expansive fill, crushed rock and vapor barrier, and/or aggregate base section is installed. Slab subgrades and non-expansive fill should be prepared and compacted as recommended in the section of this report titled "Earthwork." Exterior flatwork and interior slabs-on-grade should be underlain by a layer of non-expansive fill as described below. The non-expansive fill should consist of imported soil with a Plasticity Index no greater than 15, preferably Class 2 aggregate base.

Considering the potential for expansive soil movements of the surface soil, we expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

Exterior Flatwork

Concrete walkways and exterior flatwork should be at least 4 inches thick and should be constructed on at least 10 inches of Class 2 aggregate base. The potential for distress to exterior slabs due to expansive soil movements could be reduced by placing and compacting 4 inches of non-expansive fill, or aggregate base, below the minimum 10-inch thick layer of aggregate base recommended above. To improve performance, exterior slabs-on-grade may be constructed with a thickened edge to improve edge stiffness and to reduce the potential for water seepage under the edge of the slabs.

Interior Slabs

Concrete slab-on-grade floors for the building should be constructed on a layer of non-expansive fill at least 14-inches thick and constructed on a properly prepared and compacted soil subgrade. Since the building floor will likely support various loads within the warehouse, we recommend that the building floor slab be at least 5 inches in thickness. Recycled aggregate base should not be used for non-expansive fill below interior slabs-on-grade, since adverse vapor could occur from crushed asphalt components.

Moisture Considerations

In areas where dampness of concrete floor slabs would be undesirable, such as within building interiors, concrete slabs and mat should be underlain by at least 4 inches of clean, free-draining gravel, such as ½-inch to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used. The crushed rock should be compacted with vibratory equipment and may be considered at the upper portion of the non expansive fill recommended above.

To reduce vapor transmission up through at-grade concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor retarder meeting the minimum ASTM E 1745, Class C requirements or better. If moisture-sensitive floor coverings are proposed and/or additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.01 perms (such as 15-mil thick "Stego Wrap Class A") may be used rather than a Class C vapor retarder. The vapor retarder or barrier should be placed directly below the concrete slab. Sand above the vapor retarder/barrier is not recommended. The vapor retarder/barrier should be installed in accordance with ASTM E 1643. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations.

The permeability of concrete is affected significantly by the water:cement ratio of the mix, with lower water:cement ratios producing more damp-resistant slabs and higher strength. Where moisture protection is important and/or where the concrete will be placed directly on the vapor barrier, the water:cement ratio should be 0.45 or less. To increase the workability of the concrete, mid-range plasticizers may be added to the mix. Water should not be added to the mix unless the slump is less than specified and the water:cement ratio will not exceed 0.45. Other steps that may be taken to reduce moisture transmission through concrete slabs-on-grade include moist curing for 5 to 7 days and allowing the slab to dry for a period of two months or longer prior to placing floor coverings. Prior to installation of floor coverings, it may be appropriate to test the slab moisture content for adherence to the manufacturer's requirements to determine whether a longer drying time is necessary.

RETAINING WALLS

Retaining walls should be designed to resist lateral pressures from the adjacent native and fill soils and backfill. We recommend retaining walls with level backfill that are not free to deflect or rotate, such as the loading dock ramp retaining walls, be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot, plus an additional uniform lateral pressure of 8H pounds per square foot, where H is the height of the wall in feet. Retaining walls with level backfill that are free to rotate, such as site retaining walls, may be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot. Wherever walls will be subjected to surcharge loads, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge load for restrained walls and one-third of the surcharge load for unrestrained walls.

Based on the site peak ground acceleration (PGA), on Seed and Whitman (1970); Al Atik and Sitar (2010); and Lew et al. (2010); seismic loads on retaining walls that can yield may be simulated by a line load of $7H^2$ (in pounds per foot, where H is the wall height in feet). Seismic loads on walls that cannot yield may be subjected to a seismic load as high as about $13H^2$. This seismic surcharge line load should be assumed to act at 1/3H above the base of the wall (in addition to the active wall design pressure of 45 or 65 pounds per cubic foot).

To prevent buildup of water pressure from surface water infiltration, a subsurface drainage system should be installed behind the walls. The drainage system should consist of a 4-inch diameter perforated pipe (perforations placed down) embedded in a section of 1/2- to 3/4-inch, clean, crushed rock at least 12 inches wide. Backfill above the perforated drain line should also consist of 1/2- to 3/4-inch, clean, crushed rock to within about 1½ to 2 feet below exterior finished grade. A filter fabric should be wrapped around the crushed rock to protect it from infiltration of native soil. The upper 1½ to 2 feet of backfill should consist of compacted native soil. The perforated pipe should discharge into a free-draining outlet or sump that pumps to a suitable location. Damp-proofing of the walls should be included in areas where wall dampness and efflorescence would be undesirable.

Miradrain, Enkadrain or other drainage fabrics approved by our office may be used for wall drainage as an alternative to the gravel drainage system described above. If used, the drainage fabric should extend from a depth of about 1 foot below the top of the wall backfill down to the drain pipe at the base of the wall. A minimum 12-inch wide section of ½-inch to ¾-inch clean crushed rock and filter fabric should be placed around the drainpipe, as recommended previously.

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy equipment is used for compaction of wall backfill, the walls should be temporarily braced. Preferably, the backfill behind the walls should be placed on level benches, rather than directly on the sloping grade.

Loading dock ramp retaining walls and site retaining walls may be supported on a continuous shallow footing as presented previously.

VEHICLE PAVEMENTS

Asphalt Concrete Pavements

Based on the anticipated composition of the surface soils, and an estimated traffic index for the proposed pavement loading conditions, we developed the minimum pavement sections presented in Table 3 below based on Procedure 630 of the Caltrans Highway Design Manual.

The Traffic Indices used in our pavement thickness calculations are considered reasonable values for this development and are based on engineering judgment rather than on detailed traffic projections. Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of the Caltrans Standard Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

Table 3. Pavement Sections
Warehouse Building
San Jose, California

Traffic Loading Condition	Design Traffic Index	Asphalt Concrete (inches)	Aggregate Base* (inches)	Total Thickness (inches)
Automobile Parking	4.0	3.0	6.0	9.0
Automobile Access	4.5	3.0	7.0	10.0
Light Truck Traffic	5.0	3.0	8.0	11.0
Moderate Truck Traffic	6.0	4.0	10.0	14.0
Heavy Truck Traffic	7.0	4.0	13.0	17.0

*Caltrans Class 2 Aggregate Base (minimum R-value = 78).

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscape areas. Seepage of water into the pavement base material tends to soften the subgrade, increasing the amount of pavement maintenance that is required and shortening the pavement service life. Deepened curbs extending 4-inches below the bottom of the aggregate base layer are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

<u>Portland Cement Concrete Pavements</u>

If Portland Cement Concrete (PCC) pavements are to be used on portions of the site, the minimum required thickness of the PCC pavements should be based on the anticipated traffic loading, the modulus of rupture of the concrete that will be used for pavement construction, and the composition and supporting characteristics of the soil subgrade below the pavement section.

To provide a general guideline for the minimum required thickness of PCC pavements, we used information in the Portland Cement Association publication titled "Thickness Design for Concrete Highway and Street Pavements." We assumed "low" subgrade support from the on-site soils, considering typical residential street traffic (up to 25 daily trucks with maximum single axle loads of 22 kips and maximum tandem axle loads of 36 kips), aggregate-interlock joints (i.e. no dowels), no concrete shoulder or curb, a modulus of rupture of concrete of 550 psi (which correlates to a concrete compressive strength of approximately 3,700 psi), at least 8 inches of Class 2 aggregate base below the PCC pavement, and 20-year pavement service life. Sufficient control joints should be incorporated in the design and construction to limit and control cracking.

Based on the design assumptions described above, a PCC pavement with a thickness of at least 6 inches would be adequate for average daily truck traffic (ADTT) of one; a thickness of at least 6.5 inches would be adequate for ADTT of 13; and a thickness of at least 7 inches would be adequate for ADTT of 110.

EARTHWORK

<u>Clearing and Subgrade Preparation</u>

All deleterious materials, such as existing foundations, pavements, flatwork, utilities to be abandoned, vegetation, root systems, surface fills, topsoil, etc. should be cleared from areas of the site to be built on or paved. The actual stripping depth should be determined by a member of our staff in the field at the time of construction. Excavations that extend below finished grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section of this report titled "Compaction."

After the site has been properly cleared, stripped, and excavated to the required grades, exposed soil surfaces in areas to receive structural fill or slabs-on-grade should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section of this report titled "Compaction."

On-site soils, foundation and utility trench excavations, and slab and pavement subgrades should be kept in a moist condition throughout the construction period to help mitigate the potential effects of the expansive on-site soils on the proposed surface improvements.

Material For Fill

All on-site soil containing less than 3 percent organic material by weight (ASTM D2974) may be suitable for use as structural fill. Structural fill should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported, non-expansive fill should have a Plasticity Index no greater than 15, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. A member of our staff should approve proposed import materials prior to their delivery to the site.

Building Pad Recommendations

In our opinion, the existing fill located along the northwest area of the site (if not removed) should be excavated and compacted below the building, exterior flatwork, pavements, and other site improvements, with a 5 foot overbuild, where possible. The fill should be excavated down to stiff native soil and compacted under our direction. Imported backfill materials should be approved by a member of our staff prior to delivery to the site. The backfill should be moisture conditioned, and compacted as recommended in the section of this report titled "Compaction." A member of our staff should observe and test during re-working of the existing fill and compaction of new fill across the building pad, as required.

Temporary Slopes and Excavations

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards.

Due to the potential for variation of the on-site soil, field modification of temporary cut slopes may be required. Unstable materials encountered on excavations and slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination.

Protection of structures near cuts should also be the responsibility of the contractor. In our experience, a preconstruction survey is generally performed to document existing conditions prior to construction, with intermittent monitoring of the structures during construction.

Compaction

Scarified soil surfaces and all structural fill should be compacted in uniform lifts no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 4 below. The relative compaction and moisture content recommended in Table 4 is relative to ASTM Test D1557, latest edition.

Table 4. Compaction Recommendations
Warehouse Building
San Jose, California

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of native soil.	87 to 92 percent	At least 3 percent above optimum
• Structural fill composed of non-expansive fill.	90 percent	Above optimum
Pavement Areas		
• Upper 6-inches of soil below aggregate base.	93 percent	2 to 3 percent above optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	87 to 92 percent	At least 3 percent above optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

At the start of site grading and earthwork construction, and prior to subgrade preparation and placement of non-expansive fill, representative samples of on-site soil and import material will need to be collected in order for a laboratory compaction test to be performed for use during on-site density testing. Sampling of on-site soil and proposed import material should be requested by the contractor at least 5 days prior to when our staff will be needed for density testing to allow time for soil sampling and laboratory testing to be performed prior to our on-site compaction testing.

<u>Finished Slopes</u>

We recommend that finished slopes be cut or filled to an inclination no steeper than 2:1 (horizontal:vertical). Exposed slopes may be subject to minor sloughing and erosion, which could require periodic maintenance. We recommend that all slopes and soil surfaces disturbed during construction be planted with erosion-resistant vegetation.

<u>Surface Drainage</u>

Finished grades should be designed to prevent ponding and to drain surface water away from foundations and edges slabs and pavements, and toward suitable collection and discharge facilities. Slopes of at least 2 percent are recommended for flatwork and pavement areas with 5 percent preferred in landscape areas within 8 feet of the structures, where possible. At a minimum, splash blocks should be provided at the ends of downspouts to carry surface water away from perimeter foundations. Preferably, downspout drainage should be collected in a closed pipe system that is routed to a storm drain system or other suitable discharge outlet.

Infiltration basins or unlined bioswales, if any, preferably should not be placed within about 10 feet of the building foundation or slab or flatwork areas. Drains should be provided for infiltration basins that direct water to an appropriate outlet as required by the civil engineer.

Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction. We recommend that an as-built plan be prepared to show the locations of all surface and subsurface drain lines and clean-outs. Drainage facilities should be periodically checked to verify that they are continuing to function properly. The drainage facilities will probably need to be periodically cleaned of silt and debris that may build up in the lines.

FUTURE SERVICES

<u>Plan Review</u>

Romig Engineers should review the completed grading and foundation plans for conformance with the recommendations presented in this report. We should be provided with these plans as soon as possible upon their completion in order to limit the potential for delays in the permitting process that might otherwise be attributed to our review. In addition, it should be noted that many of the local building and planning departments now require "clean" geotechnical plan review letters prior to acceptance of plans for their final review. Since our plan reviews typically result in recommendations for modification of the plans, our generation of a "clean" review letter often requires two iterations. At a minimum, we recommend the following note be added to the plans.

"Earthwork, foundation construction, slab subgrade preparation, utility trench backfill, pavement construction, and site drainage should be performed in accordance with the geotechnical report prepared by Romig Engineers, Inc., dated March 10, 2021. Romig Engineers should be notified at least 48 hours in advance of any earthwork or foundation construction and should observe and test during earthwork and foundation construction as recommended in the geotechnical report." Romig Engineers should be notified at least 5 days prior to earthwork, trench backfill and subgrade preparation work to allow time for sampling of on-site soil and laboratory compaction curve testing to be performed prior to on-site compaction density testing."

Construction Observation and Testing

The earthwork and foundation phases of construction should be observed and tested by us to 1) confirm that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications, and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations presented in this report are based on a limited amount of subsurface exploration. The nature and extent of variation across the site may not become evident until construction. If variations are exposed during construction, it will be necessary to reevaluate our recommendations.

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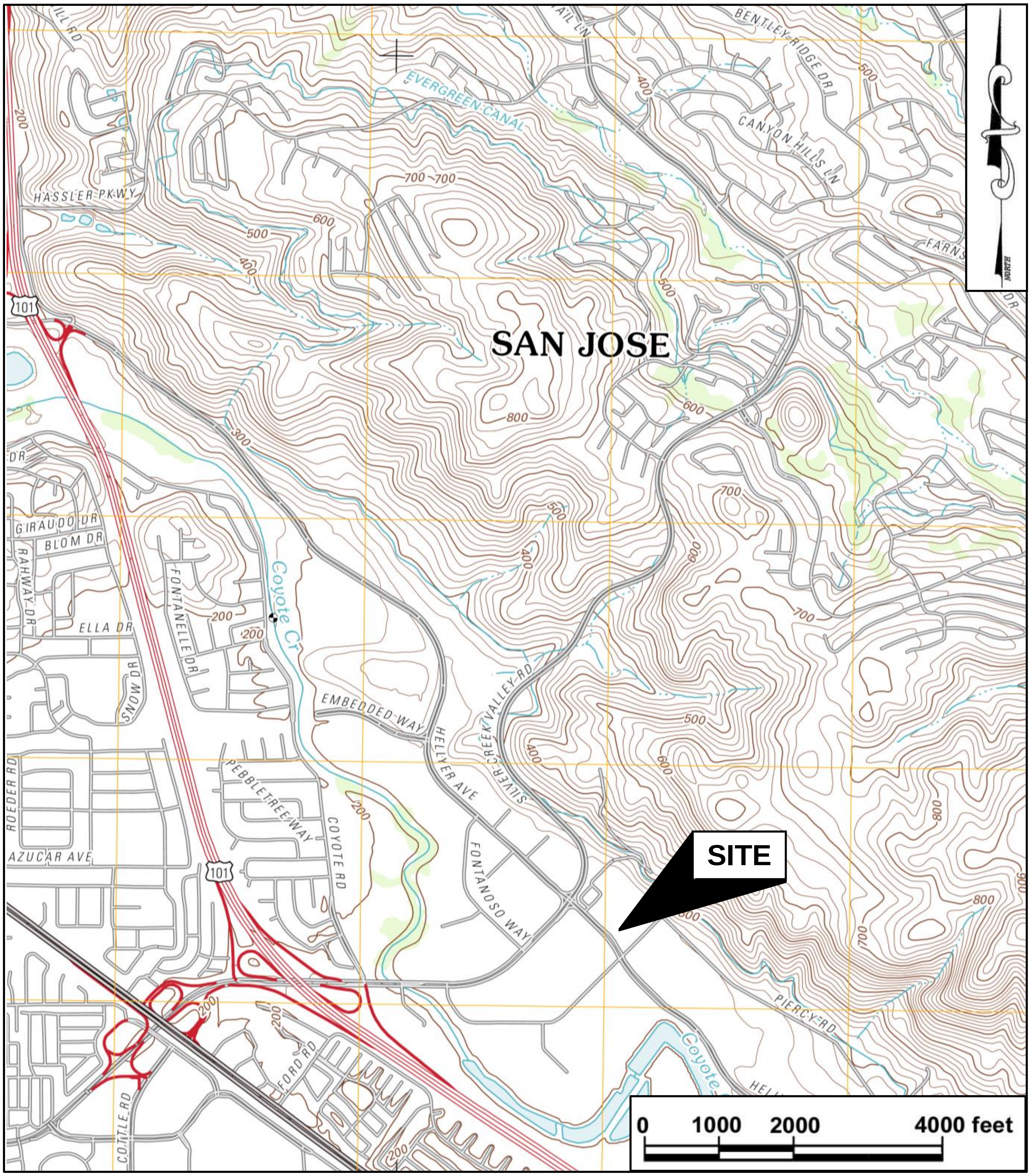


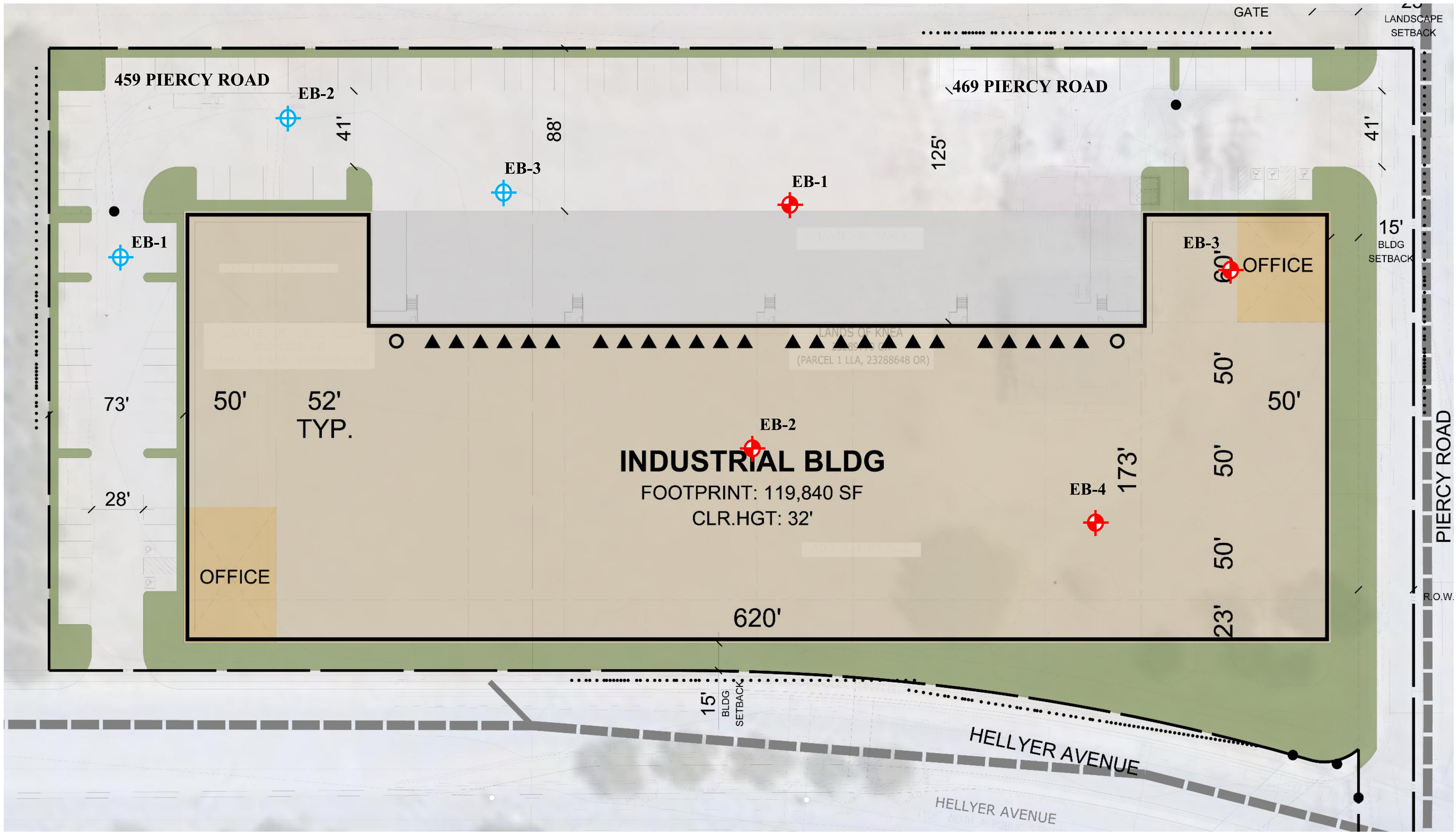
Scale: 1 inch = 2000 feet

Base is United States Geological Survey San Jose East 7.5 Minute Quadrangle, dated 2012.

VICINITY MAP
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

FIGURE 1
MARCH 2021
PROJECT NO. 5341-1

ROMIG ENGINEERS



LEGEND

EB-4 Approximate Location of Exploratory Borings (August, 2017).

EB-3 Approximate Location of Exploratory Borings (December, 2017).

Approximate Scale: 1 inch = 60 feet.

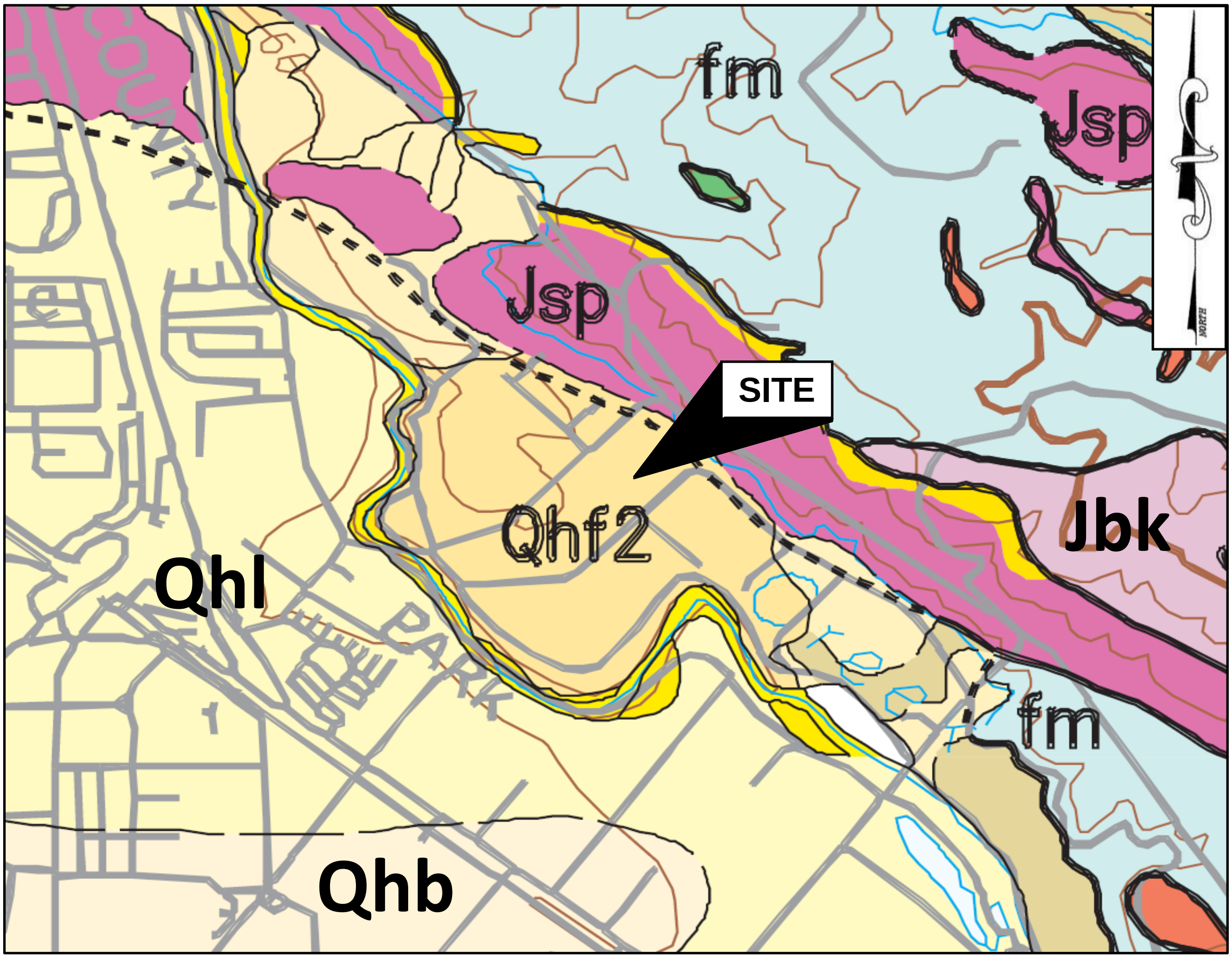
Base is site plan prepared by Ware Malcomb, dated February 9, 2021.

0 30 60 120 feet

SITE PLAN
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

ROMIG ENGINEERS

FIGURE 2
MARCH 2021
PROJECT NO. 5341-1



LEGEND

Qhb	Basin Deposits
Qhl	Levee Deposits
Qhf2	Alluvial Fan Deposits
fm	Franciscan Complex Melange
Jbk	Basalt, Keratophyre, and Quartz Keratophyre
Jsp	Serpentinized Harzburgite and Dunite

Fault - dashed where approximate, dotted where inferred.

Geologic Contact - dashed where approximate, dotted where inferred.

0 1000 2000 4000 feet

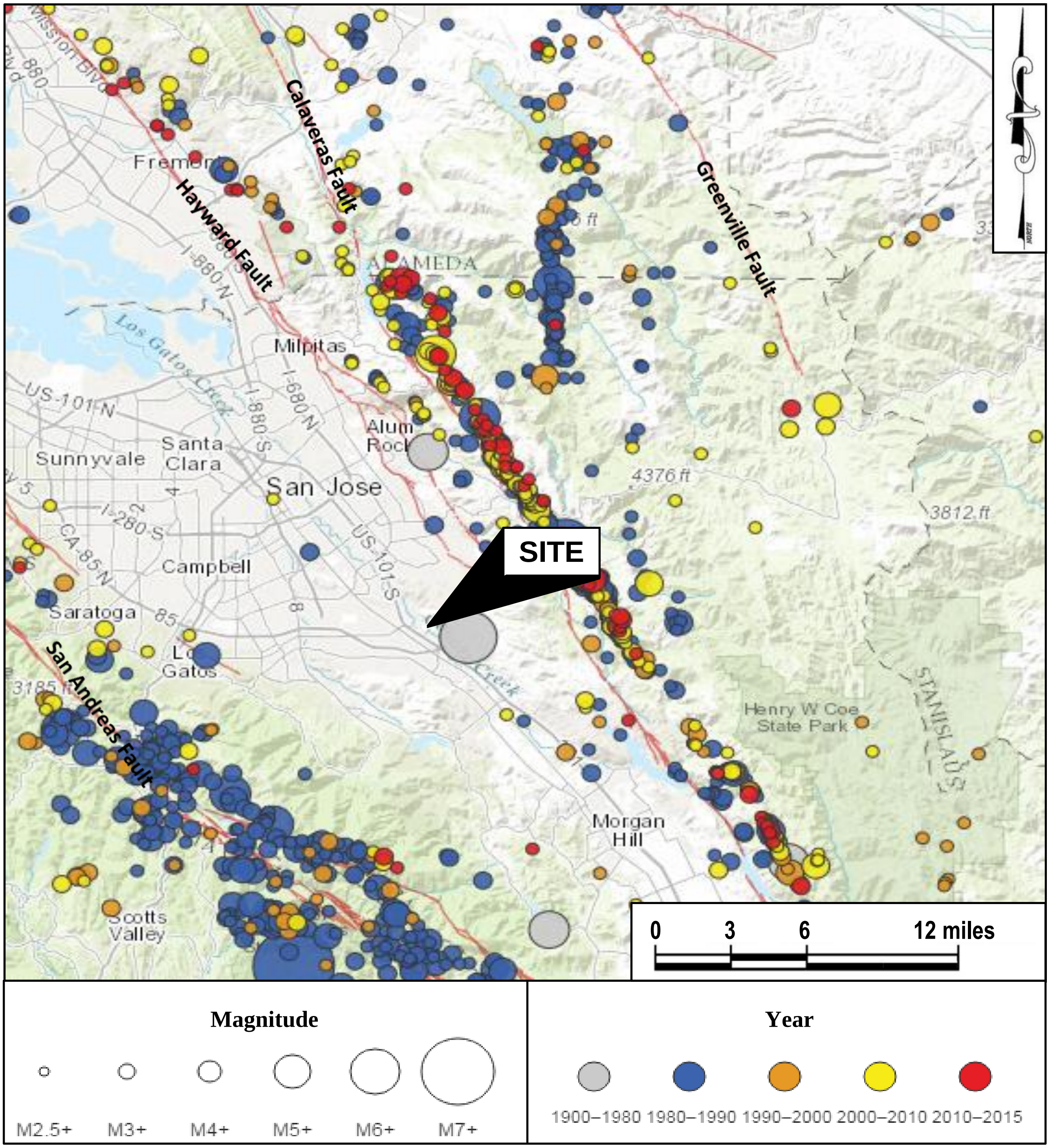
Scale: 1 inch = 2000 feet

Base is Geologic Map of San Jose 30 x 60 Minute Quadrangle (Wentworth, Blake, McLaughlin, Graymer, 1999).

VICINITY GEOLOGIC MAP
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

FIGURE 3
MARCH 2021
PROJECT NO. 5341-1

ROMIG ENGINEERS



Earthquakes with M5+ from 1900 to 1980, M2.5+ from 1980 to January 2015. Faults with activity in last 15,000 years. Based on data sources from Northern California Earthquake Data Center and USGS Quaternary Fault and Fold Database, accessed May 2015.

REGIONAL FAULT AND SEISMICITY MAP
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

FIGURE 4
MARCH 2021
PROJECT NO. 5341-1

ROMIG ENGINEERS

APPENDIX A

FIELD INVESTIGATION

The soils encountered during drilling were logged by our representative and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were evaluated and classified in accordance with the Unified Soil Classification System. The logs of our borings and a summary of the soil classification system used on the logs (Figure A-1), are attached.

Several tests were performed in the field during drilling. The standard penetration test resistance was determined by dropping a 140-pound hammer through a 30-inch free fall and recording the blows required to drive the 2-inch diameter sampler 18 inches. The standard penetration test (SPT) resistance is the number of blows required to drive the sampler the last 12 inches and is recorded on the boring logs at the appropriate depths. Soil samples were also collected using 2.5-inch and 3.0-inch O.D. drive samplers. The blow counts shown on the logs for these larger diameter samplers do not represent SPT values and have not been corrected in any way.

The location of the borings was established by pacing using the site plan provided to us and should be considered accurate only to the degree implied by the method used.

The boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

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USCS SOIL CLASSIFICATION

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (< 50 % Fines)	GRAVEL	CLEAN GRAVEL (< 5% Fines)	**GW**	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL with FINES	**GM**	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND (< 5% Fines)	**SW**	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND WITH FINES	**SM**	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (> 50 % Fines)	SILT AND CLAY Liquid limit < 50%		**ML**	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY Liquid limit > 50%		**MH**	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			**Pt**	Peat and other highly organic soils.
BEDROCK			**BR**	Weathered bedrock.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
FIRM	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12 " 3" 0.75" 4 10 40 200

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

* Standard Penetration Test (SPT) resistance, using a 140 pound hammer falling 30 inches on a 2 inch O.D. split spoon sampler; blow counts not corrected for larger diameter samplers.

^ Unconfined Compressive strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

KEY TO SAMPLERS

Modified California Sampler (3-inch O.D.)
Mid-size Sampler (2.5-inch O.D.)
Standard Penetration Test Sampler (2-inch O.D.)

KEY TO EXPLORATORY BORING LOGS **FIGURE A-1**
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD) MARCH 2021
SAN JOSE, CALIFORNIA PROJECT NO. 5341-1

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 31 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Light brown, Sandy Lean Clay, moist, fine to medium sand, low to moderate plasticity, trace subrounded to round gravel.	Very Stiff	CL		0					
							13		
						17	13		
							10		
						17	16		
				5					
Light brown to gray, Clayey Gravel, slightly moist, fine to coarse sand, $^{1}/_{4}$- to 1-inch diameter subangular to round gravel.	Medium Dense	GC					3		
						25	2		
				10		14	2		
Brown, Sandy Lean Clay, moist, fine to coarse sand, low plasticity, trace fine angular to round gravel.	Very Stiff	CL							
				15		19	12		
Light brown, Poorly Graded Sand, moist, slightly moist, fine to medium sand, trace fine subrounded to rounded gravel.	Dense	SP							
● 11% Passing No. 200 Sieve.				20	●	36	5		

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 31 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Light brown, Poorly Graded Sand, moist, slightly moist, fine to medium sand, trace fine subrounded to rounded gravel.	Dense	SP		20					
Brown, Sandy Lean Clay, moist, fine sand, moderate plasticity.	Hard	CL		25		37	12		
Very moist, decreased plasticity.				30		32	33		> 4.5
▼ Ground water measured at 31 feet after drilling.				▼					
				35		33	31		
Botton of Boring at 35 feet.									
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.									
*Measured using Torvane and Pocket Penetrometer devices.				40					

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 21 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium sand, low to moderate plasticity, trace subangular gravel, some small roots.	Very Stiff to Hard	CL		0					
						39	14		>4.5
						53	15		>4.5
				5					
						34	14		>4.5
				10		37	16		>4.5
Low plasticity.									
				15		22	15		
Orange mottling, low to moderate plasticity.									
	Stiff								
				20		11	33		

Continued on Next Page

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium sand, moderate plasticity, trace gravel.	Very Stiff to Hard	CL		20 ▼					
▼ Ground water measured at 21 feet after drilling.									
● 79% Passing No. 200 Sieve.				25	●	20	29		2.0
				30		29	32		3.8
				35		59	31		4.0
				40		26	28		

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 21 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS✱ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium sand, moderate plasticity, trace gravel, some small roots.	Very Stiff	CL		40					
				45		67	28		
				50		90	29		>4.5
Bottom of Boring at 50 feet.									
				55					
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.									
*Measured using Torvane and Pocket Penetrometer devices.				60					

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 21 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Silt, moist, fine sand, low to moderate plasticity, some small roots. ■ Liquid Limit = 40, Plasticity Index = 19.	Hard	CL		0					
						39	17		>4.5
						48	2		>4.5
				5					
						40	9		
							9		
				10		32	11		>4.5
Light brown to gray, Clayey Gravel, moist, fine to coarse sand, fine to coarse subangular to round gravel.	Dense to Very Dense	GC							
				15		54	7		
				20		47	10		
Continued on Next Page									

EXPLORATORY BORING LOG EB-3
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 21 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Light brown to gray, Clayey Gravel, moist, fine to coarse sand, fine to coarse gravel, low plasticity. ▼ Ground water measured at 21 feet after drilling. Ground water encountered during drilling at 24 feet.	Dense to Very Dense	GC		20 ▼					
				25		60	32		
Brown, Sandy Lean Clay, fine to medium sand, moderate plasticity.	Very Stiff	CL							
				30		19	32		
				35		19	31		1.5
				40		16	25		

Continued on Next Page

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Sandy Lean Clay, fine to coarse sand, low to moderate plasticity.	Very Stiff to Hard	CL		40					
				45		20	25		
				50		76	35		
Bottom of Boring at 50 feet.									
				55					
				60					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 25 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy lean Clay, moist, fine to medium sand, low to moderate plasticity, trace subangular to round gravel.	Very Stiff to Hard	CL		0					
						33	14		
						49	13		
				5					
						37	13		>4.5
● 84% Passing No. 200 Sieve.				10	●	30	13		
				15					
	Stiff								
				20		12	30		0.5

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 25 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 08/30/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium sand, low to moderate plasticity, trace gravel. ▼ Ground water measured at 25 feet after drilling.	Very Stiff	CL		20					
				▼					
				30		27	29		2.5
Bottom of Boring at 30 feet.									
				35					
				40					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

EXPLORATORY BORING LOG EB-4
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

BORING EB-4

MARCH 2021
PROJECT NO. 5341-1

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 42 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Fill: Grayish brown, Sandy Lean Clay with gravel, slightly moist, fine to medium grained sand, fine angular gravel, low plasticity.	Very Stiff	CL		0		31	5		
Dark brown, Sandy Lean Clay, very moist, fine to medium grained sand, moderate to high plasticity. ■ Liquid Limit = 49, Plasticity Index = 25.	Stiff to Hard	CL				37	20		4.3
				5		38	19		4.5
Brown, increased sand content, low to moderate plasticity.						31	18		3.5
				10					
Fine rounded gravel.									
				15		34	28		3.0
				20		35	14		3.0

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 42 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS∗ (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Sandy Lean Clay, very moist, fine to medium grained sand, low to moderate plasticity.	Stiff to Hard	CL		20					
				25		60	25		4.5
				30		41	30		4.3
Fine to coarse sand, some fine to coarse gravel, wet.									
				35		30	23		1.0
				40		10	28		

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 42 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium grained sand, low to moderate plasticity.	Stiff to Hard	CL		**40**					
▼ Ground water measured at 42 feet after drilling.				▼					
Fine to coarse sand, fine to coarse angular-rounded gravel.									
				45		54	25		
Orange-brown color									
				50		50/5"	22		>4.5
Bottom of Boring at 49.9 feet.									
				55					
Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.									
*Measured using Torvane and Pocket Penetrometer devices.									
				60					

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 24 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Grayish brown, Sandy Lean Clay, moist, fine to coarse sand, fine angular to sub-angular gravel, low plasticity, (possible surface fill).	Hard	CL		0					
					Sample	56	17		
Brown, Sandy Lean Clay, very moist, fine to medium grained sand, moderate to high plasticity.	Hard	CL							
				5	Sample	38	21		3.8
					Sample	34	22		
				10					
Low to moderate plasticity, fine to medium sand.				15	Sample	44	27		
				20	Sample	40	21		3.5

Continued on Next Page

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 24 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium grained sand, low to moderate plasticity.	Hard	CL		20					
Fine angular to rounded gravel.									
▼ Ground water measured at 24 feet after drilling.				▼					
				25		73	24		>4.5
				30		50/6"	27		>4.5
				35		50/5"	26		>4.5
				40		50/6"	27		3.0

Continued on Next Page

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, very moist, fine grained sand, low to moderate plasticity.	Hard	CL		40					
				45		50/5"	27		
Bottom of Boring at 44.9 feet.									
				50					
				55					
				60					

Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual.

*Measured using Torvane and Pocket Penetrometer devices.

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 23 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Dark brown to brown, Sandy Lean Clay, moist, fine to medium grained sand, low plasticity.	Stiff to Hard	CL		0					
Trace fine to mediumangular to sub-angular gravel.						39	14		>4.5
						56	14		>4.5
				5					
						31	14		>4.5
Increase in fine to medium grained sand.						26	9		
				10					
● 69% Passing No. 200 Sieve.				15	●	24	11		
				20		15	12		
Continued on Next Page									

EXPLORATORY BORING LOG EB-3
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD)
SAN JOSE, CALIFORNIA

BORING EB-3

MARCH 2021
PROJECT NO. 5341-1

ROMIG ENGINEERS

DRILL TYPE: Mobile Drill B-61 with 8" Hollow Stem Auger **LOGGED BY:** LF

DEPTH TO GROUND WATER: 23 feet **SURFACE ELEVATION:** NA **DATE DRILLED:** 12/20/17

CLASSIFICATION AND DESCRIPTION	SOIL CONSISTENCY/ DENSITY or ROCK HARDNESS* (Figure A-2)	SOIL TYPE	SOIL SYMBOL	DEPTH (FEET)	SAMPLE INTERVAL	PEN. RESISTANCE (Blows/ft)	WATER CONTENT (%)	SHEAR STRENGTH (TSF)*	UNCONFIN. COMP. (TSF)*
Brown, Sandy Lean Clay, moist, fine grained sand, trace gravel, low plasticity. ▼ Ground water measured at 23 feet after drilling.	Very Stiff	CL		20					
Dark gray, Poorly Graded Sand, moist, fine to coarse grained sand, trace fine sub-angular to rounded gravel.	Medium Dense to Very Dense	SP		25		34	4		
● 8% Passing No. 200 Sieve.				30	●	27	9		
				35		58	17		
Bottom of Boring at 35 feet. Note: The stratification lines represent the approximate boundary between soil and rock types, the actual transition may be gradual. *Measured using Torvane and Pocket Penetrometer devices.				40					

APPENDIX B

LABORATORY TESTS

Samples from subsurface exploration were selected for tests to help evaluate the physical and engineering properties of the soils encountered at the site. The tests that were performed are briefly described below.

The natural moisture content was determined in accordance with ASTM D2216 on nearly all of the soil samples recovered from the borings. This test determines the moisture content, representative of field conditions at the time the samples were collected. The results are presented on the boring logs at the appropriate sample depths.

The Atterberg Limits were determined on two samples of soil in accordance with ASTM D4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of this test are presented in Figure B-1 and on the boring logs at the appropriate sample depth.

The amount of silt and clay-sized material present was determined on five samples of soil in accordance with ASTM D422. The results are presented on the boring logs at the appropriate sample depths.

An unconsolidated-undrained triaxial test was performed on one sample of soil in accordance with ASTM D2850. The result of this test is presented on Figure B-2.

❖ ❖ ❖ ❖ ❖

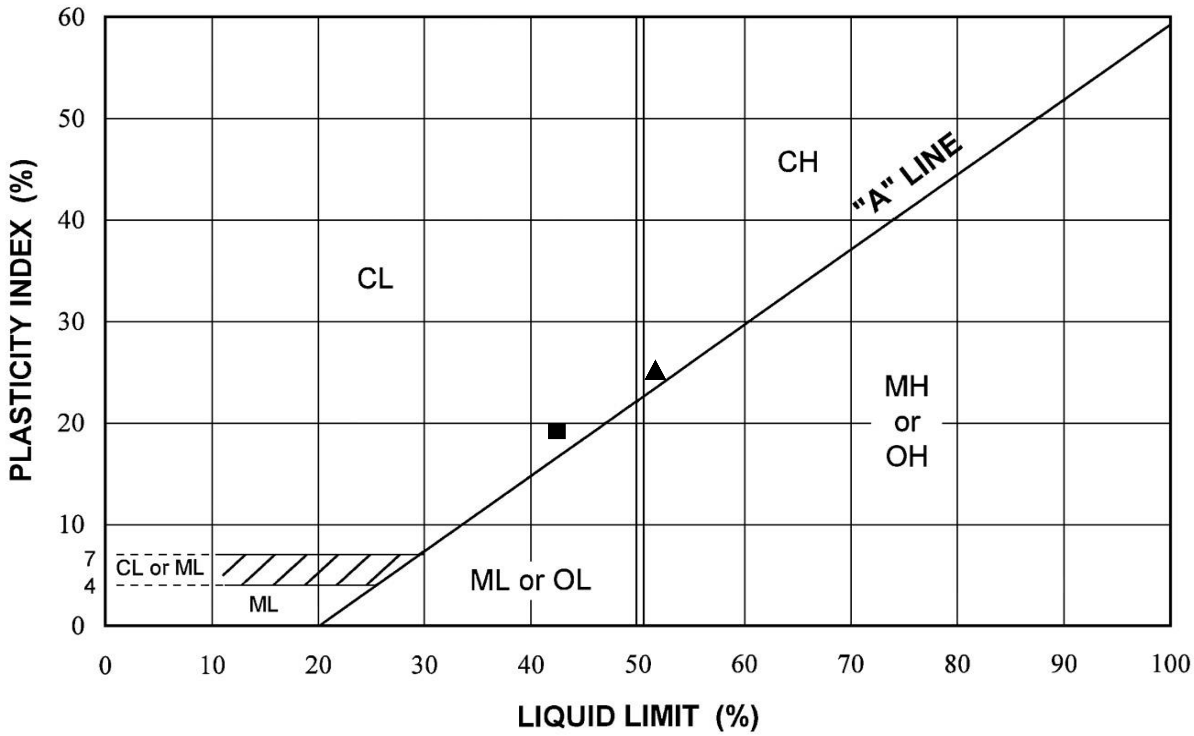
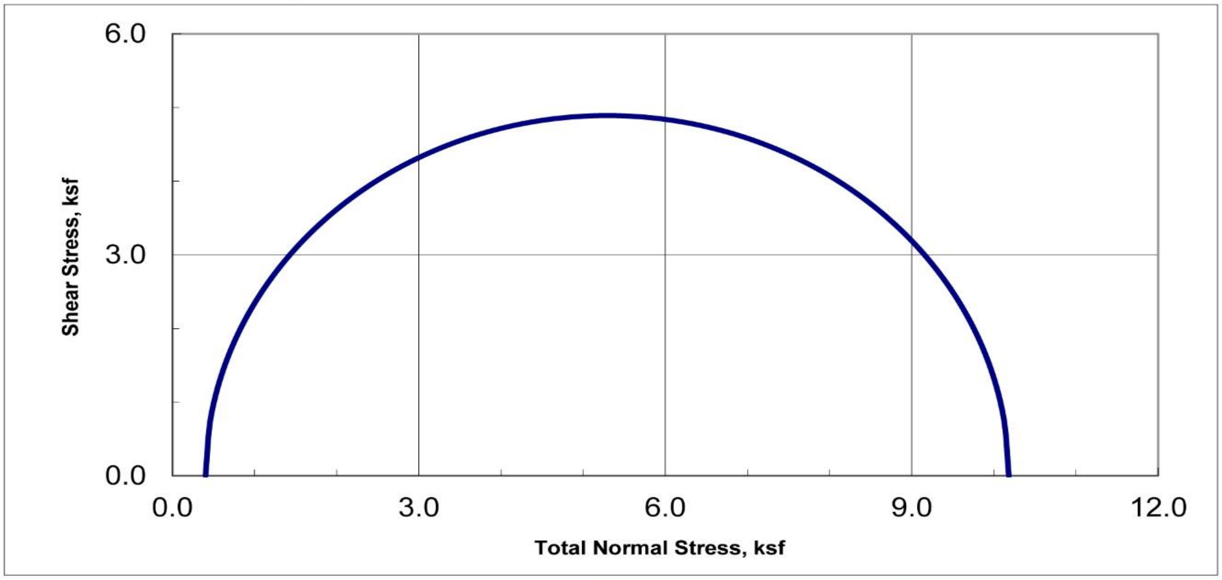


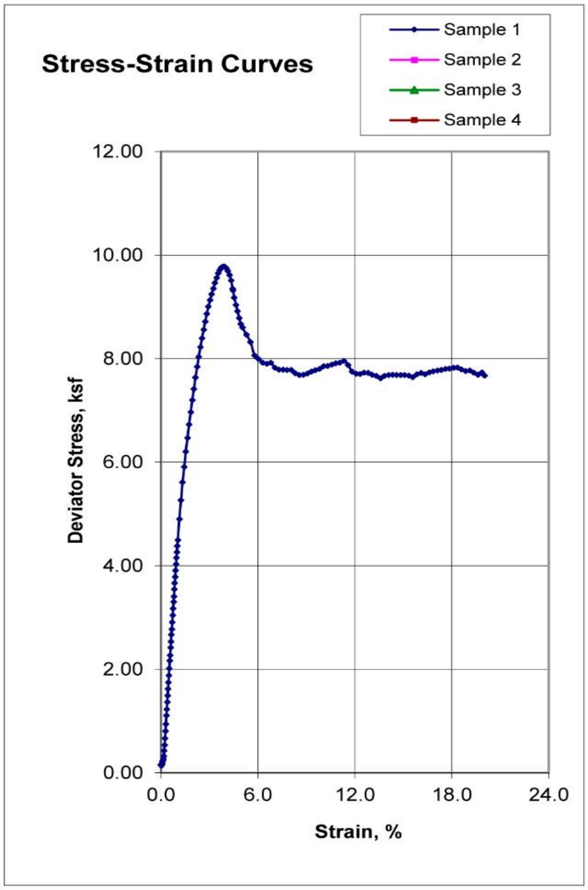
Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-3 (August, 2017)	1-2.5	17	40	19	-21		CL
▲	EB-1 (December, 2017)	3-4.5	20	49	25	-16		CL

PLASTICITY CHART **FIGURE B-1**
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD) MARCH 2021
SAN JOSE, CALIFORNIA PROJECT NO. 5341-1

ROMIG ENGINEERS

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	13.8			
Dry Den,pcf	113.7			
Void Ratio	0.482			
Saturation %	77.1			
Height in	5.00			
Diameter in	2.41			
Cell psi	2.8			
Strain %	3.94			
Deviator, ksf	9.780			
Rate %/min	1.00			
in/min	0.050			
Job No.:	**192-213**			
Client:	**Romig Engineers**			
Project:	**Piercy Hotel - 4167-1**			
Boring:	EB-4			
Sample:				
Depth ft:	6-6.5			

Visual Soil Description	
Sample #	
1	Olive Brown Clayey SAND
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST **FIGURE B-2**
WAREHOUSE BUILDING (459 AND 469 PIERCY ROAD) MARCH 2021
SAN JOSE, CALIFORNIA PROJECT NO. 5341-1

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