GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED COMMERCIAL RE-DEVELOPMENT

DURDEN CONSTRUCTION, INC
1096 LINCOLN AVENUE
SAN JOSE, CALIFORNIA

Prepared For:

Durden Construction, Inc.
Attn: Mr. John Durden
P.O. Box 966
San Juan Bautista, CA 95045

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P.O. Box 966
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SUBJECT: GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED COMMERCIAL RE-DEVELOPMENT
DURDEN CONSTRUCTION, INC.
1096 LINCOLN AVENUE
SAN JOSE, CALIFORNIA

Dear Mr. Durden:

As requested, we have performed a geologic and geotechnical study for the proposed commercial re-development of the property at 1096 Lincoln Avenue in the Willow Glen area of San Jose, California. The accompanying report presents the results of our study and testing, and our conclusions and recommendations concerning the geologic and geotechnical engineering aspects of the project. The findings and recommendations presented in this report are contingent upon our review of the final grading, foundation, and drainage control plans; our observation of the grading; and the installation of the foundation and drainage control systems.

This report includes information that is vital to the success of your project. We strongly urge you to thoroughly read and understand its contents. Please refer to the text of the report for detailed findings and recommendations.

Sincerely,
Upp Geotechnology
a division of C2Earth, Inc.

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

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1. INTRODUCTION

This report presents the results of our geologic and geotechnical study for the commercial re-development of your property at 1096 Lincoln Avenue (APN 264-56-082) in the Willow Glen area of San Jose, California (see Figure 1, Site Location Map). The purpose of our study was to explore the geotechnical/geologic conditions on the subject property in the area of the proposed improvements and to develop findings and recommendations for the earthwork and foundation engineering aspects of the proposed re-development.

At the time of our study, the property was vacant; the site was formerly occupied by a gas station that has since been razed. The underground gas storage tanks have also been removed and the excavation was backfilled. The current project involves constructing an approximately 9,600-square-foot, mixed-use commercial building on the southwestern half of the property. The northeastern half of the property will be occupied by parking stalls.

We issue this report with the understanding that the owner or owner's representative is responsible for ensuring that the information and recommendations contained in this report are brought to the attention of the project architect and engineer and are incorporated into the plans and specifications of the development. The owner must also ensure that the contractor and sub-contractors follow the recommendations during construction.

2. SCOPE OF SERVICES

We conducted this study in accordance with the scope and conditions presented in our proposal dated 21 October 2015 (Document Id. 15164C-01P1). The methodology of our evaluation is discussed in the body of this report. We make no other warranty, either expressed or implied. Our scope of services for this study included:

- reviewing selected geologic literature and aerial photographs to evaluate the prevailing geotechnical and geologic conditions;
- performing an engineering geologic reconnaissance of the site and site vicinity;
- preparing a site plan and a profile;
- conducting subsurface exploration;
- performing field and laboratory testing;
- analyzing geologic and geotechnical engineering properties from collected data;
- qualitatively evaluating settlement and earthquake-induced liquefaction; and
- preparing this report.

We have prepared this report as a product of our service for the exclusive use of Durden Construction, Inc. for the proposed re-development of the subject property. Other parties may not use this report, nor may the report be used for other purposes, without prior written authorization from Upp Geotechnology, a division of C2Earth, Inc (C2).

Because of possible future changes in site conditions or the standards of practice for geotechnical engineering and engineering geology, the findings and recommendations of this report may not be considered valid beyond three years from the report date, without review by C2. In addition, in the event that any changes in the nature or location of the proposed improvements are planned, the conclusions and recommendations of this report may not be considered valid unless we review such changes, and modify or verify in writing the conclusions and recommendations presented in this report.

Our study excluded an evaluation of hazardous or toxic substances, corrosion potential, chemical properties, and other environmental assessments of the soil, subsurface water, surface water, and air on or around the subject property. The lack of comments in this report regarding the above does not indicate an absence of such substances and/or conditions.

3. GEOLOGY AND SEISMICITY

We reviewed selected geologic maps and aerial photographs to evaluate the prevailing geologic conditions of the site and in the vicinity. The Regional Geologic Map and Regional Seismic Hazard Zones Map are presented on Figures 2 and 3, respectively.

3.1. Geology

The subject property is located in the flat lying alluvial basin of the Santa Clara Valley within the California Coast Ranges geomorphic province (see Figure 1). According to the Preliminary Geologic Map of the San Jose 30-minute by 60-minute Quadrangle, California (Wentworth et al., 1999), the subject site is underlain by older alluvial fan deposits of the Holocene epoch (approximately 10,000 years old or younger) (see Figure 2, Regional Geologic Map). The alluvial fan deposits are described as brown, unconsolidated, gravelly sand that grades upwards to silty clay. These deposits were placed by flooding streams and rivers.

3.2. Liquefaction, Lateral Spreading, and Cyclic Densification

Liquefaction is the temporary transformation of soil from a solid to a liquefied state. During cyclic loading, especially earthquake-induced loading, excess pore water pressure builds-up causing saturated soil to temporarily lose its shear strength. Soils susceptible to liquefaction include saturated loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Lateral spreading is a phenomenon in which surficial soil displaces along a slip surface that forms within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces. Earthquake shaking can also cause cyclic densification, where by, dry coarse-grained soil becomes more dense, resulting in vertical ground settlement.

The property is mapped near, but outside of current State seismic hazard zones for earthquake-induced liquefaction (see Figure 3, Regional Seismic Hazard Zones Map). These zones were established to minimize the loss of life and property by identifying and mitigating seismic hazards related to liquefaction and ground deformation. Because of the possibility of liquefaction and ground deformation, we have performed a qualitative evaluation of the liquefaction, lateral spreading, and cyclic densification hazards. The results of our evaluation are presented below in the Findings.

3.3. Seismicity

Geologists and seismologists recognize the greater San Francisco Bay Area as one of the most active seismic regions in the United States. The seismicity in the region is related to activity within the San Andreas fault system, a major rift in the earth's crust that extends for at least 700 miles along the California Coast. Faults within this system are characterized predominantly by right-lateral, strike-slip movement. The four major faults that pass through the Bay Area in a northwest direction have produced approximately 12 earthquakes per century strong enough to cause structural damage. These major faults are the San Andreas, Hayward, Calaveras, and San Gregorio faults.

The site can be expected to experience periodic minor earthquakes or even a major earthquake (Moment magnitude 6.7 or greater) on one of the nearby active or potentially active faults during the design life of the proposed project. The Moment magnitude scale is directly related to the amount of energy released during an earthquake and provides a physically meaningful measure of the size of an earthquake event.

The U.S. Geological Survey (2015) estimates that by 2044, the probability of a Moment magnitude 6.0 earthquake occurring on one of the active faults in the San Francisco region is 98%. The probability of a Moment magnitude 6.7 or greater earthquake occurring on one of the active faults in the San Francisco region is 72%. The following table provides corresponding estimates for the probability of a major earthquake (Moment magnitude 6.7 or greater) for three major faults in the Bay Area.

Fault	Probability (%)
Hayward	14.3
Calaveras	7.4
San Andreas	6.4

30-Year Probability of Magnitude 6.7 or Greater Earthquake

The following table indicates the approximate distance and direction from the site to active and potentially active faults.

Fault	Approx. Distance From Fault	Direction From Site
Hayward (southern extension)	6 miles	Northeast
Calaveras	9½ miles	Northeast
San Andreas	10 miles	Southwest
San Gregorio	25¼ miles	Southwest

Regional Fault Distances and Directions

According to the California State Special Studies Zones Map by the California Division of Mines and Geology, the site is mapped outside of the current Alquist-Priolo earthquake fault zones for areas prone to earthquake ground rupture. In addition, the site is not within a Santa Clara County geologic hazard zone corresponding to potential for fault rupture.

Because of the site's proximity to the Hayward and San Andreas faults and the site's geology, maximum anticipated ground shaking intensities for the area are characterized as very strong and equal to a Modified Mercalli (MM) intensity of VII (Borcherdt, et. al., 1975). An earthquake having a MM intensity of VII generally causes slight to moderate damage to well-built ordinary structures and negligible damage to specially designed earthquake-resistant structures (Yanev, 1974) (see Table I, Modified Mercalli Scale of Earthquake Intensities).

The intensity of an earthquake differs from the Moment magnitude, in that intensity is a measure of the effects of an earthquake, rather than a measure of the energy released. These effects can vary considerably based on the earthquake magnitude, distance from the earthquake's epicenter, and site geology.

Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the MM scale occurred east of the Monterey Bay on the San Andreas fault (Toppozada and Borchardt, 1998). The estimated Moment magnitude (M_w) for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of lives lost and cost of property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista, about 290 miles in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt as far away as Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta earthquake of 17 October 1989, occurring in the Santa Cruz Mountains, which had a M_w of about 6.9. Ground shaking equal to an MM intensity of between VI and VII was felt at the site during the Loma Prieta Earthquake (Stover, et al., 1990).

In 1868 an earthquake with an estimated maximum MM intensity of X and M_w of about 7.0 occurred on the southern segment of the Hayward fault, between San Leandro and Fremont. In 1861, an earthquake of unknown magnitude (likely having an M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill Earthquake, that had an M_w of about 6.2.

4. SITE CHARACTERIZATION

4.1. Regional Setting

We reviewed the aerial photographs and topographic maps for the site and vicinity. The site and area were developed with retail establishments in the early 1960's. The subject property is a square shaped, ½-acre lot. The topography across the site and vicinity is flat. The subject property is bounded to the southwest by Lincoln Avenue and to the southeast by Willow Street. Retail establishments border the northwest and northeast sides of the subject property.

4.2. Site Description

On 23 October 2015, our principal geologist/engineer performed a reconnaissance of the site and site vicinity. The subject property is undeveloped; the prior gas station structures had been previously removed. The approximate location of the prior gas station structures and the proposed improvements are shown on Figure 4, Site Plan and Engineering Geologic Map. The map is based upon an architectural site plan by Garcia Teague Architecture + Interiors, dated 7 October 2015, supplemented by field tape and compass measurements.

The site is covered with a thin layer of gravel except in areas where the prior asphalt pavement remains. Drainage across the site is generally characterized as uncontrolled sheet to the storm drainage facilities along Willow Street and Lincoln Avenue. In addition, several environmental monitoring wells are located around the property.

During our site reconnaissance, we observed evidence of damage to flatwork along and adjacent to Lincoln Avenue. The damage consisted of cracks in the street, sidewalk and curbs. Some of the cracks had been patched and subsequently re-opened. There was no notable evidence of damage to flatwork along or adjacent to Willow Street.

4.3. Subsurface

On 23 October 2015, our staff geologist visited the site to observe the subsurface conditions at discrete three locations in the vicinity of the proposed improvements. The approximate locations of the exploration test borings are shown on Figure 4. We determined the approximate boring locations by measuring distance and bearing from known points on the supplied site plan; these locations are only as accurate as implied by the mapping technique used. Our interpretations of subsurface conditions are depicted on Figure 5, Geologic Cross-Section A-A'. The following is a summary of the subsurface site characteristics.

Our staff geologist logged three test borings, one drilled to a depth of approximately 25 feet, and two drilled to depths of approximately 44½ feet, using a Mobile B-40 truck-mounted drill rig. We logged the borings in general accordance with the Unified Soil Classification System described on Figure 6, Key to Logs. A Summary of Field Sampling Procedures is presented on Figure 7. The boring logs are presented on Figures 8 through 15, Logs of Borings 1 through 3. The logs show our interpretation of the subsurface conditions at the locations and on the date indicated. We do not warrant that they are representative of the subsurface conditions at other locations and times.

The borings were excavated within the approximate proposed building footprint. Boring 1 was drilled in the central southeast portion of the site, Boring 2 was drilled in the central portion of the site, and Boring 3 was drilled near the northwest property corner, see Figure 4.

In general, all of the excavations encountered a similar sequence of layered alluvial deposits (alluvium). Boring 2, encountered two distinct layers of fill over the alluvium. The upper layer of fill was about 5½ feet thick and consisted of dense sandy gravel. This fill layer was also encountered in Boring 1; an approximately ¾ foot thick layer of fill was encountered over the alluvium. Below the upper layer of fill in Boring 2 was a second layer of fill about 9 feet thick that was composed of loose silty gravel. These fill layers appear to have been placed as part of the backfill process associated with the removal of the underground storage tanks.

All of the borings encountered layered alluvium that persisted to the bottom of each boring. The upper approximately 28 feet of the alluvium consisted of weak brown to black, firm to stiff, silt and clay. In both Borings 2 and 3 below a depth of about 28 feet, the alluvium was coarse-grained, medium dense to very dense, sand and gravel.

While our study excluded an evaluation of hazardous substances, some of the samples obtained and cuttings generated during the test drilling program emitted a strong odor consistent with high concentrations of volatile organic compounds (VOC's). The odor was strongest in the samples and cuttings within the alluvium in Boring 2.

4.4. Groundwater

We did not encounter groundwater in any of the borings. Fluctuations in the level of subsurface water could occur due to variations in rainfall, temperature, and other factors not evident at the time our observations were made. According to the map of depths to historically high groundwater presented in the Seismic Hazard Report for the San Jose West 7.5-Minute Quadrangle (California Division of Mines and Geology, 2002), the depth to groundwater beneath the site is greater than 40 feet.

4.5. Laboratory Testing

We developed our laboratory testing program to supplement our evaluation of the geotechnical engineering properties of the soil at the site. We retained soil samples from the borings for laboratory classification and testing. The results of moisture content and dry density tests are presented on the logs.

5. FINDINGS

Based upon the results of our study, it is our opinion that, from a geotechnical engineering perspective, the subject property may be re-developed as planned, provided that the recommendations presented in this report are incorporated into the design and construction of the proposed improvements. In our opinion, the primary constraints to the proposed development include:

- the presence of undocumented backfill related to the removal of former underground storage tanks;
- the presence of VOC's within the subsurface materials and the potential for corrosion;
- the potential for differential foundation settlement from subsurface deformation of weak alluvium layers; and
- the site's seismic setting.

5.1. Proposed Building Site

As currently planned, the approximately 9,600-square-foot, single story, mixed-use retail and restaurant building is planned in the southwestern half of the property. A parking lot is planned for the northeastern portion of the site. The development will be accessed by a driveway from Willow Street near the southeast corner of the property.

The prior gas station buildings on the site have been razed, and the underground storage tanks have been removed. The tank excavation was backfilled with undocumented fill that appears to consist of compacted fill over loose silty gravel. The backfilled excavation underlies the central portion of the proposed building and the fill may undergo differential settlement.

As discussed above in the section titled "Subsurface", the property and building area beneath the backfill appear to be underlain by relatively horizontal layers of alluvium. The upper approximately 28 feet of the alluvium consists of fine-grained materials having relatively low SPT blow counts (generally less than 10, except for boring B3 where the values were slightly higher). Below the fine-grained alluvium is a layer of coarse-grained, medium dense alluvium.

In our opinion, the coarse-grained alluvium encountered at depth or improvement of the shallow fine-grained alluvium will provide adequate support for the foundation of the proposed building.

It should also be noted that the building site is underlain by subsurface material that contain VOC's. These chemicals compounds could present an environmental hazard and/or cause an increased rate of corrosion. The designers should account for these conditions in their design.

5.2. Foundation Settlement

The property is located in the Willow Glen area of San Jose. Historically, the Willow Glen area was a marshy area cut by meandering rivers, sloughs, and tributary streams. The site area is underlain by stream and river deposits and undocumented fill. We have observed numerous instances of localized soil settlement within the Willow Glen area. This settlement manifests as deformations to building foundations and undulations in the streets and sidewalks, which were presumably built level.

Based upon our experience, we generally believe that this settlement is the result of chemical and physical changes in soil that was deposited and buried in a former alluvial plain, flood plain, or swampy environment. The depth of soil subject to this settlement is thought to be confined to weak fine-grained soil layers. Processes such as significant groundwater elevation fluctuations, fill settlement, consolidation, expansive soil, hydro-compaction of collapsible soil, and/or the decomposition of organic-rich soil may also be contributing to the observed settlement.

Elimination of the foundation movement under these soil conditions is difficult, because the fine-grained alluvium could be subject to settlement, shrinkage, and possibly swelling. Mitigating the risk of settlement to tolerable levels requires either 1) performing ground improvements in the upper fine-grained alluvial layers or 2) supporting structures on deep foundations bearing in the medium dense sand and gravel alluvial layers below 28 feet beneath the ground surface. Recommendations for both options are presented in the following section.

It should be noted that ground improvements or deep foundations will reduce the risk for differential movement to affect the structural integrity of the proposed improvements. However, minor cosmetic damage and distress may occur and may require periodic maintenance.

5.3. Liquefaction, Lateral Spreading, and Cyclic Densification

As mentioned above, the site is mapped near a State of California seismic hazard zone for earthquake-induced liquefaction. Earthquake shaking can cause dynamic (vertical) ground settlement or bearing capacity failure of saturated soil. Based upon the results of our subsurface evaluation, we did not observe evidence of current or sustained prior groundwater within the upper 45 feet of the underlying ground surface. In our opinion, based on the lack of groundwater in the upper alluvial layers beneath the site, we judge there is negligible risk for liquefaction related bearing capacity failure or ground surface deformations.

Lateral spreading occurs as surficial soil displaces along a slip surface that has formed within an underlying, continuous liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a creek channel, by earthquake and gravitational forces. Because the potential for liquefaction is negligible, we also judge the potential for lateral spreading at the site is negligible.

Earthquake shaking can also cause cyclic densificaton and vertical ground settlement of dry sandy soil. Because the upper 28 feet of the alluvium consists of fine-grained soil and is underlain by medium dense coarse-grained soil, in our opinion, there is a negligible risk for ground surface deformation from cyclic densification.

5.4. Seismicity

Our reconnaissance and review of published geologic maps and aerial photographs revealed that no known active or potentially active faults pass through the subject property. However, it is reasonable to assume that the site will be subjected to strong ground shaking from a major earthquake on at least one of the nearby active faults during the design life of future improvements. During such an earthquake, it is our opinion that the danger from fault offset through the site is negligible.

6. RECOMMENDATIONS

Because the proposed project is still in a relatively early phase of development, it is conceivable that changes and additions will be made to the proposed redevelopment concept following submission of this report. We recommend that as various changes and additions are made, you contact us to evaluate the geotechnical aspects of these modifications.

To reduce the magnitude of potential settlement at the site to tolerable levels, we recommended the proposed building be supported on grade beams that are structurally supported by deep foundations extending to the coarse-grain alluvial layers encountered in our borings at about 28-feet below the ground surface. As an alternative, conventional foundations may be used in areas where in-situ ground improvement measures are used to densify the upper fine-grained alluvial layers.

Differential settlement may occur within the parking lot area planned for the northeastern side of the site and pavements in this area may become distressed and require sealing, maintenance, or periodic repaving. If this risk is unacceptable, deep foundation elements or in-situ ground improvement measures may be used in this area of the site as well.

The following recommendations must be incorporated into all aspects of future development.

6.1. Location of Proposed Improvements

The proposed improvements must be confined to the approximate building area shown on Figure 4. Do not construct improvements outside of this generalized area without written approval from C2. If other structures are planned in the future, we must evaluate their location to provide appropriate geotechnical engineering design criteria.

6.2. Seismic Design Criteria

We recommend that the project structural design engineer provide appropriate seismic design criteria for proposed foundations and associated improvements. The following information is intended to aid the project structural design engineer to this end and is based on criteria set forth in the 2013 California Building Code (CBC). The mapped spectral accelerations and site coefficients were computed using the USGS Seismic Design Maps tool with the 2010 ASCE 7 design code reference (updated 2013).

Design Parameters

Latitude = 37.3089º
Longitude = -121.9010º
Site Class = D
$S_s = 1.500$
$S_1 = 0.600$
$F_a = 1.0$
$F_v = 1.5$

Experience has shown that earthquake-related distress to structures can be substantially mitigated by quality construction. We recommend that a qualified and reputable contractor and skilled craftsmen build the associated improvements. We also recommend that the project structural design engineer and project architect monitor the construction to make sure that their designs and recommendations are properly interpreted and constructed.

6.3. Earthwork

At the time of this study, the full extent of any proposed earthwork had not been finalized. We anticipate that a minor amount of grading will be required to prepare the subgrade for the building and parking areas and for the installation of new below-grade utilities. Any proposed earthwork should be performed in accordance with the recommendations provided below.

6.3.1. Fill Material

- Based on our study, it is our opinion that the materials encountered in the borings are not suitable for use as engineered fill. Imported materials must meet the requirements specified below to be used as engineered fill.
- On-site materials may be off-hauled from the site or used for on-site landscaping purposes only.
- Import materials used for engineered fill must meet the following requirements:
  1) Have an organic content less than 3% by volume;
  2) no rocks or lumps greater than 6 inches in maximum dimension;
  3) no more than 15% of the fill may be greater than 2½ inches in maximum dimension; and
  4) have a plasticity index (PI) of 15% or less.
- **Contact C2 with samples of proposed fill materials at least four days prior to fill placement for laboratory testing and evaluation.**

6.3.2. Compaction Procedures

- Prior to fill placement, scarify the surface to receive the fill to a depth of 6 inches.
- Moisture condition the imported fill to the materials' approximate optimum moisture content.
- Spread and compact the fill in lifts not exceeding 8 inches in loose thickness.
- Compact the fill to at least 90% relative compaction by the Modified Proctor Test method, in general accordance with the ASTM Test Designation D1557 (latest revision).
- **Contact C2 to observe the placement and test the compaction of engineered fill.** Provide at least two working days notice prior to placing fill.

6.3.3. Trench Backfill

- Backfill all utility trenches with compacted engineered fill.
- Place suitable on-site soil into the trenches in lifts not exceeding 8 inches in uncompacted thickness, and compact it to at least 90% relative compaction by mechanical means only.
- If imported sand is used, compact it to at least 90% relative compaction. Do not use water jetting to obtain the minimum degree of compaction in imported sand backfill. If the trench is greater than 50 feet long, located on sloping ground greater than 5:1 (horizontal to vertical), and is backfilled with sand, check dams should be installed to reduce the potential of the sand washing out.

- Compact the upper 6 inches of trench backfill to at least 95% relative compaction in all pavement areas.
- **Contact C2 to observe and test compaction of the fill.**

6.4. In-Situ Ground Improvement

As discussed above, the upper fine-grained alluvial soil layers may experience differential settlement related to the chemical and physical changes to the soil structure. One option for mitigating the potential for distress to the building (and parking area) is to improve the material strength of the upper fine-grained alluvial soil layers. We considered several methods, and judge that compaction grouting or stone-columns are appropriate for the site conditions.

The area to be improved should extend 5 feet laterally beyond the footprint of the structure, where feasible. In addition, ground improvement process will disrupt the ground surface and shallow subsurface. Consequently, following the installation of the ground improvement, the upper 4 feet of soil should be removed and replaced with imported engineered fill, placed in accordance with the Earthwork recommendations presented above. If other methods of in-situ ground improvements are planned, we must be contacted to develop appropriate recommendations.

6.4.1. Compaction Grouting

Compaction grouting consists of pumping a low slump (less than two inches) soil-cement grout under high pressure through steel grout pipes. The grout does not penetrate the soil voids, but rather expands under pressure to form a bulb up to two feet in diameter. The expansion of the hole causes compression and densification of the surrounding soil. The grout columns also act to reinforce the soil as vertical members.

- Compaction grouting should generally be performed on a grid pattern with injection points spaced approximately 4 to 8 feet on center (in plan view); however, the injection point spacing should be established by a specialty contractor following the results of a test section, which is described in the subsequent section.
- Contact C2 evaluate the results of the production operation, by performing Standard Penetration Tests (SPTs) in test borings or by performing Cone Penetration Tests (CPTs). The locations of the tests will be determined after production operation.
- If SPTs are used for quality control, the SPTs should be performed at a 3-foot intervals or less in each test hole. The improved soil should have minimum and average SPT blow counts (normalized to an overburden pressure of 1 tsf and corrected for field conditions), over three consecutive runs, of at least 20 and 25 blows per foot, respectively.
- If CPTs are used for quality control, the improved soil should have minimum and average tip resistances (normalized to an overburden pressure of 1 tsf and

corrected to account for the effects of fines content), over an interval of 3 feet, of at least 80 and 100 tons per square foot (tsf), respectively.

- Prior to proceeding with production work, the contractor should perform a test section to show that the required degree of improvement can be achieved. The compaction grouting should be performed using the same equipment and procedure planned for the production operation. Two SPT borings or CPTs should be performed to evaluate the effectiveness of the compaction grouting after the test section work is completed.

- Prior to performing the test section, the contractor should submit a test program for our review. The test program should include the equipment and procedure to be used for compaction grouting, the spacing, and the schedule.

- **Contact C2 to evaluate the results of the test program and make recommendations for the production operation, as appropriate.**

6.4.2. Stone Columns

Stone columns are installed using a vibrating, cylindrical-shaped probe that is advanced to the desired depth of improvement using water or air jetting. The voids created through the densification of the surrounding soil are backfilled with compacted gravel or crushed rock while withdrawing the probe. This procedure creates dense stone columns typically 3 to 4 feet in diameter surrounded by densified soil. The stone columns serve as drains to allow rapid dissipation of pore pressure that may develop in the native soil during an earthquake. In addition, the stone columns reinforce the soil.

- Stone columns should be installed by a specialty contractor experienced in this type of construction.

- The stone columns should be at least 30 feet deep. Backfill for stone columns should consist of gravel or crushed rock generally between 3/8 and 2 inches in diameter with less than 10 percent by weight passing the # 4 sieve. Submit a sample or gradation test results of the proposed backfill material for our testing or review, prior to the work.

- For preliminary estimating, the stone columns should be assumed to be at least 3 feet in diameter and installed in triangular pattern at spacing no greater than 10 feet. The actual diameter and spacing of the columns should be determined by the specialty contractor following a test program, which is described in the next section.

- We should evaluate the results of the production operation, by performing Standard Penetration Tests (SPTs) in test borings or by performing Cone Penetration Tests (CPTs). The locations of the tests will be determined after the production operation.

- If SPTs are used for quality control, they should be performed at a 3-foot interval or less in each test hole. The improved soil should have minimum and average SPT blow counts (normalized to an overburden pressure of 1 tsf and corrected for field conditions), over three consecutive SPTs, of at least 20 and 25 blows per foot, respectively.

- If CPTs are used for quality control, the improved soil should have minimum and average tip resistance (normalized to an overburden pressure of 1 tsf and corrected to account for the effects of fines content), over a depth of 3 feet, of at least 80 and 100 tons per square foot (tsf), respectively.

- Prior to proceeding with production work, the contractor should perform a test program to show that the required degree of improvement can be achieved by the proposed procedure. The stone columns for the test program should be installed using the same equipment planned for the production operation. The test program should consist of three sections, each with a different spacing. A total of 3 SPT borings or CPTs should be performed to evaluate the effectiveness of the stone columns after the stone column installation.

- Prior to installing the test section, the contractor should submit a test program for our review. The test program should include the equipment and procedure to be used for stone column installation, the probe spacing, and the schedule.

- **Contact C2 to evaluate the results of the test program and make recommendations for the production operation, as appropriate.**

- During soil improvement by stone columns, the ground surface could settle or heave. The ground movements will not be uniform across the improved areas. Ground movements may also occur beyond the immediate work area. The movements of adjacent structures should be monitored during performance of the test program and production operation. This monitoring is necessary to check that the ground movements will not adversely affect the performance of the foundations of the nearby structures.

6.5. Foundations

We recommend the proposed building be supported on grade beams that are structurally supported by deep foundations extending to the coarse-grain alluvial layers encountered in our borings at about 28 feet below the ground surface. Based on the site conditions, we recommend that the deep foundation elements consist of helical piers or pipe-piles. As an alternative, conventional spread footing foundations may be used in areas where in-situ ground improvement measures are used to densify the upper fine-grained alluvial layers.

In our opinion, a foundation supported on engineered fill over improved ground or on deep foundation elements that are designed and constructed in accordance with the following recommendations will reduce the risk of differential movement to a tolerable level.

We recommend that your engineer design and your contractor construct the proposed foundation elements in accordance with the following recommendations.

6.5.1. Helical Piers or Pipe-Piles

- Install helical piers or pipe piles to a depth of 30 feet (within a tolerance of 2 feet) to transfer building loads to the coarse-grained alluvial layers.
- Connect the tops of helical piers or pipe piles to grade beams in accordance with the designer's recommendations. If other isolated shallow footings are to be used for exterior signs or trellises, a bearing block should be constructed around the tops of the anchors.
- A structural engineer experienced in the use of helical piers or pipe piles should determine the loading at each pier location and develop bearing requirements at each anchor location.
- Design for lateral load resistance using batter helical piers or pipe piles, as the structural engineer deems appropriate.
- Consider the corrosion potential from high concentrations of VOC's and design as the structural engineer deems appropriate.
- The structural engineer should develop a testing regimen to verify pipe piles or helical anchors will have sufficient bearing at their install depth.
- **Contact C2 to observe the helical piers or pipe piles as they are being installed** to verify that the anchors are founded in material of sufficient depth and supporting capacity.

6.5.2. Grade Beams

- Reinforce grade beams with top and bottom reinforcement to provide structural continuity and to permit the spanning of local irregularities.
- Provide good structural continuity between the grade beam and the helical piers or pipe piles.
- The structural design engineer must determine the actual size and reinforcement of the grade beams.
- Remove any concrete overpour before the concrete has achieved its design strength.

6.5.3. Conventional Spread Footings

- Embed spread footings a minimum of 12 inches into engineered fill over improved ground.
- Design the spread footings supported in engineered fill for an allowable bearing pressure of 2,500 psf for dead plus live loads, with a 1/3 increase for transient loads, including wind and seismic.

- All footings adjacent to utility trenches must have their bearing surface below an imaginary plane projected upward from the bottom edge of the trench at a 1:1 (horizontal to vertical) slope.
- Lateral loads may be resisted by friction between the foundation bottoms and the supporting subgrade using a friction coefficient of 0.35.
- The structural design engineer must determine concrete reinforcing; but, as a minimum, all continuous footings must be provided with at least two No. 4 steel reinforcing bars, one placed at the top and one placed at the bottom of the footing, to provide structural continuity and to permit the spanning of any local irregularities.
- Clear the bottoms of the footing excavations of loose cuttings and soil fall-in prior to the placement of concrete.
- **Contact C2 to observe the footing excavations prior to placing reinforcing steel** to evaluate depth into supportive material.

6.5.4. Parking Lot and Sidewalks

We anticipate that the proposed parking lot and new sidewalks will be constructed using flexible pavement (asphalt) or concrete slabs-on-grade pavement. As noted above, these areas are also subject to differential ground settlement that could result in distress to pavement areas. If distress to these areas are not acceptable, parking lot and sidewalk areas could be improved by the above recommended in-situ ground improvement methods and/or the sidewalk slabs could be structurally supported on deep helical piers or pipe piles.

6.5.5. Flexible Pavement (Asphalt)

The following recommendations are based upon an anticipated Traffic Index (TI) of 3. If a greater TI is required for the project, contact C2 for appropriate recommendations. For flexible pavement we recommend the following minimum requirements:

- Scarify and re-compact the upper of 6 inches of the sub base to the requirements for fill given above.
- Use a minimum pavement section of 2 inches of asphalt over 6 inches of CalTrans Class II baserock compacted to at least 95% relative compaction in accordance with the requirements for engineered fill given above.
- **Contact C2 to observe and test compaction of the sub base recompaction and baserock compaction.**

6.5.6. Rigid Pavement (Concrete)

Concrete pavement has a higher compressive strength than flexible pavement, but is more susceptible to cracking and cosmetic damage. In our opinion, cosmetic damage to the concrete should not compromise the ability of the pavement section to support the required loading. If the visual or aesthetic performance of the pavement section is of concern, contact us to perform additional laboratory testing and develop additional recommendations, if necessary.

At a minimum, we recommend that a rigid pavement section consist of 4 inches of reinforced concrete over 12 inches of CalTrans Class II baserock. Design and construct the rigid pavement section in accordance with the following:

- Scarify and re-compact the upper of 6 inches of the sub base to the requirements for engineered fill given above.
- Compact CalTrans Class II baserock to at least 95% relative compaction in accordance with the requirements for engineered fill given above.
- Proof-roll the surface of the non-expansive fill to provide a smooth, firm surface for slab support prior to placement of reinforcing steel.
- Design slab reinforcement in accordance with anticipated use and loading, but at a minimum, reinforce slabs with No. 3 rebar on 18-inch centers each way, placed mid-height in the slab.
- Support the reinforcing from below on concrete blocks (or similar) during concrete pouring to make sure that it remains mid-height in the slab.
- Place grooves in the concrete slabs at 10-foot intervals, or in accordance with the structural design engineer's recommendations, to help control cracking.

6.6. Drainage

Control of surface drainage is critical to the successful performance of the proposed improvements. The results of improperly controlled runoff may include foundation heave and/or settlement, gullying, or ponding. To mitigate the risk of improperly controlled runoff, we recommend that you implement the following:

- Prevent surface water from ponding in areas adjacent to the foundation of the proposed structure and associated improvements by grading adjacent areas to create proper drainage by sloping them away from the structures.
- As an alternative, install area drains to collect surface runoff.
- Provide roof gutters with downspouts on the structures.
- Do not allow water collected in the gutters to discharge freely onto the ground surface adjacent to the foundation.
- Convey water from downspouts and/or area drains away from the structure via buried, closed conduits or lined surfaces.
- Discharge collected water into the local stormwater system in an appropriate manner under the direction of the project civil engineer.

- Use buried conduits consisting of rigid, smooth-walled pipes (PVC). **<u>Do not use flex-pipes</u>.** As an alternative, downspouts may discharge onto pavement areas that slope away from the building.
- Provide downspouts with slip-joint connectors or clean-outs, where they are connected to buried pipes, to facilitate maintenance.
- Perform annual maintenance of the surface drainage systems, including:

    1) Inspecting and testing roof gutters and downspouts to make sure that they are in good working order and do not leak;
    2) inspecting and flushing area drains to make sure that they are free of debris and are in good working order; and
    3) inspecting surface drainage outfall locations to verify that introduced water flows freely through the discharge pipes.

7. PLAN REVIEW AND CONSTRUCTION OBSERVATION

We must be retained to review the final grading, foundation, and drainage control plans in order to verify that our recommendations have been properly incorporated into the proposed project. **<u>WE MUST BE GIVEN AT LEAST ONE WEEK TO REVIEW THE PLANS AND PREPARE A PLAN REVIEW LETTER.</u>**

We must also be retained to observe the grading and the installation of foundations and drainage systems in order to:

- verify that the actual soil conditions are similar to those encountered in our study;
- provide us with the opportunity to modify the foundation design, if variations in conditions are encountered; and
- observe whether the recommendations of our report are followed during construction.

Sufficient notification prior to the start of construction is essential, in order to allow for the scheduling of personnel to insure proper monitoring.

<u>WE MUST BE NOTIFIED AT LEAST TWO WEEKS PRIOR TO THE ANTICIPATED START-UP DATE. IN ADDITION, WE MUST BE GIVEN AT LEAST TWO WORKING DAYS NOTICE PRIOR TO THE START OF ANY ASPECTS OF CONSTRUCTION THAT WE MUST OBSERVE.</u>

The phases of construction that we must observe include, but are not necessarily limited to, the following.

1. **EARTHWORK:** During construction to observe and document ground improvement and to test compaction of engineered fill
2. **HELICAL PIER or PIPE PILE INSTALLATION:** During installation to evaluate whether the anchors are founded in material of sufficient supporting capacity

3. **FOOTING EXCAVATION:** Prior to placement of reinforcing steel to evaluate depth to supportive material
4. **SLABS-ON-GRADE AND FLEXIBLE PAVEMENT:** Prior to and during placement of non-expansive fill to observe the subgrade preparation and to test compaction of non-expansive fill
5. **SURFACE DRAINAGE SYSTEMS:** Near completion to evaluate installation and discharge locations

* * * * * * * * *

A Bibliography, a List of Aerial Photographs, and the following Figures and Table are attached and complete this report.

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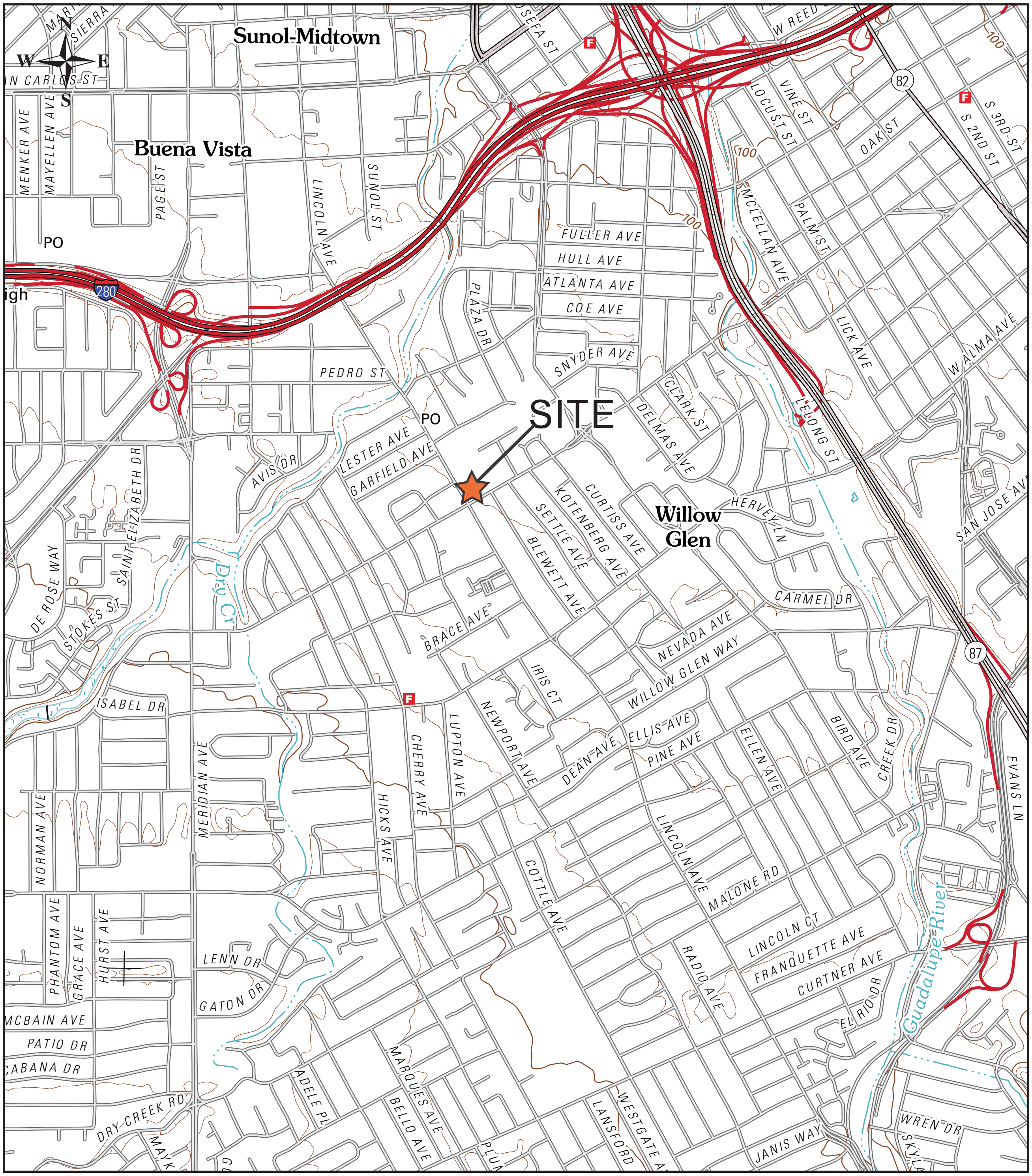
LIST OF AERIAL PHOTOGRAPHS

"BAY AREA TRANSPORTATION STUDY", black and white, dated May 16, 1965, at a scale of 1" = 1,000', Aerial Survey Contract No. 67615, Serial Nos. SCL 11-85 and SCL 11-86, State of California Highway Transportation Agency, Division of Highways.

FIGURES AND TABLE

FIGURE NO.

TABLE NO.



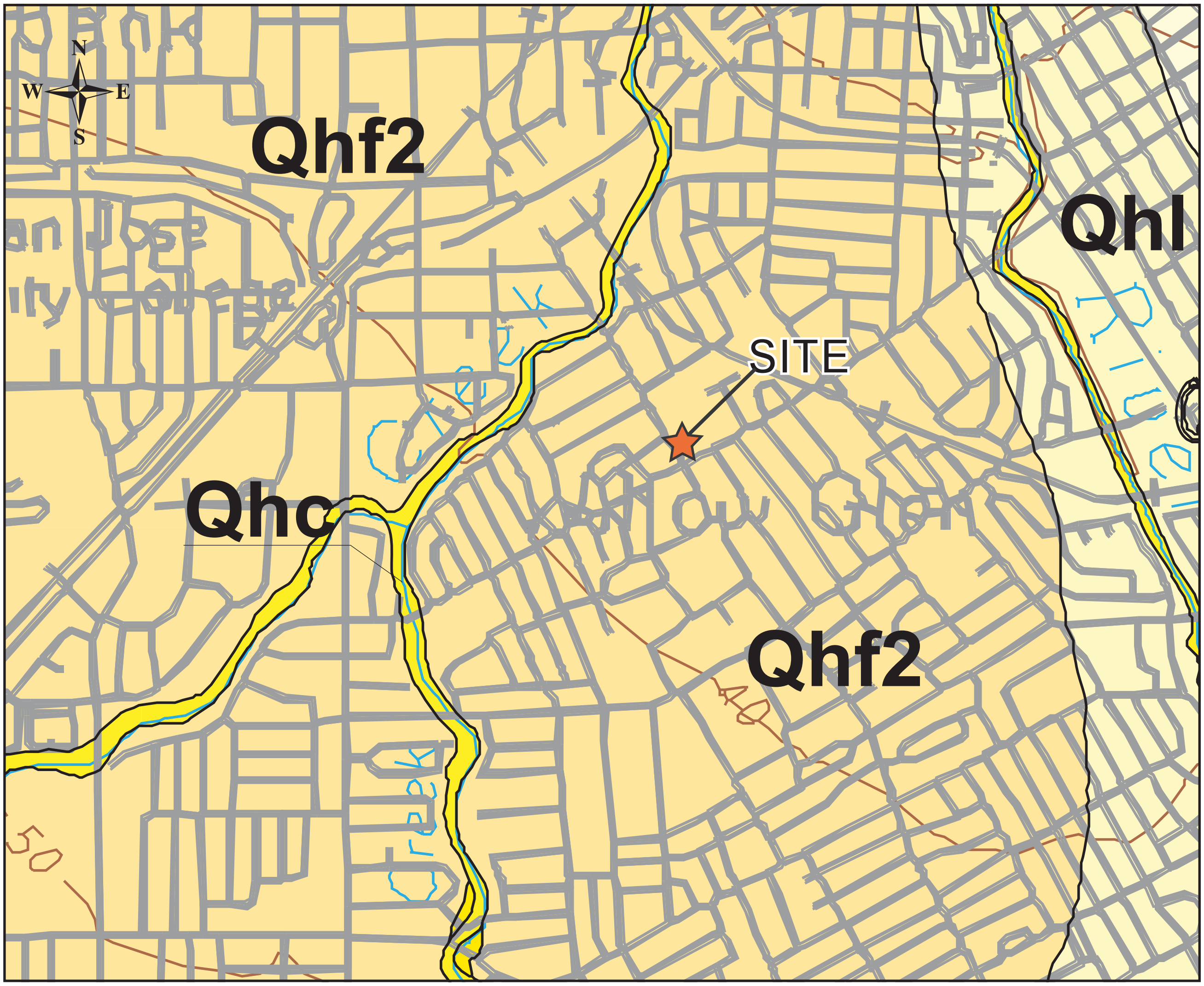
BASE: The National Map US Topo; UNITED STATES GEOLOGICAL SURVEY; 2012

SITE LOCATION MAP

C2EARTH

DURDEN CONSTRUCTION, INC.
1096 Lincoln Avenue
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
RH/CR	1" = 2,000'	15164C-01R1	February 2016	Figure 1



EXPLANATION

Qhc - Stream Channel Deposits (Holocene)
Qhl - Levee Deposits (Holocene)
Qhf2 - Older Alluvial Fan Deposits (Holocene)

Geologic contact
dashed where approximate
and dotted where concealed

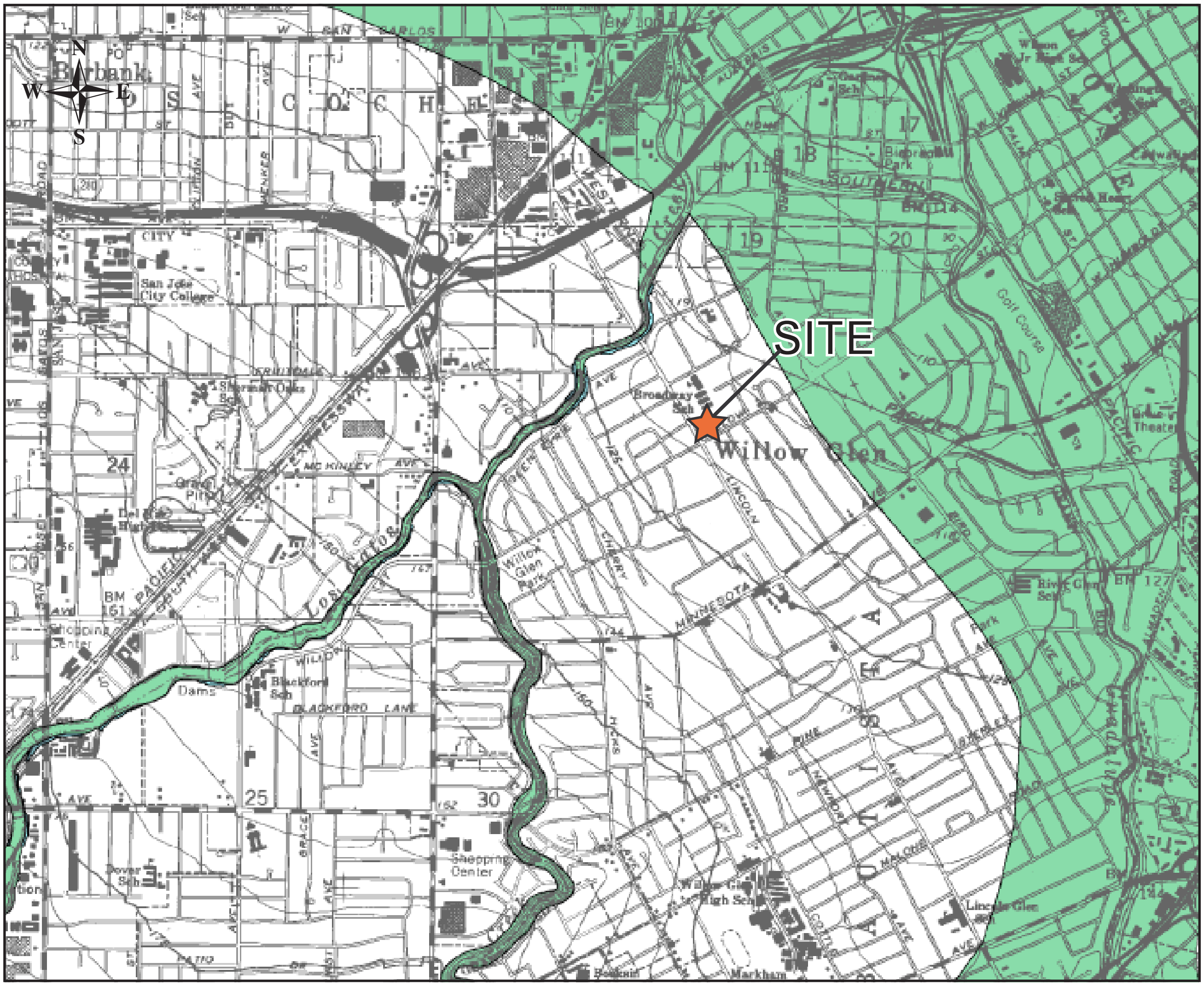
BASE: Preliminary Geologic Map of the San Jose 30 X 60-Minute Quadrangle, California; WENTWORTH, ET AL.; 1999

REGIONAL GEOLOGIC MAP

C2EARTH

DURDEN CONSTRUCTION, INC.
1096 Lincoln Avenue
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
RH/CR	1" = 2,000'	15164C-01R1	February 2016	Figure 2



EXPLANATION

- **Earthquake-Induced Landslides;** Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

- **Liquefaction;** Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

BASE: Seismic Hazard Zones, San Jose West Quadrangle; California Geological Survey; 7 February 2002

REGIONAL SEISMIC HAZARD ZONES MAP

C2EARTH

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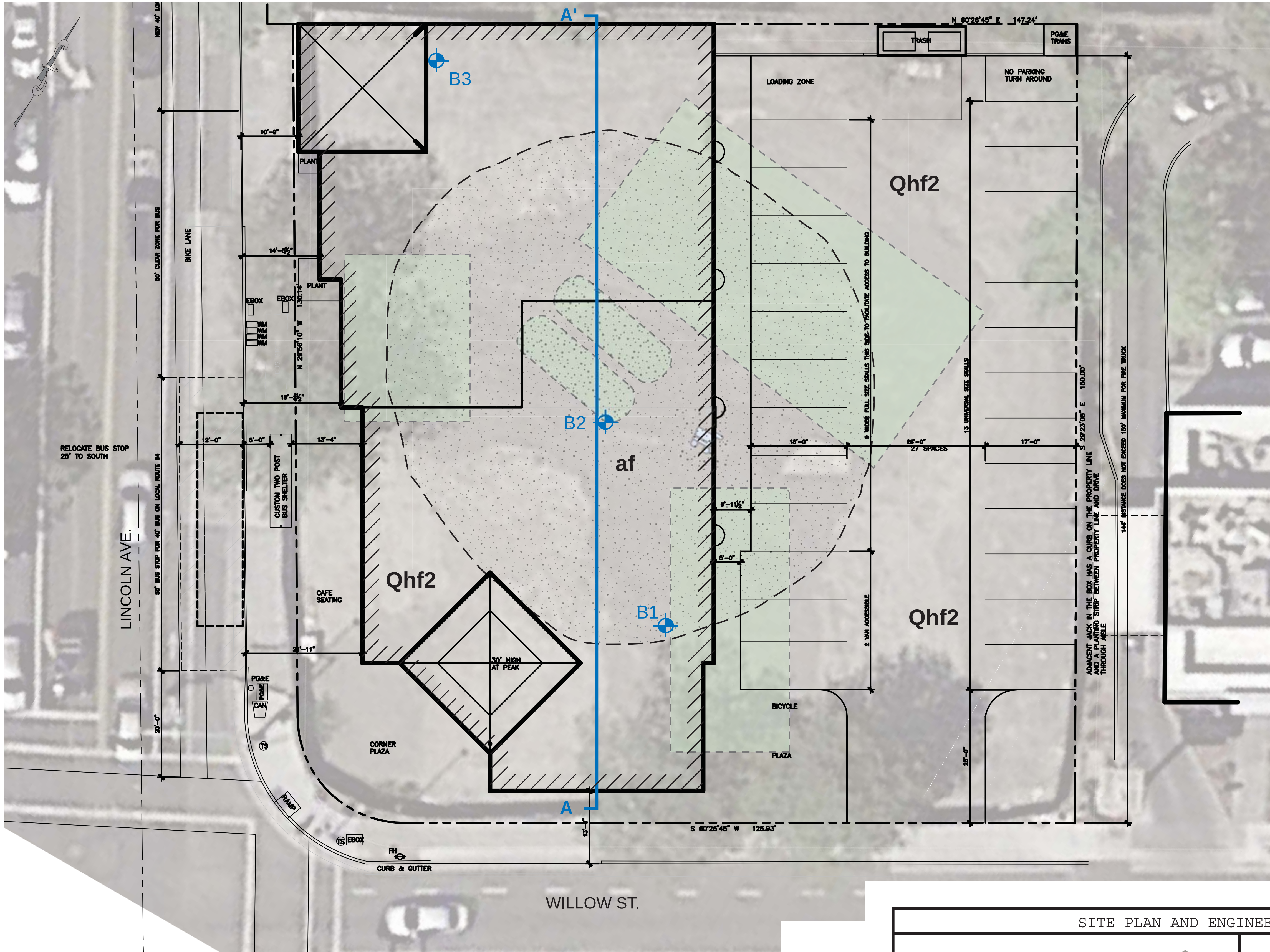
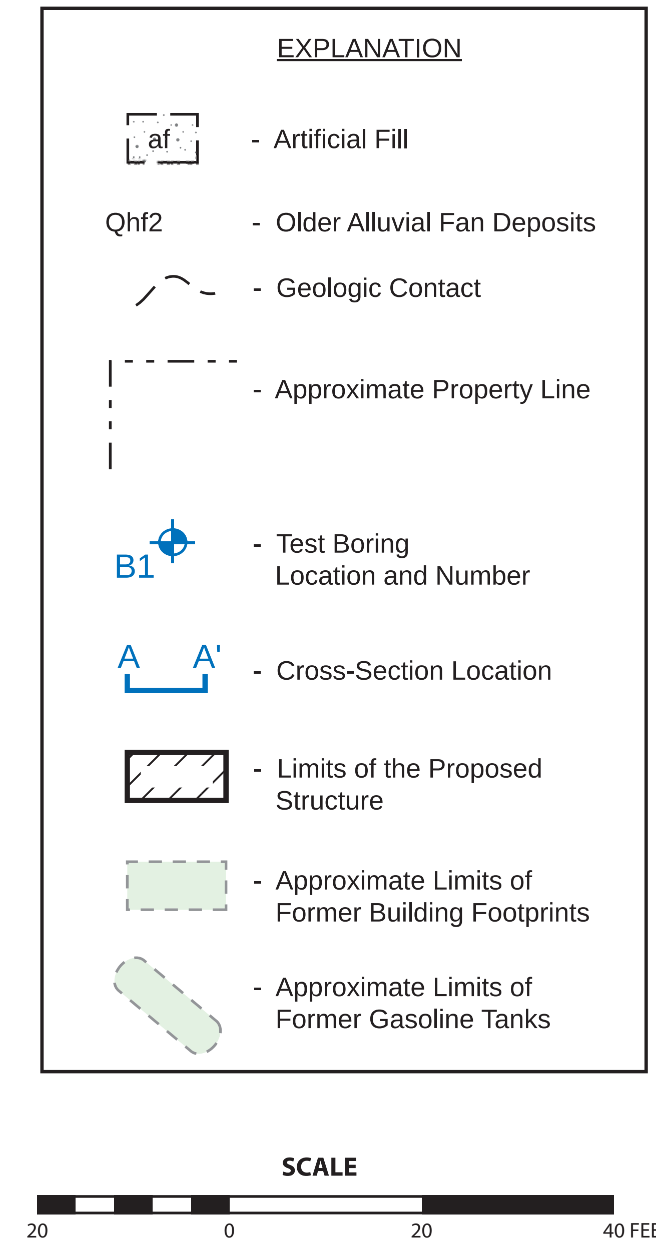
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
RH/CR	1" = 2,000'	15164C-01R1	February 2016	Figure 3

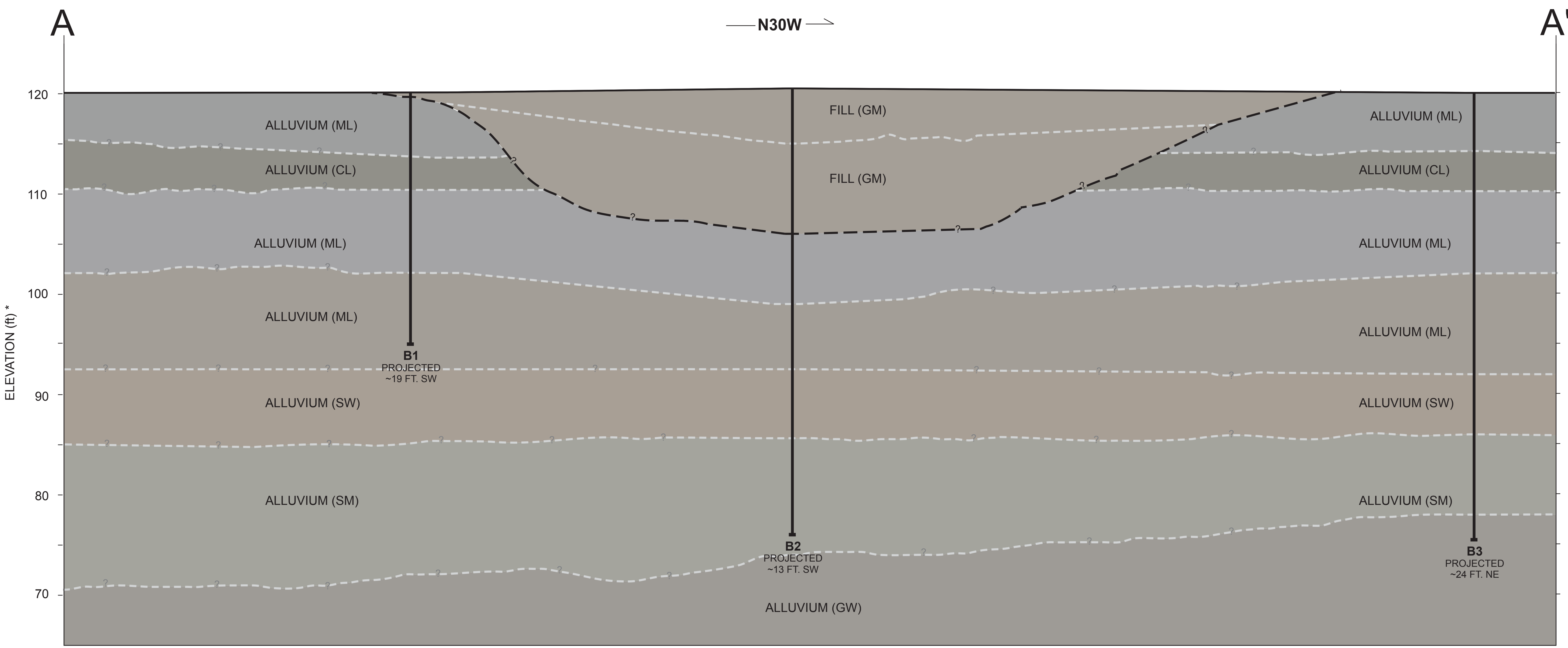




NOTE: This plan is a conceptual illustration of observed geotechnical and geologic features and should not be used for any other purpose.

BASE: Site Plan; Sheet A0; GARCIA TEAGUE ARCHITECTURE & INTERIORS; 10-7-15

SITE PLAN AND ENGINEERING GEOLOGIC MAP				
C2EARTH		DURDEN CONSTRUCTION, INC. 1096 Lincoln Avenue San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
RH/CR	As Shown	15164C-01R1	February 2016	Figure 4



*NOTE: Vertical elevation approximated from Google Earth

NOTE: This cross-section is a conceptual illustration of general geologic relationships and should not be used for any other purpose.

BASE: Site Plan; Sheet A0; GARCIA TEAGUE ARCHITECTURE & INTERIORS; 10-7-15

GEOLOGIC CROSS-SECTION A-A'				
C2EARTH		DURDEN CONSTRUCTION, INC. 1096 Lincoln Avenue San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
RH/CR	1"= 10'	15164C-01R1	February 2016	Figure 5

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels; gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels 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.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or layey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium pasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			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.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE	200	40	10	4	¾"	3"	12"	SIEVE OPENINGS

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

CONSISTENCY AND RELATIVE DENSITY

CONSISTENCY

SILTS AND CLAYS	STRENGTH[2]	BLOWS/FOOT[1]
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

REALATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT[1]
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

[1] Number of blows of 140-pound hammer falling 30 inches to drive a 2-inch O.D (1 3/8-inch I.D) split spoon

[2] Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated in general conformance with the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation

KEY TO LOGS

C2EARTH

DURDEN CONSTRUCTION, INC.
1096 Lincoln Avenue
San Jose, California

DOCUMENT ID.	DATE	
15164C-01R1	February 2016	Figure 6

The standard penetration resistance (SPT) blow counts are obtained in general accordance with ASTM Test Designation D1586. The drive weight assembly consists of a 140-pound hammer dropped through a 30-inch free fall. A blow count is defined as the number of hammer blows per six inches of penetration, or 50 blows for 6 inches or less of penetration. The driving of samplers was discontinued if the observed blow count was 50 for 6 inches or less of penetration.

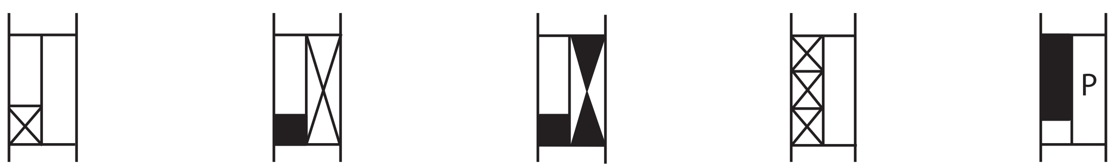
SPT samples are collected in a standard, 2-inch outer diameter, split-barrel sampler without liners (see Figure A below). Samplers holding 2-inch diameter liners (see Figure B below) and 2½-inch diameter liners (see Figure C below) are used to obtain "undisturbed" samples. Occasionally a portable power driven sampler holding 1-inch diameter liners is used for field sampling (see Figure D below). Resistance is measured in seconds per foot and does not correlate with the ASTM SPT. Undisturbed samples may also be collected using a Pitcher Barrel sampler (see Figure E below). Material recovered over the length of the sampler is shaded. A measure of resistance is not collected with this technique.

Blow counts are converted to SPT counts which are shown on the boring logs by the following relation:

$$B = \frac{N\,W\,H}{(140)(30)} \left(\frac{D_{o\ SPT}^{2} - D_{i\ SPT}^{2}}{D_o^{2} - D_i^{2}} \right)$$

B = Equivalent number of blows per foot with a SPT
N = Number of blows per foot actually recorded
W = Weight of hammer (lb)
H = Height of hammer drop (in)
D_o = Outside Diameter (in)
D_i = Inside Diameter (in)

The blow counts used for these conversions were taken over the last two sample intervals if the sampler was driven 12 inches or more. If the sampler is driven less than 12 inches, the blow counts of the last sample were converted to SPT counts of 50 blows over an equivalent SPT run length.



SPT Figure A | 2" Liner Figure B | 2.5" Liner Figure C | 1" Liner Figure D | Pitcher Barrel Figure E

■ = Undisturbed Sample ☒ = Disturbed Sample

Where obtained, the shear strength of the soil samples is shown on the boring logs in far right-hand column.

SUMMARY OF FIELD SAMPLING PROCEDURES

C2EARTH

DURDEN CONSTRUCTION, INC.
1096 Lincoln Avenue
San Jose, California

DOCUMENT ID.	DATE	
15164C-01R1	February 2016	Figure 7

EQUIPMENT	Truck-Mounted Mobile B-40	ELEVATION*	120 feet	LOGGED BY	R. Hohn
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO SUPPORT	Not Encountered	DATE DRILLED	10-23-15

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE
SANDY GRAVEL; black (10YR 2/1); heterogeneous; subrounded to subangular gravels; medium to coarse grained sand; poorly sorted; moist (Fill)	Dense	GM
		ML
CLAYEY SILT; very dark gray (7.5YR 3/4); mottled; homogeneous; trace very fine-grained sand; low to moderate plasticity; oxidation staining; wet (Alluvium)	Firm	
SILTY CLAY; black (7.5YR 2.5/1); homogeneous; moderately plastic; oxidation staining; moist (Alluvium)	Stiff	CL
CLAYEY SILT; strong brown (7.5YR 4/6) to grayish brown (2.5Y 5/2); homogeneous; trace very fine-grained sand; low to moderate plasticity; oxidation staining; moist to wet (Alluvium)	Firm	ML
SANDY SILT; grayish brown (2.5Y 5/2) and strong brown (7.5YR 5/6); mottled; heterogeneous; medium grained, subrounded sand lenses up to 1 inch in size; low plasticity; oxidation staining; moist; trace charcoal (Alluvium)	Firm	ML

DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
5		8	38	74	
10		10			
15		5	30	92	
20		7			

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 1

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	8

EQUIPMENT Truck-Mounted Mobile B-40	ELEVATION* 120 feet	LOGGED BY R. Hohn
DEPTH TO GROUNDWATER Not Encountered	DEPTH TO SUPPORT Not Encountered	DATE DRILLED 10-23-15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE						
SANDY SILT (continued from above)	Firm	ML	21 22 23 24 25		10	22	98	
Bottom of Boring = 25 feet			26 27 28 29 30 31 32 33 34 35 36 37 38 39 40					

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 1 (CONTINUED)

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	9

EQUIPMENT	Truck-Mounted Mobile B-40	ELEVATION*	120½ feet	LOGGED BY	R. Hohn
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO SUPPORT	28 feet	DATE DRILLED	10-23-15

DESCRIPTION AND CLASSIFICATION								
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY GRAVEL; black (10YR 2/1); heterogeneous; subrounded to subangular gravels; medium to coarse grained sand; poorly sorted; dry (Fill)	Dense	GM	0–5½	2–3½	48	6	122	
SILTY GRAVEL; dark brown (7.5YR 3/2); heterogeneous; subrounded gravels; well sorted; moist (Fill)	Loose	GM	5½–14½	6½–8	7			
				11½–13	5			
CLAYEY SILT; grayish brown (7.5YR 5/2); heterogeneous; trace fine-grained sand; moderate plasticity; moist; carbonized and oxidized roots and rootlets; moist (Alluvium)	Stiff	ML	14½–20	16½–18	10	25	100	

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 2		
DURDEN CONSTRUCTION, INC. San Jose, California		
DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	10

EQUIPMENT	Truck-Mounted Mobile B-40	ELEVATION*	120½ feet	LOGGED BY	R. Hohn
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO SUPPORT	28 feet	DATE DRILLED	10-23-15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE						
CLAYEY SILT (continued from above)	Stiff	ML	21					
SANDY SILT; grayish brown (2.5YR 5/2) to yellowish brown (10YR 5/8); mottled; heterogeneous; very fine-grained subangular to subrounded sand; scattered gravels at base of layer; low plasticity; moist (Alluvium)	Stiff	ML	22 – 28		10 (22–23); 8 (27–28)			
SAND WITH GRAVEL; dark olive brown (2.5Y 3/3); heterogeneous; subrounded, fine- to medium-grained sand; subrounded gravel; trace subrounded cobbles; oxidation staining; moist (Alluvium)	Medium Dense	SW	28 – 35		24 (32–33)			
SILTY SAND; dark-greenish gray (GLEY 1) to dark yellowish brown (10YR 3/4); mottled; heterogeneous; subangular to subrounded very fine- to fine-grained; scattered subrounded gravels; oxidation and reduction staining; poorly sorted; moist; carbonized organics (Alluvium)	Medium Dense Very Dense	SM	35 – 40		62 (37–38)			

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 2 (CONTINUED)

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	11

EQUIPMENT Truck-Mounted Mobile B-40	ELEVATION* 120½ feet	LOGGED BY R. Hohn
DEPTH TO GROUNDWATER Not Encountered	DEPTH TO SUPPORT 28 feet	DATE DRILLED 10-23-15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE						
SILTY SAND (continued from above)	Very Dense	SM	- 41 -					
			- 42 -					
			- 43 -		13			
			- 44 -		17			
Bottom of Boring = 44½ feet			- 45 -					
			- 46 -					
			- 47 -					
			- 48 -					
			- 49 -					
			- 50 -					
			- 51 -					
			- 52 -					
			- 53 -					
			- 54 -					
			- 55 -					
			- 56 -					
			- 57 -					
			- 58 -					
			- 59 -					
* Vertical elevation approximated from Google Earth			- 60 -					

C2EARTH

LOG OF BORING 2 (CONTINUED)

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	12

EQUIPMENT	Truck-Mounted Mobile B-40	ELEVATION*	120 feet	LOGGED BY	R. Hohn
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO SUPPORT	28 feet	DATE DRILLED	10-23-15

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
CLAYEY SILT; dark gray (10YR 4/1) to dark yellowish brown (10YR 4/6); heterogeneous; mottled; trace subangular gravels; low to moderate plasticity; oxidation staining; moist; trace roots (Alluvium)	Firm to Stiff	ML	0–5	Sample at 0–1.5 ft	12			
SILTY CLAY; black (7.5YR 2.5/1); homogeneous; moderately plastic; oxidation staining; moist to wet (Alluvium)	Stiff	CL	5–10	Sample at 5–6.5 ft	5	30	80	
CLAYEY SILT; dark gray (2.5Y 4/1); homogeneous; ow to moderate plasticity; oxidation staining; trace rootholes; moist; trace roots (Alluvium)	Firm	ML	10–17.5	Samples at 10–11.5 ft and 15–16.5 ft	12, 18			
SANDY SILT; strong brown (7.5YR 5/6) to grayish brown (2.5Y 5/2); mottled; heterogeneous; fine-grained sand; trace subrounded gravels; low plasticity; well sorted; oxidation staining; trace roots; moist (Alluvium)	Stiff to Very Stiff	ML	17.5–20					

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 3

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	13

EQUIPMENT Truck-Mounted Mobile B-40	ELEVATION* 120 feet	LOGGED BY R. Hohn
DEPTH TO GROUNDWATER Not Encountered	DEPTH TO SUPPORT 28 feet	DATE DRILLED 10-23-15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE						
SANDY SILT (continued from above)	Stiff to Very Stiff	ML	21		17	16	107	
			22					
			23					
			24					
			25					
			26		13			
			27					
SAND WITH GRAVEL; dark olive brown (2.5Y 3/3); heterogeneous; subrounded, fine- to medium-grained sand; subrounded gravel; trace subrounded cobbles; oxidation staining; dry (Alluvium)	Medium Dense	SW	28					
			29		28			
			30					
			31					
			32		30		108	
			33					
SILTY SAND; light yellowish brown (2.5Y 6/3) to yellowish brown (10YR 5/6); mottled; very fine- to fine-grained; trace subrounded gravels; oxidation staining; carbonized oragnics; moist (Alluvium)	Medium Dense	SM	34					
			35					
			36		20	14	114	
			37					
			38					
			39					
			40					

* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 3 (CONTINUED)

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	14

EQUIPMENT	Truck-Mounted Mobile B-40	ELEVATION*	120 feet	LOGGED BY	R. Hohn
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO SUPPORT	28 feet	DATE DRILLED	10-23-15

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE						
SILTY SAND (continued from above); grayish brown (2.5Y 5/2) to yellowish brown (10YR 5/6); slightly mottled; oxidation staining; moist; carbonized organics (Alluvium)	Medium Dense	SM	- 41 -		12	22	101	
SANDY GRAVELS; olive brown (2.5Y4/3); subangular to subrounded; poorly sorted; subrounded to subangular fine- to coarse-grained sand; moist (Alluvium)	Very Dense	GW	- 42 - - 43 - - 44 -		59			
Bottom of Boring = 44½ feet			- 45 - - 46 - - 47 - - 48 - - 49 - - 50 - - 51 - - 52 - - 53 - - 54 - - 55 - - 56 - - 57 - - 58 - - 59 - - 60 -					

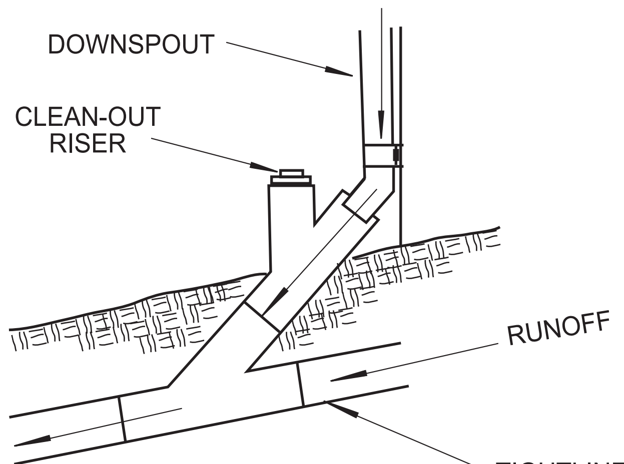
* Vertical elevation approximated from Google Earth

C2EARTH

LOG OF BORING 3 (CONTINUED)

DURDEN CONSTRUCTION, INC.
San Jose, California

DOCUMENT ID.	DATE	FIGURE NO.
15164C-01R1	February 2016	15



CONCEPTUAL DOWNSPOUT CLEAN-OUT DIAGRAM

C2EARTH

DURDEN CONSTRUCTION, INC.
1096 Lincoln Avenue
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
TB/CR	Not Applicable	15164C-01R1	February 2016	Figure 16

TABLE I

MODIFIED MERCALLI SCALE OF EARTHQUAKE INTENSITIES

I. Not felt by people, except under especially favorable circumstances.

II. Felt only by persons at rest on the upper floors of buildings. Some suspended objects may swing.

III. Felt by some people who are indoors, but it may not be recognized as an earthquake. The vibration is similar to that caused by the passing of light trucks. Hanging objects swing.

IV. Felt by many people who are indoors, by a few outdoors. At night some people are awakenad. Dishes, windows and doors are disturbad: walls make creaking sounds; stationary cars rock noticeably. The sensation is like a heavy object striking a building; the vibration is similar to that caused by the passing of heavy trucks.

V. Felt indoors by practically everyone, outdoors by most people. The direction and duration of the shock can be estimated by people outdoors. At night, sleepers are awakened and some run out of buildings. Liquids are disturbed and sometimes spilled. Small, unstable objects and some furnishings are shifted or upset. Doors close or open.

VI. Felt by everyone, and many people are frightened and run outdoors. Walking is difficult. Small church and school bells ring. Windows, dishes, and glassware are broken; liquids spill; books and other standing objects fall; pictures are knocked from walls; furniture is moved or overturned. Poorly built buildings may be damaged, and weak plaster will crack.

VII. Causes general alarm. Standing upright is very difficult. Persons driving cars also notice the shaking. Damage is negligible in buildings of very good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or designed structures. Some chimneys are broken; interiors and furnishings experience considerable damage; architectural ornaments fall. Small slides occur along sand or gravel banks of water channels; concrete irrigation ditches are damaged. Waves form in the water and it becomes muddied.

VIII. General fright and near panic. The steering of cars is difficult. Damage is slight in specially designed earthquake-resistant structures, considerable in well-built ordinary buildings. Poorly built or designed buildings experience partial collapses. Numerous chimneys fall; the walls of frame buildings are damaged; interiors experience heavy damage. Frame houses that are not properly bolted down may move on their foundations. Decayed pilings are broken off. Tress are damaged. Cracks appear in wet ground and on steep slopes. Changes in the flow or temperature of springs and wells are noted.

IX. Panic is general. Interior damage is considerable in specially designed earthquake-resistant structures. Well-built ordinary buildings suffer severe damage, with partial collapses; frame structures thrown out of plumb or shifted off of their foundations. Unreinforced masonry buildings collapse. The ground cracks conspicuously and some underground pipes are broken. Reservoirs are damaged seriously.

X. Most masonry and many frame structures are destroyed. Specially designed earthquake-resistant structures may suffer serious damage. Some well-built bridges are destroyed, and dams, dikes and embankments are seriously damaged. Large landslides are triggered by the shock. Water is thrown onto the banks of canals, rivers and lakes. Sand and mud are shifted horizontally on beaches and flat land. Rails are bent slightly. Many buried pipes and conduits are broken.

XI. Few, if any, masonry structures remain standing. Other structures are severely damaged. Broad fissures, slumps and slides develop in soft or wet soils. Underground pipe lines and conduits are put completely out of service. Rails are severely bent.

XII. Damage is total, with practically all works of construction severely damaged or destroyed. Waves are observed on ground surfaces, and all soft or wet soils are greatly disturbed. Heavy objects are thrown into the air, and large rock masses are displaced.

YANEV,P.,1974, Peace of Mind in Earthquake Country, Chronicle Books, San Francisco, California.

APPLICATION TO USE

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GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED COMMERCIAL RE-DEVELOPMENT

DURDEN CONSTRUCTION, INC.
1096 LINCOLN AVENUE
SAN JOSE, CALIFORNIA

Document Id. 15164C-01R1
Dated 17 February 2016

TO: Upp Geotechnology
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