341 BELLA VISTA AVENUE
LOS GATOS, CALIFORNIA

Architecture and Site Application S-12-103
Subdivision Application M-12-008
Mitigated Negative Declaration ND-16-001

INITIAL STUDY AND ENVIRONMENTAL CHECKLIST

ATTACHMENT 2
GEOLOGIC AND GEOTECHNICAL STUDY

UPDATED GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT

ROSS PROPERTY
339 AND 341 BELLA VISTA AVENUE
LOS GATOS, CALIFORNIA

Prepared For:

Mr. Dan Ross
233 West Main Street
Los Gatos, California

25 June 2015
Document Id. 15068C-01R1

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

25 June 2015
Document Id. 15068C-01R1
Serial No. 17200

Mr. Dan Ross
233 West Main Street
Los Gatos, CA 95030

SUBJECT: UPDATED GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
ROSS PROPERTY
339 AND 341 BELLA VISTA AVENUE
LOS GATOS, CALIFORNIA

Dear Mr. Ross:

As you requested, we have performed an updated geologic and geotechnical study for the proposed residential development of your property at 339 and 341 Bella Vista Avenue in Los Gatos, California. We understand that you are merging the two lots and plan on constructing a single residence on the combined site. The property is in a geologically sensitive area; the nearest trace of the potentially active Shannon fault is mapped southwest of the property. In addition, the site is partially mapped within a State Seismic Hazard Zone for earthquake-induced landsliding. In summary, we conclude that, from a geologic and geotechnical engineering perspective, the site is suitable for the proposed residential development. We judge that there is a low potential for surface fault rupture to manifest on the site from an earthquake or coseismic event, or for slope instability to affect the proposed improvements.

The accompanying report presents the results of our study, 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.

Jennifer Buckley
Project Geologist

Christopher R. Hundemer, Principal
Certified Engineering Geologist 2314
Certified Hydrogeologist 882

THIS DOCUMENT HAS BEEN DIGITALLY SIGNED

REGISTERED PROFESSIONAL ENGINEER
CRAIG N. REID
No. 3060
GEOTECHNICAL
STATE OF CALIFORNIA

Craig N. Reid, Principal
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Distribution: Addressee (3 hard copies to be picked up and via e-mail to dan.ross@wellsfargo.com)

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1. INTRODUCTION

This report presents the results of our updated geologic and geotechnical study for the proposed residential development of your property at 339 and 341 Bella Vista Avenue in Los Gatos, California (see Figure 1, Site Location Map). The purposes of our study were: to become the geotechnical and geologic consultant of record for the project; update the geologic and geotechnical findings from prior studies performed for the development of the subject property; and develop updated geotechnical recommendations for the earthwork and foundation engineering aspects of the proposed development.

We understand that you are merging the two parcels into one combined lot and you plan to construct one single-family residence on the southern portion of the merged lot. The residence will be a three-story structure, partially set into the northwest-facing hillside, with the lower two levels daylighting to the northwest. The uppermost level will consist of a two-car garage and roof-top deck. We understand that a bridge will be constructed to serve as a driveway and provide access to the garage from Bella Vista Avenue.

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 residential 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 10 June 2015 (Document Id. 15068C-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, aerial photographs, our prior reports of the area and previous consultants’ reports of the subject property and site vicinity to evaluate the prevailing geologic and geotechnical conditions;
- consulting with the Town's reviewing geologist, Mr. Bob Wright of AMEC Foster Wheeler, about his concerns with the site and vicinity;
- performing engineering geologic reconnaissance and mapping of the site in the area of the proposed improvements;
- preparing an updated site plan and an updated geologic cross-section;
- analyzing geologic and geotechnical engineering properties from previously collected data; and
- preparing this report.

We have prepared this report as a product of our service for the exclusive use of Mr. Dan Ross for the proposed residential 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 and geologic hazard maps, aerial photographs, our prior reports, and other consultant's reports for studies performed for the subject site and nearby sites to evaluate the prevailing geologic conditions of the site and vicinity. Regional and local geologic and geologic hazard maps are presented on Figures 2 through 8.

3.1. Geology

The subject property lies within the Coast Ranges geomorphic province, which is characterized by northwest-southeast trending valleys and ridges. The subject property is sited on a moderately steep, northwest-facing slope of a deeply-incised fluvial terrace riser. (see Figure 1).

According to the Geologic Map of the Los Gatos Quadrangle (McLaughlin et al., 2001), the subject site is underlain by Pleistocene age (approximately 10,000 to 2.6 million years old) alluvial fan deposits (see Figure 2, Regional Geologic Map). The alluvial fan deposits generally consist of silts, clays, sands, and gravels deposited by stream flow from mountains onto adjacent lowlands. Within the site vicinity, the alluvium is several tens of feet thick and overlies bedrock of the Franciscan assemblage, Monterey formation, and Santa Clara formation. The Pleistocene age alluvium within the incised valley northwest of the site is locally overlain by younger, Holocene age (approximately 10,000 year old to present) alluvium.

Locally, the alluvial fan deposits are overlain by a relatively thin veneer of slope debris (colluvium) on the subject property and across most of the hillside areas in the site vicinity. Where the colluvium is located on moderate to steep slopes, it is subject to downhill creep, a process by which the soil moves downslope at an imperceptibly slow rate as a result of gravity.

3.2. Landsliding

Our site reconnaissance, review of prior studies, and review of stereo-paired aerial photographs revealed no evidence of recent landslides in the vicinity of the proposed improvements. The topography across the subject property descends moderately toward the northwest. According to the Regional Geologic Map, no landslides are mapped on the site or in the site vicinity (see Figure 2).

While there is no evidence of landsliding in the site vicinity, the northwestern edge of the site is mapped within a State of California Seismic Hazard Zone for earthquake-induced landsliding (see Figure 3, Regional Seismic Hazard Zones Map). The same area is also mapped near a zone designated as having a “landslide potential” on the Town's Landslide Hazard Areas map (see Figure 4, Local Landslide Hazard Areas Map). These zones were established to minimize the loss of life and property by identifying and mitigating seismic hazards related to landslides. Consequently, we have conducted a qualitative slope stability evaluation of the northwest-facing slope to evaluate the risk of landsliding.

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 San Andreas fault has a regional trend of approximately N34W; however, the segment of the San Andreas fault located within the central Santa Cruz Mountains southwest of the site strikes approximately N44W, forming a restraining bend. This restraining bend has created a compressional zone along the eastern side of the Santa Cruz Mountains, resulting in the formation of the Frontal thrust system, comprised of reverse and right-reverse faults, including the Berrocal, Monte Vista, and Shannon faults within the eastern foothills and the alluvial plain adjacent to the foothills (Angell et al., 1997). This thrust fault system bounds the southwest margin of the Santa Clara Valley.

According to geologic mapping by McLaughlin et al., (2001), the site is situated near a concealed trace of the potentially active Shannon fault (see Figure 2). Because of the proximity to this and other faults within the region, the site is mapped within a zone of high fault rupture potential by the Town of Los Gatos (see Figure 5, Local Fault Rupture Potential Map). These zones were created based upon the Faults, Folds, and Zones of Lineaments Map by William Lettis and Associates, Inc., 1994 (see Figure 6) and the Fault, Lineament, and Coseismic Deformation Map by Nolan Associates, 1999 (see Figure 7).

A concentration of coseismic ground deformation associated with the Loma Prieta Earthquake is mapped about 500 feet southeast of the site (see Figure 7). According to mapping by Schmidt et al., 1995, this deformation generally consisted of fresh pavement breaks or buckles suggestive of contractional deformation and pavement breaks with an unspecified sense of deformation (see Figure 8, Map of Observed Distress from the 1989 Loma Prieta Earthquake).

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

Fault	Approx. Distance From Fault	Direction From Site
Shannon (nearest trace)	500 feet	Southwest
Shannon (second trace)	1,600 feet	Northeast
Berrocal	2,000 feet	Southwest
San Andreas	3¼ miles	Southwest
Hayward	12½ miles	Northeast
Calaveras	15 miles	Northeast
San Gregorio	19½ 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 Zone for areas prone to earthquake ground rupture.

Because of the site's proximity to the San Andreas fault 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 VIII (Borcherdt, et. al., 1975). An earthquake having a MM intensity of VIII generally causes considerable damage to well-built, ordinary structures and partial collapse to poorly built 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 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.

3.4. Liquefaction and Lateral Spreading

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. The subject site is not mapped within a State Seismic Hazard Zone for earthquake-induced liquefaction (see Figure 3), and in our opinion the potential for liquefaction to affect the proposed development is negligible.

4. PRIOR REPORTS

As part of our study, we reviewed prior geologic and geotechnical reports prepared by us and other consultants for the subject site and for other properties in the site vicinity that overlie nearby, concealed traces of the Shannon fault. The locations of the properties for which the studies were performed are shown on Figure 2. The following subsections present summaries of the reports that we reviewed.

4.1. ADCO Engineering, 1998

ADCO Engineering (ADCO) prepared a Soil and Foundation Investigation report, dated 12 June 1998, for the subject property. During their study, ADCO drilled one boring in the upper, central portion of the property to a depth of approximately 20 feet below ground surface (bgs) and performed Plasticity Index (PI) testing on a sample obtained between 1 and 2 feet bgs. The test results revealed a PI of 14 percent, which is considered to be slightly expansive. The location of the test boring is shown on Figure 9, Updated Site Plan and Engineering Geologic Map. The boring log and plasticity index test results are presented in the Appendix. ADCO recommended that the residence be supported on a pier and grade beam foundation system.

4.2. Cotton, Shires, and Associates, 1999

Cotton, Shires, and Associates (CSA) performed a geological evaluation at 16641 Kennedy Road in 1999, approximately 2,700 feet east of the subject property. According to CSA's report, *"There is no geomorphic evidence to indicate that the Shannon fault passes through the Bowman parcel...we conclude that the risk of surface faulting at this site to be low."*

4.3. AIBM Soil Testing Engineers, 2002

AIBM Soil Testing Engineers (AIBM) prepared a Soil and Foundation Investigation report, dated 25 November 2002, for the subject property. As part of their study, AIBM performed a quantitative slope stability analysis. The results of their analysis revealed a minimum factor of safety of 1.26 under pseudo-static (seismic) conditions. They observed no slope failures on the subject property and provided updated geotechnical recommendations for site development.

4.4. Pacific Geotechnical Engineering, 2005

Pacific Geotechnical Engineering (PGE) prepared an Engineering Geologic Investigation report (2005) for the property at 16661 Kennedy Road, about 2,600 feet east of the subject property. During their study, they excavated an approximately 135-foot long by 10-foot deep fault exploration trench. They encountered essentially flat-lying alluvial deposits. According to PGE, a *"thin, fairly continuous bed of gravelly sand at a depth of about 7 feet traversed the entire trench. Its lateral continuity and uninterrupted horizontality provide clear evidence for the absence of vertical faulting...since our observations indicate the sedimentary layers of the recent alluvium exposed in the trench are almost perfectly flat, it can be inferred that there has not been tectonic deformation at the site within the recent geologic past."*

4.5. Ali M. Oskoorouchi, 2005

Ali M. Oskoorouchi, Ph.D., P.E., G.E. (AMO) submitted an Updated Geotechnical Investigation report (2005) for the subject property. As part of his study, AMO reviewed the two older reports for the subject property prepared by AIBM (2002) and ADCO (1998). In order to substantiate ADCO's subsurface exploration, AMO hand-augured a boring to approximately 6 feet bgs and found that the material encountered in their boring correlated with the material found in ADCO's boring. According to AMO, both borings encountered brown sandy gravel with cobbles, silt, and clay. No groundwater was encountered in either boring. In addition, AMO performed a gradation test of the sample obtained from their boring at a depth of about 2 to 2½ feet bgs (see Appendix).

AMO concluded the risk for earthquake-induced ground rupture to occur across the property to be low. They recommended that the residence be structurally supported by a pier and grade beam foundation system.

4.6. Steven F. Connelly, 2007

Steven F. Connelly, C.E.G (SFC) submitted a Limited Fault Investigation report (2007) for 16621 Kennedy Road, located about 2,800 feet east of the subject property. Connelly suggested, based upon geomorphic evidence, that a trace of the Berrocal (Shannon) fault may be located south of the site and *"it is unlikely that primary ground rupture due to faulting will impact the existing residence or the proposed addition and remodel."*

4.7. Gilpin Geosciences, Inc., 2007

Gilpin Geosceinces, Inc. (GGI) performed an Updated Geological and Geotechnical Investigation for the subject property. The results of their study was presented in a draft report dated 23 April 2007, and a final report dated 31 July 2007. As part of their study, GGI logged four borings drilled to depths of between about 14½ and 22½ feet bgs. The boring locations are presented on Figure 9 and the boring logs are presented in the appendix.

According to GGI, they *"did not identify any shallow landsliding or evidence for deep seated slope instabilities on the site."* They also concluded that *"the risk of fault offset at the site from a known active fault is low, however the risk of coseismic distributed ground surface deformation is moderate to high. To accommodate the effects of distributed ground rupture the new structures should be constructed on concrete pier and grade beam foundations."*

4.8. Ali M. Oskoorouchi, 2007

Ali M. Oskoorouchi (AMO) submitted an Updated Geological and Geotechnical Investigations report, dated 1 August 2007, for the subject property. The study addressed the potential hazard for earthquake-induced landsliding in accordance with the Guidelines for Evaluating and Mitigating Seismic Hazards in California (Special Publication 117). AMO found no evidence of landsliding on the site. In addition, AMO performed several additional gradation tests on samples obtained from the GGI borings. The gradation test results are presented in the Appendix. AMO provided updated geotechnical recommendations for the project.

4.9. Treadwell & Rollo, 2008

Treadwell and Rollo (T&R) prepared a Revised Fault Hazard Investigation report (2008) for a site located at 50 Los Gatos-Saratoga Road. The central portion of this site is located about 700 feet southwest of the subject property and overlies the nearest mapped concealed trace of the Shannon fault. During their study, T&R drilled and logged six borings to depths of between 69½ and 84 feet deep. The borings encountered alluvium to depths of between 62 and 78 feet bgs. The alluvium was underlain by Monterey formation bedrock. T&R observed a six-foot vertical offset of the bedrock between two of the borings and interpreted the offset as a thrust fault. In order to evaluate if the offset observed in the bedrock horizon extended into recent deposits, T&R excavated a 100-foot long by 12-foot deep fault exploration trench. According to T&R, *"the contacts between alluvial layers were distinct, and were not offset by faulting."* In addition, T&R obtained two samples of carbon (charcoal/wood fragments) from two layers of alluvium in the trench for age dating. The results of the carbon dating revealed one sample to be approximately 11,000 years old and the other to be older than 11,000 years.

T&R concluded that "the upper 12 feet of alluvium at the site has not experienced active faulting. Considering the thickness of the alluvium, and the results of the age dating from the carbon samples obtained from the trench, we conclude there is no evidence that this splay of the Shannon fault has been active during Holocene time (the past 11,000 years). Furthermore, there is no evidence that the fault has broken the ground surface. Therefore, we conclude that the potential from fault offset through the property is negligible, and a setback from this fault should not be required." T&R determined that if the fault splay were to move as sympathetic (coseismic) movement during a large earthquake on the San Andreas fault, the movement would dissipate through the alluvial gravels, and would result in maximum differential settlement of the alluvium on either side of the fault of ¾ -inch, per event.

4.10. C2Earth, Inc. 2013

We reviewed our prior Geologic and Geotechnical Study report, dated 26 April 2013, that was prepared for the residential redevelopment of a site approximately 1,500 feet northeast of the subject property, APN 529-20-055, on Pine Avenue in Los Gatos, California. During our study, we drilled and logged three test borings to depths of approximately 21 feet or less bgs. Our subsurface exploration revealed uniform layers of alluvial fan deposits, with no evidence suggestive of fault offset through the subject site. The alluvial deposits are potentially 100 feet thick or more beneath the site. We concluded that even if the fault were to be located at depth beneath the site, the thick alluvium overlying the bedrock would disperse and realign during a seismic event, resulting in a dispersal of ground rupture energy and minimal fault offsets or rupture at the site. We provided recommendations for supporting the residence on shallow foundations bearing in the alluvium.

4.11. C2Earth, 2014

We reviewed our prior Geologic and Geotechnical Study report, dated 5 February 2014, that was prepared for the residential redevelopment of a site approximately 2,100 feet east-northeast of the subject property at 16600 Englewood Avenue in Los Gatos, California. During that study, we logged three test borings drilled to depths of approximately 19½ feet or less bgs. Our subsurface exploration revealed uniform layers of alluvial fan deposits, with no evidence substantiating the mapped location of a fault trace beneath the site. Furthermore, we concluded that fault movements that resulted in displacements within the bedrock beneath the alluvial fan deposits would propagate upward in a flower pattern, causing shifting and realignments of the uncemented alluvial deposits. This would disperse the amount of deformation at the ground surface. We concluded that there was minimal risk for fault related ground deformation and provided recommendations for supporting the residence on the alluvial deposits.

4.12. Achievement Engineering Corp., 2014

Achievement Engineering Corp. (AEC) provided a Geotechnical Upgrade Report (2014) for the subject property in which they accepted the prior findings and recommendations by GGI and AMO and provided updated 2013 California Building Code seismic design criteria for the project.

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5. SITE CHARACTERIZATION

5.1. Site Description

On 17 June 2015, our principal geologist performed a site reconnaissance and geologic mapping on the subject property. We updated a prior site geology map and geologic cross-section by GGI based upon an architectural site plan (Provis, Inc., dated 13 November 2014) depicting the current residential development concept that you provided, supplemented by tape and compass mapping techniques (see Figure 9, Updated Site Plan and Engineering Geologic Map and Figure 10, Updated Geologic Cross-Section A-A'). The updated site plan and profile are only as accurate as implied by the mapping techniques used. The following is a summary of the surficial site characteristics.

The roughly rectangular-shaped site is situated along a moderately steep northwest-facing slope. The site is elongated, with its long axis oriented in the northeast-southwest direction. The undeveloped subject property is bound to the northwest by a townhouse subdivision, by Bella Vista Avenue on the southeast, and by undeveloped properties on the remaining sides.

Bella Vista Avenue was created using cut and fill grading techniques. Fill was placed along the downslope edge of the southeastern property boundary to create a level area for the road. The fill slope varies between about 5 to 9 feet tall and extends laterally about 10 to 19 feet from the break in slope along the road's edge. The topography across the subject property generally descends toward the northwest, with overall slope gradients of about 2:1 (horizontal to vertical) and localized areas with gradients approaching 1¾:1.

Drainage across the site is generally characterized as uncontrolled sheet flow to the northwest. The subject property is vegetated with mature oak and almond trees, and associated grasses, brush, and poison oak.

5.2. Subsurface

We reviewed logs of borings that were presented in the prior reports by ADCO, AMO, and GGI. The approximate locations of the prior borings are shown on Figure 9. We determined the approximate boring locations by overlaying prior site plans with our updated site plan; these locations are only as accurate as implied by the mapping technique used. The logs of the borings are presented in Appendix I.

A total of six borings were drilled on the site to depths of 22½ feet or less. In general, the excavations encountered a similar sequence of subsurface materials, including fill and/or colluvium (a soil material that is deposited on or at the base of a slope from sheet flow runoff) underlain by alluvial fan deposits.

Borings 1 and 2 by GGI were drilled in the immediate vicinity of the proposed improvements. Boring 1 was located near the southern corner of the property, near the proposed bridge, and Boring 2 was located near the northern portion of the proposed residence. Boring 1 encountered about 4 feet of fill consisting of medium dense, gravelly clayey sand, underlain by colluvium. Boring 2 encountered colluvium at the ground surface. The colluvium is up to approximately 3½ feet thick and consists of medium dense, gravelly clayey sand. The colluvium is underlain by alluvial fan deposits that persisted to the bottoms of the borings. The alluvial fan deposits are comprised of medium dense to very dense, sandy silty gravel. Our interpretations of subsurface conditions are depicted on Figure 10.

5.3. Groundwater

According to the prior reports and boring logs, groundwater was not encountered in any of the borings on the subject property. Additionally, according to T&R, the borings on the site that is southwest of the subject property, at a lower elevation, encountered water at about 21 to 24 feet bgs. Fluctuations in the level of subsurface water could occur due to variations in rainfall, temperature, and other factors not evident at the time the observations were made.

5.4. Laboratory Testing

As discussed above in the "Prior Reports" section, moisture content, dry density, gradation, and plasticity index testing was performed by other consultants on samples obtained from the prior borings. The results of the moisture content and dry density tests are presented on the boring logs and the results of the other tests are included in Appendix I.

6. LANDSLIDE SCREENING EVALUATION

As noted above, the northwestern edge of the subject site is mapped within the State Seismic Hazard zone for earthquake-induced landsliding (see Figure 3). The purpose of this qualitative screening evaluation is to evaluate the severity of the potential for earthquake-induced landsliding to occur on the subject site and to determine if further analysis is warranted (CDMG, 1996). In accordance with Special Publication 117A by the California Geological Survey (2008), our screening analysis includes an evaluation of the following questions:

- ***Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?*** No. Our study and the prior studies for the subject site revealed no mapped landslides within the site or immediate vicinity and we observed no evidence of landslides on the subject property during our site reconnaissance.
- ***Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?*** No. According to the geologic map, Pleistocene age alluvial fan deposits underlie the subject site and immediate site vicinity. These materials are not known to be susceptible to landsliding in the general site area.
- ***Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?*** No. Slope areas on the site are generally uniform. We did not observe any evidence of springs or seeps in areas that could affect the proposed building site.

- ***Are susceptible landforms and vulnerable locations present? These include steep slopes, colluvium-filled swales, cliffs or banks being undercut by stream or wave action, areas that have recently slid.*** No. The site slopes are generally uniform and moderately steep, with general slope gradients of about 2:1 that are comprised of a thin veneer of fill and colluvium over alluvial fan deposits. In our opinion, these slopes and underlying materials do not represent susceptible landforms.
- ***Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?*** No. In our opinion, the current development concept will not increase the potential for landsliding on the subject site.

7. FINDINGS

Based upon the results of our updated study, it is our opinion that, from engineering geologic and geotechnical engineering perspectives, the subject property may be developed as planned, provided 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 fill and/or colluvium blanketing the supportive alluvial fan deposits on the moderately steep site slopes, and the site's seismic setting.

7.1. Proposed Building Site

Prior subsurface exploration revealed that the proposed building site is underlain at depth by alluvial fan deposits. The supportive alluvial fan deposits within the building area are blanketed by up to approximately 7 feet of non-supportive undocumented fill and/or colluvium. Where located on moderate to steep slopes, these non-supportive materials can experience imperceptibly slow downhill creep under the force of gravity. The underlying, supportive alluvial fan deposits are comprised of medium dense to very dense, sandy silty gravel. In our opinion, the alluvial fan deposits should provide adequate support for the foundations of a proposed residence and associated improvements.

Standard penetration test results suggest that cobbles and boulders within the alluvial fan deposits at the site can be very hard locally. We recommend that the contractor plan for this condition in choosing the appropriate means and methods of excavating the foundations for the proposed improvements.

7.2. Slope Stability

Based upon our site reconnaissance, review of stereo-paired aerial photographs, and our review of prior reports, our study revealed no evidence of recent landsliding on the subject property. Because of the moderately steep slopes and fill and colluvium that blanket the alluvial fan deposits on the subject property, the occurrence of a new, shallow landslide within or adjacent to the subject property cannot be excluded. A new, shallow landslide (approximately less than 10 feet deep) in this area could be triggered by excessive precipitation or strong ground shaking associated with an earthquake. In our opinion, a landslide of this nature should not constitute an immediate threat to the integrity of the proposed residence and associated improvements, provided they are designed and constructed in accordance with the recommendations of this report.

Based upon our review of the subsurface conditions defined by prior studies that revealed very dense alluvial deposits at depth and our interpretation of the geologic setting in the site vicinity, it is our opinion that the potential for deep-seated landsliding is negligible.

The long-term stability of many hillside areas is difficult to predict. A hillside will remain stable only as long as the existing slope equilibrium is not disturbed by natural processes or by the acts of Man. Landslides can be activated by a number of natural processes, such as the loss of support at the bottom of a slope by stream erosion or the reduction of soil strength by an increase in groundwater level from excessive precipitation. Artificial processes caused by Man include improper grading activities, the introduction of excess water through excessive irrigation, improperly designed or constructed leachfields, and poorly controlled surface runoff.

Although our knowledge of the causes and mechanisms of landslides has greatly increased in recent years, it is not yet possible to predict with certainty exactly when and where all landslides will occur. At some time over the span of thousands of years, most hillsides will experience landslide movement as mountains are reduced to plains. Therefore, a small but unknown level of risk is always present to structures located in hilly terrain. Owners of property located in these areas must be aware of, and willing to accept, this unknown level of risk.

7.3. Seismicity

Our review of prior studies on the subject property revealed a relatively uniform layer of alluvial fan deposits, with no evidence suggestive of fault offset through the subject site. Lineaments shown on Figure 7 (Nolan Associates, 1999) do not correlate with the trend of the concealed fault trace that was mapped southwest of the subject site by Mclaughlin et. al., 2001 (see Figure 2). We encountered no evidence suggesting that the fault trace crosses beneath the subject site.

Based upon our reconnaissance and review of published geologic maps, literature, aerial photographs, and other consultants' reports, we conclude that even if a trace of the Shannon fault exists near the subject property, there is no evidence that subsurface fault rupture has occurred at the site within the last 11,000 years.

As discussed above, the site is underlain by a thick section of alluvial deposits. Based upon borings performed by T&R and the site's regional settings, we anticipate the alluvial deposits to be several tens of feet thick below the subject site. We conclude that fault movement resulting in displacements within the bedrock beneath the alluvial deposits would propagate upward through uncemented alluvial deposits in a flower pattern. The uncemented alluvial materials would shift and realign, dispersing the amount of deformation at the ground surface. We conclude that there is a low risk for fault related ground deformation to adversely affect the structural integrity of the proposed improvements.

It is reasonable to assume that the site will be subjected to very 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 surface fault rupture through the site is low. Ground deformation may occur as the alluvial materials shift and realign, yet we anticipate total deformations of less than 6 inches across a horizontal distance of several hundred feet. This amount of potential deformation should not pose a safety risk to ordinary structures that are designed and constructed in accordance with current standards.

8. 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 development 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.

As currently planned, a new three-story, single-family residence will be constructed into the northwest-facing hillside, with the two lower levels of the home daylighting toward the northwest. The uppermost level will consist of a two-car garage and roof-top deck. We understand that a bridge will be constructed to serve as a driveway to allow access to the roof-top garage from Bella Vista Avenue. In addition, current project plans indicate that several site and building retaining walls will be constructed. Concrete slabs-on-grade may be used to construct patios, walkways, and the approach portion of the driveway leading from Bella Vista Avenue to the bridge.

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

8.1. Location of Proposed Improvements

The proposed improvements must be confined to the approximate building area shown on Figure 9. 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 locations to provide appropriate geotechnical engineering design criteria.

8.2. 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 = 37.2262º
Longitude = -121.9706º
Site Class = C
$S_s = 2.654$ $S_1 = 1.012$
$F_a = 1.0$ $F_v = 1.3$

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.

8.3. Earthwork

At the time of this study, the full extent of any proposed earthwork had not been finalized. We anticipate that a moderate amount of grading will be required to construct the proposed improvements. Any proposed earthwork should be performed in accordance with the recommendations provided below.

8.3.1. Clearing and Site Preparation

- Clear all obstructions, including brush, trees not designated to remain, and debris on any areas to be graded.
- Clear and backfill any holes or depressions resulting from the removal of underground obstructions below proposed finished subgrade levels with suitable material compacted to the requirements for engineered fill given below.
- After clearing, strip the site to a sufficient depth to remove all surface vegetation and organic-laden topsoil. This material must not be used as engineered fill; however, it may be used for landscaping purposes.

8.3.2. Fill Material

- Based on our review of the boring logs and laboratory data from prior studies, it is our opinion that the materials encountered in the borings should be suitable for use as fill. The on-site materials meeting the requirements specified below may be used as engineered fill.
- Materials used for engineered fill must meet the following requirements:
  1) They must have an organic content of less than 3% by volume,
  2) no rocks or lumps greater than 6 inches in maximum dimension, and
  3) no more than 15% of the fill may be greater than 2½ inches in maximum dimension.
- If on-site materials do not meet the requirements given above, they may be off-hauled or used for landscaping purposes only.
- In addition to the requirements above, any imported fill must have a plasticity index (PI) of 15% or less.

8.3.3. Benches

- Fill placed on slopes in excess of 5:1 must be benched into the underlying, supportive alluvial fan deposits to provide a firm, stable surface for support of the fill.
- Benches generally must be a minimum of 8 feet wide and must be excavated entirely into the supportive alluvial fan deposits.
- Temporary backslopes may be vertically excavated, provided they are constructed in the dry season and meet Cal OSHA requirements.
- Any required benches must be excavated near level in the direction parallel to the natural slope and must be provided with an approximately 2% gradient sloping into the hillside to provide resistance to lateral movement and to facilitate proper subdrainage.
- **C2 must evaluate the actual location, size, and depth of the required keyway and benches at the time of construction**.

8.3.4. 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).
- **C2 must observe the placement and test the compaction of engineered fill.** Provide at least two working days notice prior to placing fill.

8.3.5. 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.
- 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.**

8.3.6. Daylighting Basement Excavation

- Excavate the basement using shoring or an OSHA approved benching or sloping cut configuration selected by an OSHA "Competent Person".
- The contractor is solely responsible for the means and methods of construction and should designate appropriate personnel to act as the Competent Person.
- To aid the Competent Person in their selection of construction means and methods, consider the on-site alluvial fan deposits to be an OSHA Soil Type A. This soil classification must be evaluated and validated by the Competent Person during construction.

8.4. Foundations

Because the basement will be excavated into the hillside, we anticipate that the excavation will expose supportive alluvial deposits. Thus, we recommend that the residence be supported on a mat-slab foundation at the basement level, gaining support in the underlying alluvial deposits. We understand that some areas of the main level may extend laterally uphill or downhill of the basement. We recommend that these areas be structurally cantilevered to be fully supported by the basement walls.

Additionally, we recommend that the northwestern portion of the proposed bridge be structurally connected to the house and the southeastern abutment be supported by drilled, cast-in-place, straight-shaft concrete friction piers gaining support in the underlying alluvial fan deposits.

We understand that three patios are planned within light wells on the downslope side of the home. We recommend that the floor of the patios be constructed either as concrete slabs-on-grade or be integral with the mat-slab foundation for the home. Site retaining walls must be supported on either drilled pier foundations or conventional spread footings, in accordance with the recommendations provided below under the section headed "Retaining Walls."

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

8.4.1. Mat-Slab

- Support the proposed basement on a mat-slab embedded a minimum of 12 inches into the underlying alluvial deposits, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the excavation and the surface of the alluvial deposits.
- Design support for the mat-slab in the alluvial deposits 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.
- Lateral loads may be resisted by friction between the concrete mat bottom and the supporting subgrade using a friction coefficient of 0.35. If a waterproofing membrane will be placed between the bottom of the mat and the supportive

subgrade, the friction coefficient will be compromised and lateral loads must be resisted by passive pressure or other means.

- As an alternative, a passive pressure equal to an equivalent fluid weight of 350 pcf may be used for the mat if it is poured neat in excavations into the supportive material, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the excavation and the surface of the alluvial deposits.
- Use either passive pressure or the friction coefficient to design for lateral loading. Lateral loads resistance must not combine the use of the friction coefficient and passive pressure.
- We anticipate differential and total settlement of the mat slab founded in supportive material to be less than 1 inch.
- Concrete reinforcing must be provided in accordance with the recommendations of the structural design engineer.
- Provide the mat-slab with appropriate damp proofing. Damp proofing may affect the lateral load resistance (see above).
- **Contact C2 to observe the excavations prior to placing reinforcing steel** to evaluate depth into supportive material.

8.4.2. Drilled Piers

- Drill piers with a minimum diameter of 16 inches and embed them a minimum of 8 feet, or the depth of overburden (whichever is greater), into the underlying alluvial deposits, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the pier and the surface of the alluvial deposits.
- Total pier depth will vary across the building site depending on the depth of the non-supportive soil and the extent of grading. Based on our subsurface exploration, we anticipate that pier depths in the vicinity of the proposed bridge abutment may reach 20 feet below the existing ground surface.
- Design the portion of the piers in the alluvial deposits using a skin friction value of 400 psf for dead plus live loads, with a 1/3 increase for transient loads, including wind and seismic.
- Neglect any portion of the piers in fill and non-supportive colluvium and any point-bearing resistance for support.
- Figure active loads on the upper portion of the piers in the fill and colluvium on the basis of an equivalent fluid weight of 40 pcf taken over **2 times** the pier diameter. The depth of the active loads will vary across the building site depending on the depth of grading. Where the fill and surficial soil is removed by

grading, active loads will be negligible. Where proposed structures are built at existing grades, active loads may extend to depths of approximately 7 feet.

- Design for resistance to lateral loads using a passive pressure equal to an equivalent fluid weight of 350 pcf to a maximum of 3,000 psf taken over **1½ times** the pier diameter for the length of the piers in the alluvial deposits, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the pier and the surface of the alluvial deposits (see Figure 11, Conceptual Pier Pressure Diagram).
- Consider active and passive pressure loading to be negligible within the upper 2 feet of the alluvial deposits in order to account for the orientation to achieve the recommended 5 feet of horizontal separation.
- Anticipate differential and total settlement for piers founded in supportive material to be less than 1 inch.
- Because the alluvial deposits contain large boulders that can be locally very hard, it may be difficult to drill. The contractor should plan for this condition and choose the appropriate means and methods of drilling.
- Clear the bottoms of the pier excavations of loose cuttings and soil fall-in prior to the installation of the reinforcing steel and the placement of concrete.
- Remove any accumulated water in the excavations prior to the placement of steel and concrete.
- Use sono tubes in the tops of the holes to prevent overpour (mushrooming) of the concrete.
- Reinforce the piers with a full-length cage containing a minimum of four No. 5 steel reinforcing bars.
- The structural engineer must determine the actual number, size, location, depth, spacing, and reinforcement of the piers, based on the anticipated bridge and retaining wall loads and the soil engineering design parameters provided above.
- **Contact C2 to observe the piers as they are being drilled** to verify that the piers are founded in material of sufficient supporting capacity.

8.4.3. Bridge Abutment 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 piers.
- 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.

8.4.4. Spread Footing

- Embed spread footings for site retaining walls a minimum of 12 inches into the underlying, supportive alluvial deposits, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the footing and the surface of the alluvial deposits.
- Design the spread footings supported in the alluvial deposits 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.
- As an alternative, a passive pressure equal to an equivalent fluid weight of 350 pcf may be used for footings poured neat in excavations into the alluvial deposits, below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the footing and the surface of the alluvial deposits.
- Use either passive pressure or the friction coefficient to design for lateral loading. Lateral loads resistance must not combine the use of the friction coefficient and passive pressure.
- 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.
- Design for differential and total settlement for footings founded in supportive material of less than 1 inch.
- Clear the bottoms of the footing excavations of loose cuttings and soil fall-in prior to the placement of concrete.
- **C2 must observe the footing excavations prior to placing reinforcing steel** to evaluate depth into supportive material.

8.4.5. Retaining Walls

We anticipate that site and building retaining walls will be used to develop the property. The following recommendations are for cantilever type walls. Contact us to provide appropriate recommendations if you consider other types of walls.

- Support residential basement retaining walls on mat-slab foundations designed in accordance with the recommendations given above for the support of the proposed residence. Support site retaining walls on pier or spread footing foundations.

- Design retaining walls to resist both lateral earth pressures and any additional lateral loads caused by surcharge loads on the adjoining ground surface.
- Deflection of cantilever retaining walls will occur in response to lateral loading. Anticipate horizontal deflections at the top of the wall to be 2 percent of the wall height or less.
- Design unrestrained (active condition) walls with level backfill to resist an equivalent fluid pressure of 40 pcf. Design walls that are restrained from movement at the top or sides (at-rest condition) with level backfill to resist an equivalent fluid pressure of 62 pcf (see Figure 12, Conceptual Retaining Wall Pressure Diagram).
- Add an additional equivalent fluid pressure increment to the active and at-rest condition for sloping backfill, in accordance with the following:

  +5 pcf for slopes up to 4:1 (horizontal to vertical)

  +8 pcf for slopes between 3:1 and 4:1

  +12 pcf for slopes between 2:1 and 3:1

  Contact us to provide additional recommendations for slopes steeper than 2:1

- Design for seismic-loading as the structural engineer deems appropriate. In our opinion, the requirements for seismic design of retaining walls are not clearly defined. If the structural engineer considers seismic loading based upon the procedures presented by Sitar, et. al. (2012), design unrestrained (active condition) residential retaining walls to resist an additional earthquake equivalent fluid pressure (seismic increment) of 25 pcf.
- If seismic loading is considered, design basement retaining walls to resist the most critical loading: either the at-rest condition if the walls are restrained, or the active condition plus the seismic increment if the walls are unrestrained.
- Site walls are not subject to additional earthquake loading requirements.
- Wherever the walls will be subjected to surcharge loads, they must be designed for an additional uniform lateral pressure equal to 1/2 or 1/3 the anticipated surcharge load for restrained or unrestrained walls, respectively.
- The preceding pressures require that sufficient drainage be provided behind the walls to prevent the buildup of hydrostatic pressures from surface or subsurface water infiltration.
- Provide a backdrain system consisting of an approximately 1-foot thick curtain of drainrock (crushed rock or gravel) placed behind the wall.

- Separate the drainrock from the backfill by a geotextile filter fabric, such as Mirafi 140 or an alternative, approved by C2. A 4-inch diameter, heavy-duty, rigid, perforated subdrain pipe (Schedule 40, SDR 21 or equivalent), approved by C2, must be placed with the perforations down on a 2- to 3-inch layer of drainrock at the base of the drain. Where subdrain pipes will be buried deeper than 10 feet, Schedule 80 or equivalent pipe should be used. **Do not use flexible corrugated pipe.**
- As an alternative, back drainage may consist of an approved drainage mat placed directly against the wall. The bottom of the drainage mat must be in contact with the rigid, 4-inch diameter, perforated drainpipe embedded in gravel. The mat's filter fabric must be placed around the drainpipe and between the pipe and the soil.
- The backdrains should extend up the height of the back of the retaining walls to within 1 foot of the height of the retained soil and then be covered with a compacted clay soil cap.
- Details of backdrain options are presented on Figure 13, Conceptual Retaining Wall Backdrain Diagram.
- Perforated retaining wall subdrain pipes must be dedicated pipes and must not connect to the surface drain system. Install the subdrain pipes with a positive gradient of at least 1% and provide them with clean-out risers at their up-gradient ends and at all sharp changes in direction. Changes in pipe direction must be made with "sweep" elbows to facilitate future inspection and clean-out. The perforated pipes must be connected to buried solid pipes to convey collected runoff to discharge onto an energy dissipater at an appropriate downhill location, approved by C2.
- Energy dissipaters may consist of a short "T" fitting placed in a shallow trench and covered with a mound of cobbles (see Figure 14, Conceptual Energy Dissipater Diagram). The discharge must not be located on, or adjacent to, steep, potentially unstable terrain or where runoff will adversely impact adjacent parcels.
- Compact the backfill placed behind the walls to at least 90% relative compaction, using light compaction equipment, in accordance with the compaction procedures given above. If heavy compaction equipment is used, the walls should be appropriately temporarily braced, as the situation requires. If backfill consists entirely of drainrock, it should be placed in approximately 2-foot lifts and must be compacted with several passes of a vibratory plate compactor.
- Perform annual maintenance of retaining wall backdrain systems. This maintenance must include inspection and flushing to make sure that subdrain pipes are free of debris and are in good working order, and inspection of subdrain outfall locations to verify that introduced water flows freely through the discharge pipes and that no excessive erosion has occurred.

- If erosion is detected, C2 must be contacted to evaluate its extent and to provide mitigation recommendations, if needed.
- Provide retaining walls that are adjacent to living spaces and site walls with decorative facing with appropriate damp proofing. We are not qualified to recommend specific damp proofing materials or their applications. Any damp proofing product must be applied in **strict** compliance with the manufacturer's and/or architect's specifications.
- If you select an alternative retaining wall type, you should contact C2 to provide additional recommendations.

8.4.6. Flatwork

We anticipate that concrete slabs-on-grade may be used for the approach portion of the driveway, patios, and walkways. Where located on undocumeted fill and/or colluvium, the overlying flatwork will be subject to downslope migration and/or differential movement. We believe that this condition will result in minor, ongoing cosmetic damage to the flatwork. To mitigate the risk of differential movement of the flatwork, we recommend the following options:

- Option 1: Construct the flatwork using a flexible pavement system that can accommodate differential movement, such as pavers.
- Option 2: Remove and replace the colluvium and/or artificial fill with engineered fill, benched into the supportive alluvial deposits in accordance with the recommendation provided above.
- Option 3: Construct the flatwork to be structually supported on foundations gaining support within the alluvial deposits in accordance with the pier and/or footing recommendations provided above.

For concrete slabs-on-grade, we recommend the following minimum requirements:

- Support concrete slabs-on-grade on a minimum of 6 inches of non-expansive fill compacted to the requirements for compacted 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.

Where floor wetness is undesirable:

- The building designer or qualified waterproofing consultant must provide moisture barrier requirements.
- The following recommendations are typical moisture barrier standards. We do not guarantee that these measures will prevent all future moisture intrusion. If necessary, you should contact a qualified waterproofing consultant to provide waterproofing design.
- Traditionally, designers have specified the following: place 4 inches of free-draining gravel beneath the floor slab to serve as a capillary barrier between the subgrade soil and the slab. Following gravel placement, place a heavy-duty membrane over the gravel in order to minimize vapor transmission and then place 2 inches of sand over the membrane to protect it during construction. Just prior to placing concrete, lightly moisten the sand.
- More recent standards suggest using a puncture resistant, heavy-duty membrane (such as a minimum of 15 mil Stego Wrap, or equivalent) in direct contact with the floor slab and underlain by 6 inches of free-draining gravel.
- The structural designer must evaluate moisture conditions related to concrete slab curing and performance. The builder must provide appropriate drying time as determined by the designer.
- Use the gravel, heavy-duty membrane, and/or sand (if specified) in lieu of the upper 6 inches of recommended non-expansive fill.

8.5. Drainage

Based upon our review of a preliminary grading and drainage plan by TS Civil Engineering, we understand that the site surface drainage will be collected and will discharge into a detention system planned for the lower, central-northwestern portion of the property.

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, erosion, gullying, ponding, and potential slope instability. To mitigate the risk of improperly controlled runoff, we recommend that you implement the following:

- Prevent surface water from ponding in pavement areas and adjacent to the foundation of the proposed residence and associated improvements.
- Construct pavement areas for proper drainage by sloping them away from structures and by providing area drains.
- Provide the ground surface with a positive gradient sloping away from structures to mitigate ponding water adjacent to the foundations, or as an alternative, install area drains to collect surface runoff.
- Provide roof gutters and 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 away from the residence via buried, closed conduits or lined surfaces.
- Discharge collected water in an appropriate manner and at an appropriate location approved by C2. Do not locate the discharge on, or adjacent to, steep, potentially unstable terrain.
- Use buried conduits consisting of rigid, smooth-walled pipes (PVC). **Do not use flex-pipes.**
- Provide downspouts with slip-joint connectors or clean-outs, where they are connected to buried pipes, to facilitate maintenance (see Figure 15, Conceptual Downspout Clean-Out Diagram).
- Convey all collected water away from the structures via buried, closed conduit or hard surfaced drainage way and discharge into the on-site detention system at an appropriate downslope location approved by C2. The discharge must not be located on, or adjacent to, steep, potentially unstable terrain or where runoff will adversely impact adjacent parcels.
- 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 and that no excessive erosion has occurred.
- Contact C2 if erosion is detected so that we may evaluate its extent and provide mitigation recommendations, if needed.

9. 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. **WE MUST BE GIVEN AT LEAST ONE WEEK TO REVIEW THE PLANS AND PREPARE A PLAN REVIEW LETTER.**

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.

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.

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

1. **EARTHWORK:** During site grading to observe benching into supportive material and test the compaction of engineered fill
2. **MAT-SLAB:** Near completion of the mat-slab excavation to evaluate depth to supportive material
3. **DRILLED PIER EXCAVATION:** During drilling to evaluate depth to supportive material and final pier depths
4. **FOOTING EXCAVATION:** Prior to placement of reinforcing steel to evaluate depth to supportive material
5. **RETAINING WALL BACKDRAIN:** During installation
6. **RETAINING WALL BACKFILL:** During backfill to observe and test compaction
7. **SLABS-ON-GRADE:** Prior to and during placement of non-expansive fill to observe the subgrade preparation and to test compaction of non-expansive fill
8. **SURFACE DRAINAGE SYSTEMS:** Near completion to evaluate installation and discharge locations

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

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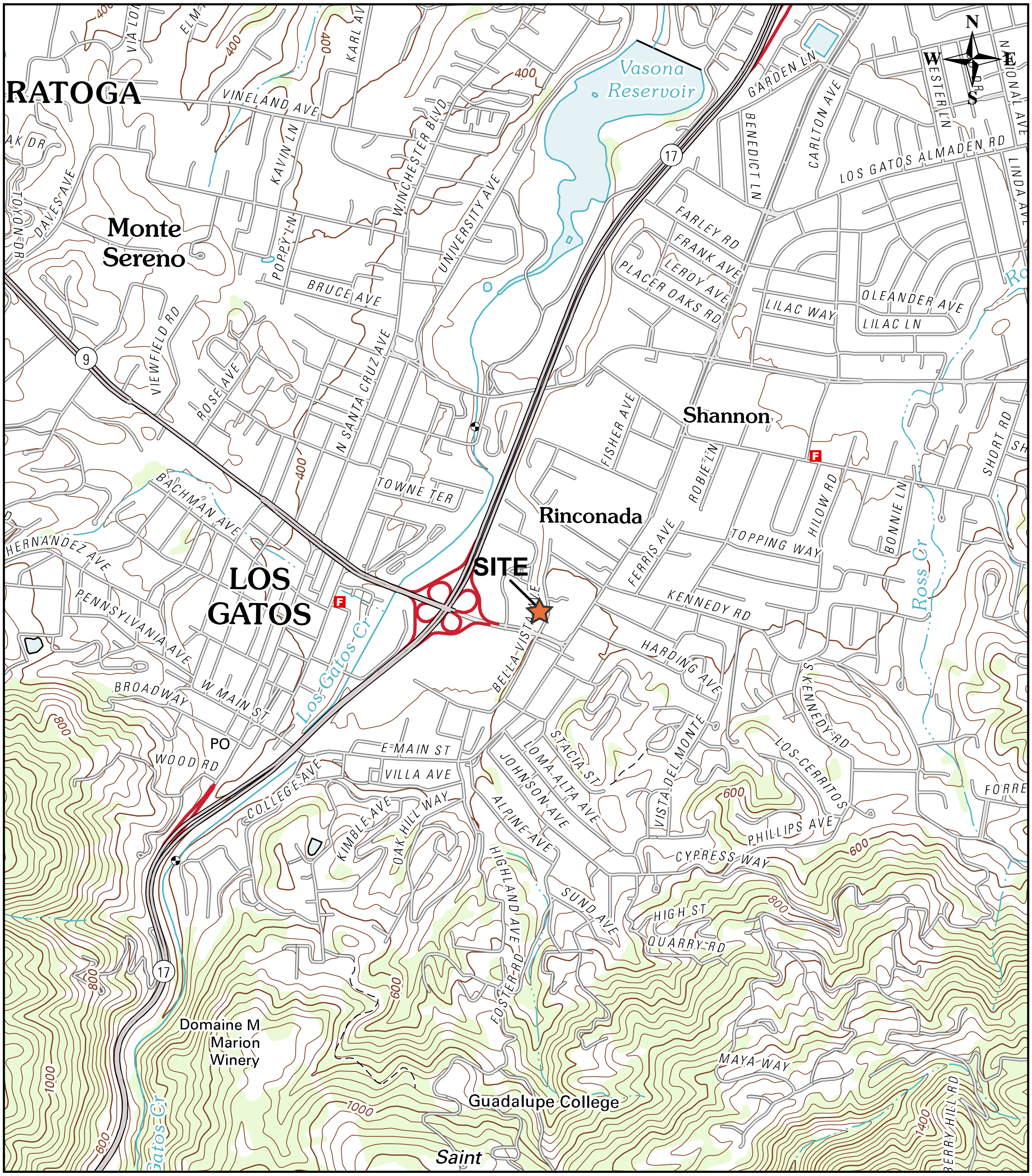
FIGURES AND TABLE

FIGURE NO.

APPENDIX

TABLE NO.



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

SITE LOCATION MAP

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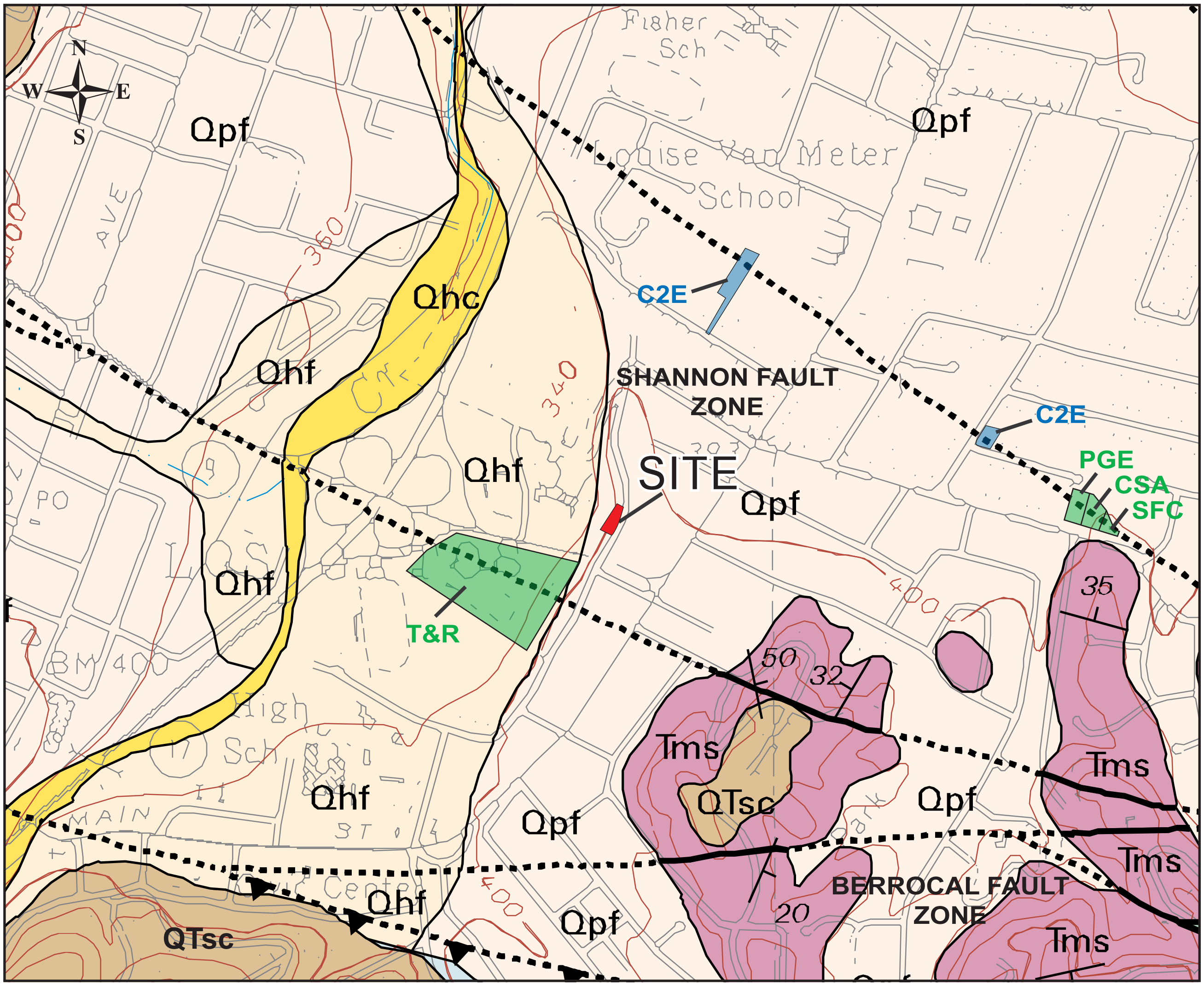
ROSS PROPERTY
339 and 341 Bella Vista Avenue
Los Gatos, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	1" = 2,000'	15068C-01R1	June 2015	Figure 1



EXPLANATION

Qhf - Alluvial fan deposits (Holocene)
Qhc - Stream channel deposits
Qpf - Alluvial fan deposits (Pleistocene)
QTsc - Santa Clara formation
Tms - Monterey shale
Jos - Serpentinized ultramafic rock
fm - Melange
fpv - Volcanic rocks

15 Strike and dip
Geologic contact
Fault
dashed where approximate and dotted where concealed
Thrust fault
barbs on upper plate

C2E - Studies by C2Earth, Inc.

Studies by Other Consultants:

T&R - Treadwell & Rollo, 2008
PGE - Pacific Geotechnical Engineering, 2005
CSA - Cotton, Shires & Associates, Inc., 1999
SFC - Steven F. Connelly, C,E.G., 2007

BASE: Sheet 1: Los Gatos Quadrangle, Geologic Maps and Structure Section of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California; MCLAUGHLIN, ET AL.; 2001

REGIONAL GEOLOGIC MAP

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JB/CH	1" = 1,000'	15068C-01R1	June 2015	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; Los Gatos Quadrangle; California Geological Survey; 9-23-02

REGIONAL SEISMIC HAZARD ZONES MAP

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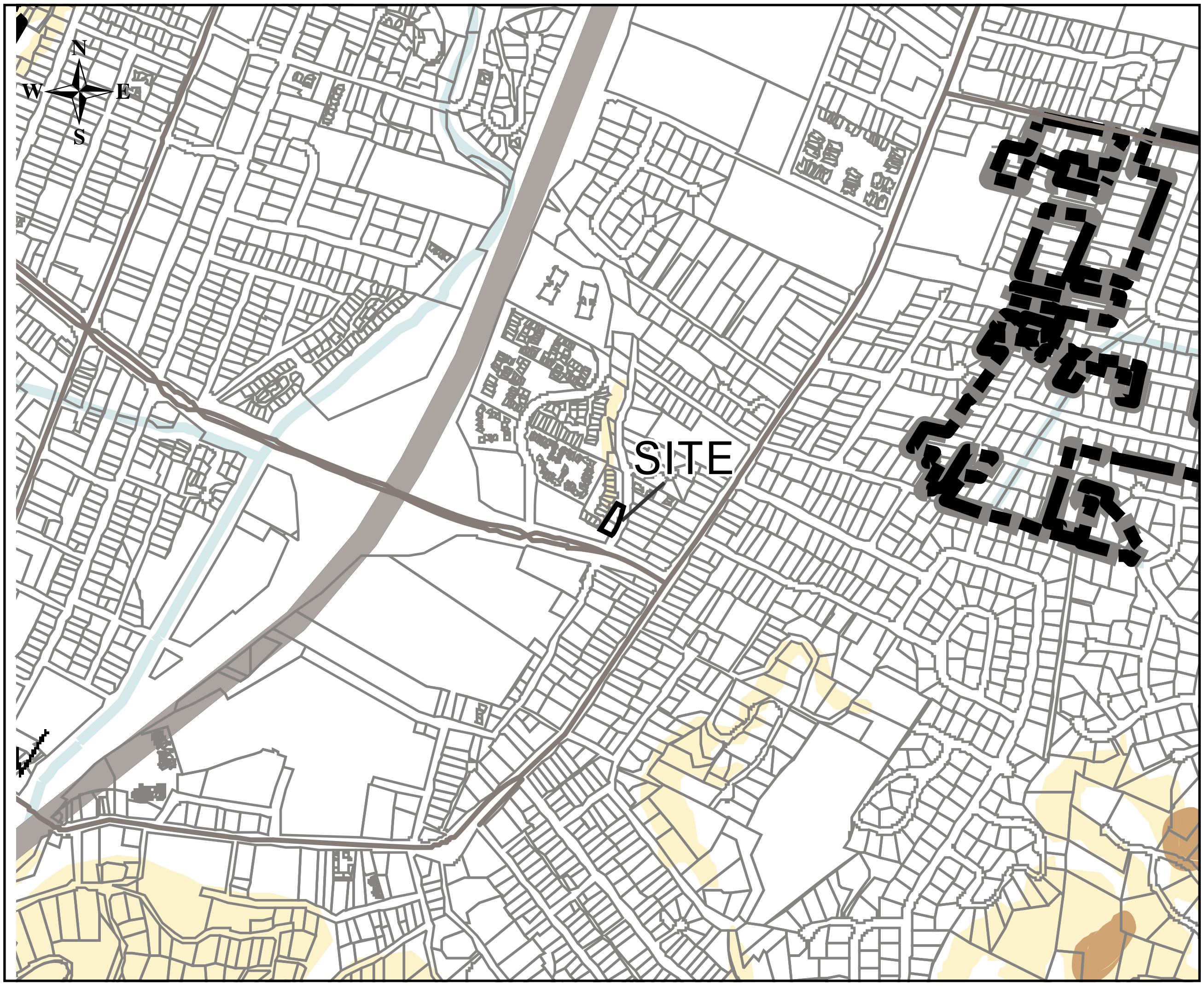
JB/CH	1" = 1,000'	15068C-01R1	June 2015	Figure 3



EXPLANATION

- Town Boundary
- Creek
- Severe Landslide Hazard
- Landslide Potential

BASE: Town of Los Gatos General Plan Update; Landslide Hazard Areas, Figure 16-4; SANTA CLARA COUNTY; 2006

LOCAL LANDSLIDE HAZARD AREAS MAP

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JB/CH	1" = 1,000'	15068C-01R1	June 2015	Figure 4

EXPLANATION

Fault Rupture Potential

- Town Boundary
- Creek
- High
- Moderate

BASE: Town of Los Gatos General Plan Update; Fault Rupture Hazard Zones, Figure 16-2; NOLAN ASSOCIATES; 1999

LOCAL FAULT RUPTURE POTENTIAL MAP

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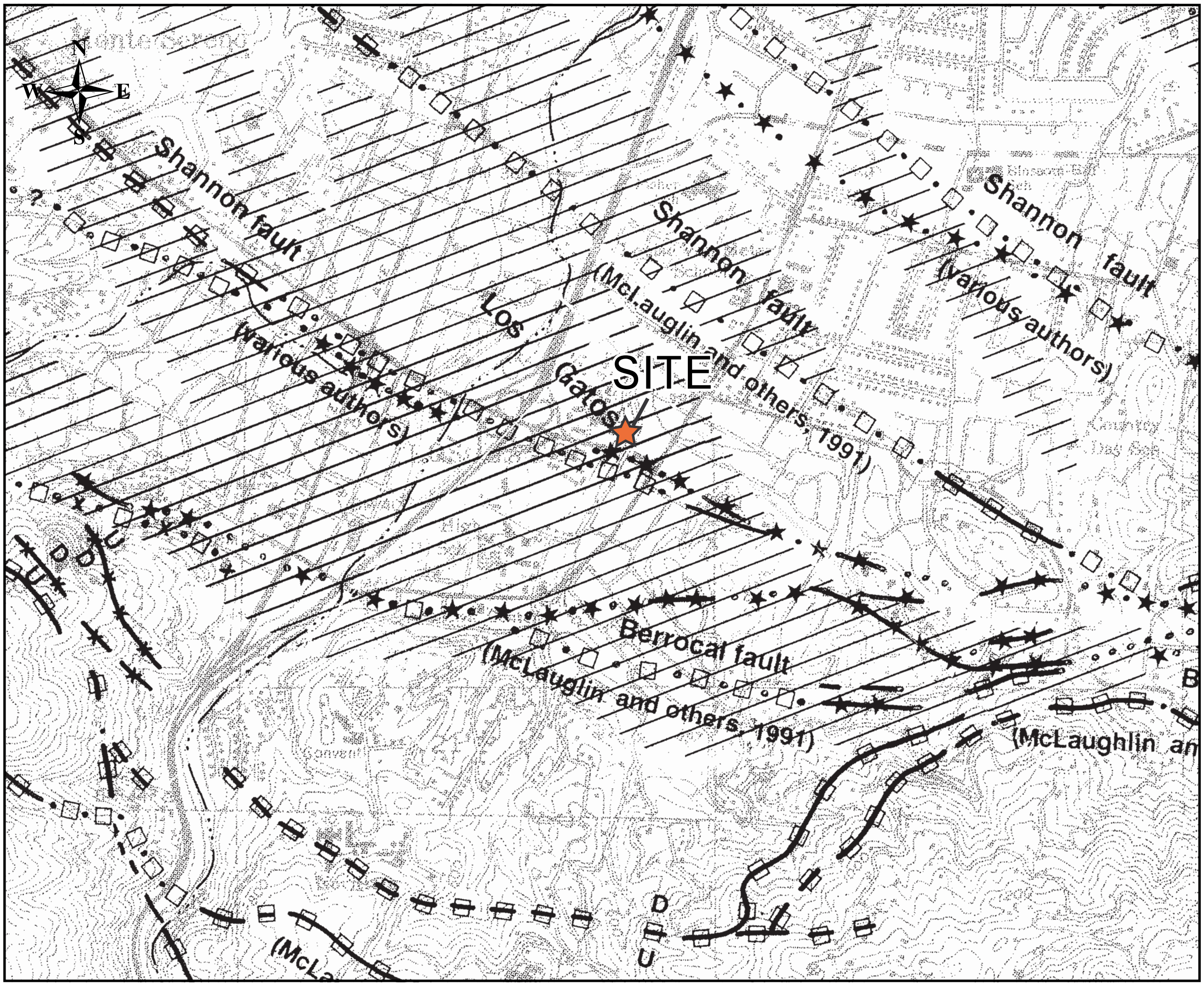
JB/CH	1" = 1,000'	15068C-01R1	June 2015	Figure 5



EXPLANATION

- **Zone of lineaments**

- **Fault**
  dashed where approximately located
  dotted where concealed
  U = up D = down

Previously identified faults:

Bailey and Everhart (1964)

Mc Laughlin and others (1991)

BASE: Faults and Folds Mapped During Previous Investigations and Zones of Lineaments Between Los Altos Hills and Los Gatos, California; Plate #1; WILLIAM LETTIS & ASSOCIATES, INC.; March 1994

FAULTS, FOLDS, AND ZONES OF LINEAMENTS MAP

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DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	1" = 2,000'	15068C-01R1	June 2015	Figure 6

EXPLANATION

- **Concentration of coseismic ground deformation**
- **Fault trace**
  dashed where uncertain
  dotted where concealed
- **Lineation indicative of faulting. Interpreted from aerial photograph analysis.**
  v = vegetation
  t = tonal
  ld = linear depression
  lf = linear front
  s = saddle

BASE: Town of Los Gatos General Plan Update; Fault, Lineament & Coseismic Deformation Map; Plate #3; NOLAN ASSOCIATES; 1/17/99

FAULT, LINEAMENT, AND COSEISMIC DEFORMATION MAP

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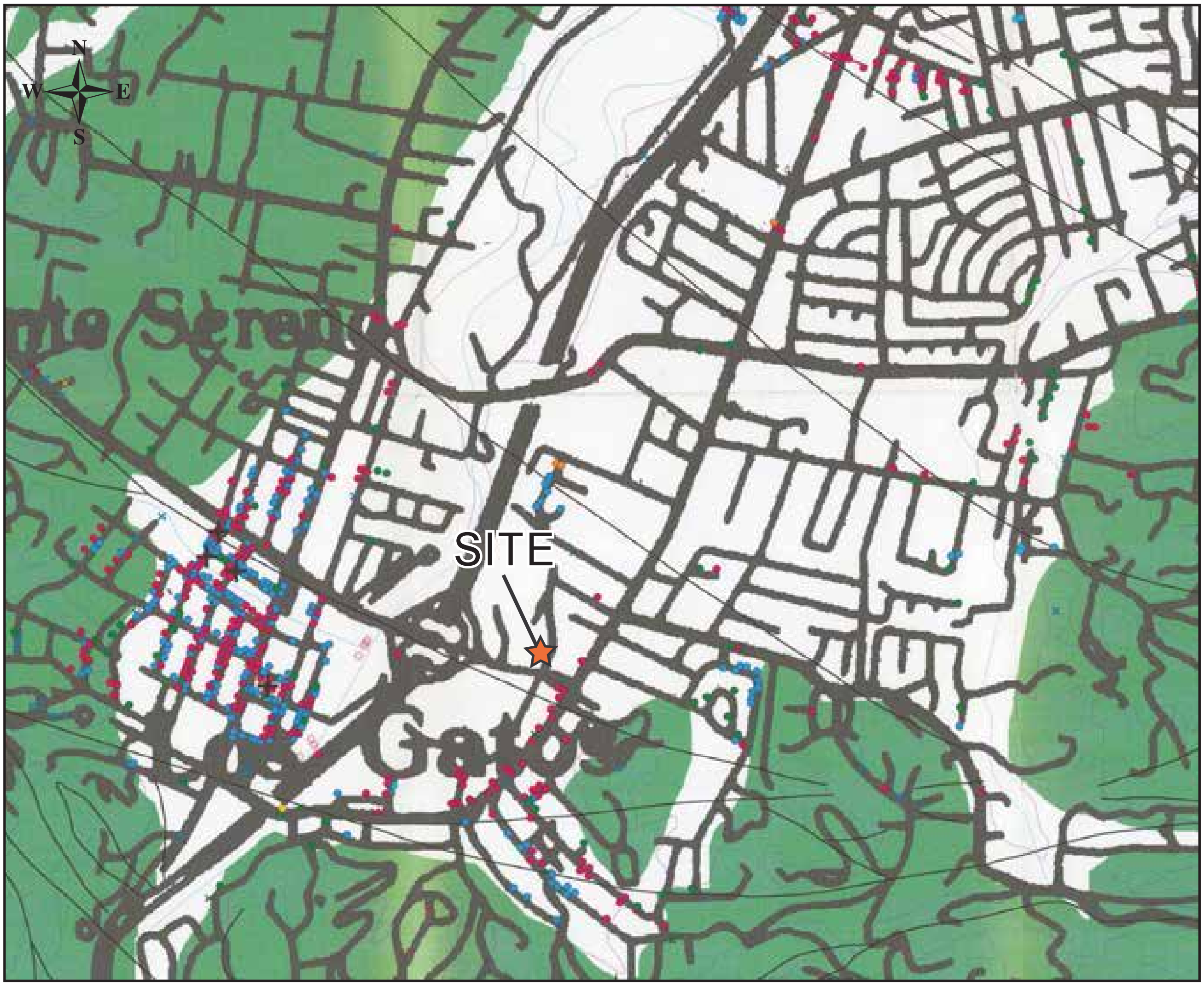
Los Gatos, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	1" = 2,000'	15068C-01R1	June 2015	Figure 7



EXPLANATION

- Fresh pavement break or buckle suggestive of contractional deformation (reported by USGS; some also reported by local governments)
- Apparently fresh pavement break with unspecified sense of deformation (reported by USGS; some also reported by local governments)
- Pavement break with unspecified sense of deformation (reported by local governments)
- Combination of pre-earthquake and coseismic break in pavement (reported by USGS and local governments)

BASE: Map of Pavement and Pipe Breaks As Indicators of Range-Front Faulting Resulting From the 1989 Loma Prieta Earthquake; SCHMIDT ET AL; 1995

MAP OF OBSERVED DISTRESS FROM THE 1989 LOMA PRIETA EARTHQUAKE

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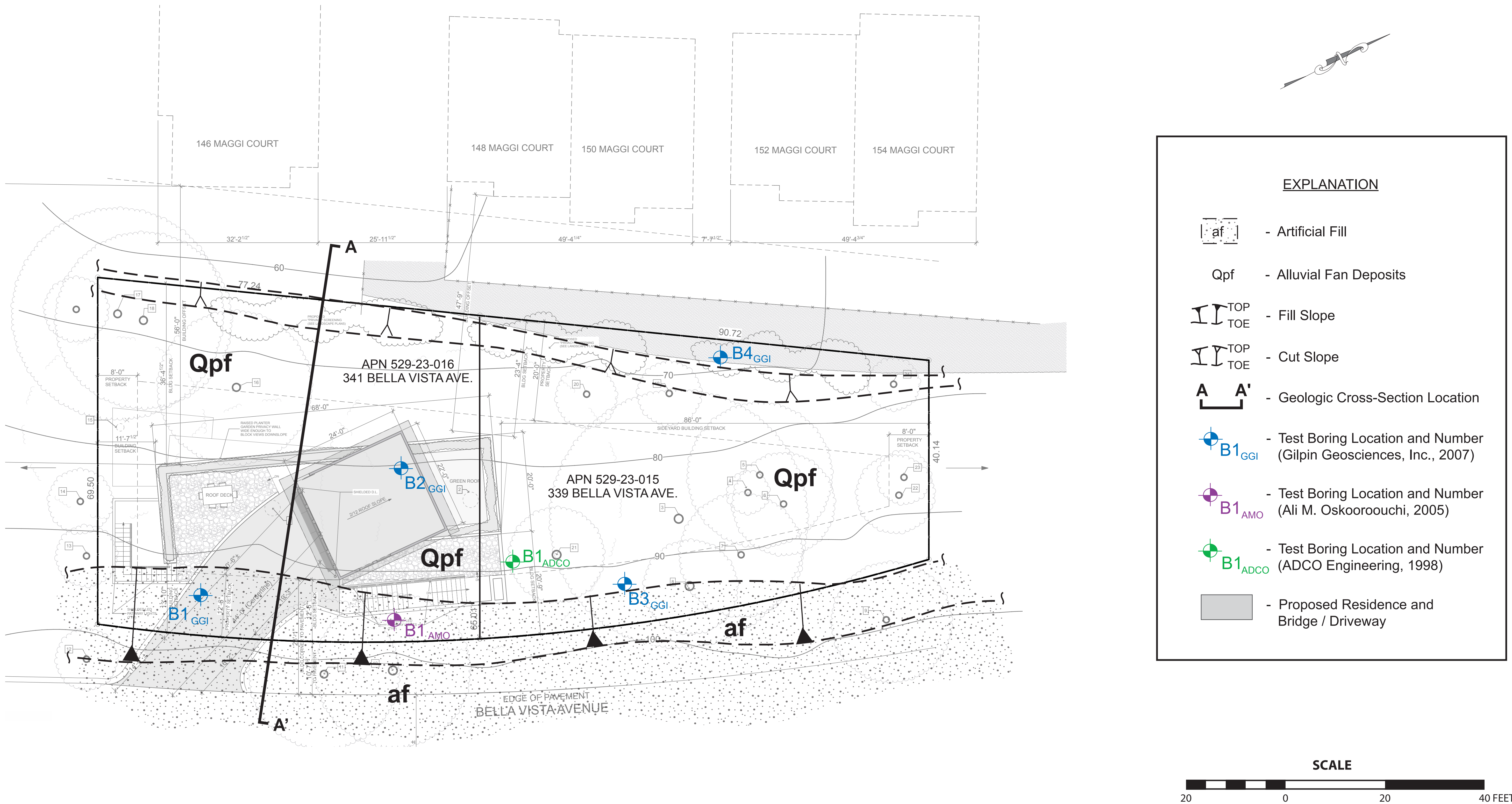
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	1" = 2,000'	15068C-01R1	June 2015	Figure 8



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 A-1.2; 13 November 2014

UPDATED SITE PLAN AND ENGINEERING GEOLOGIC MAP

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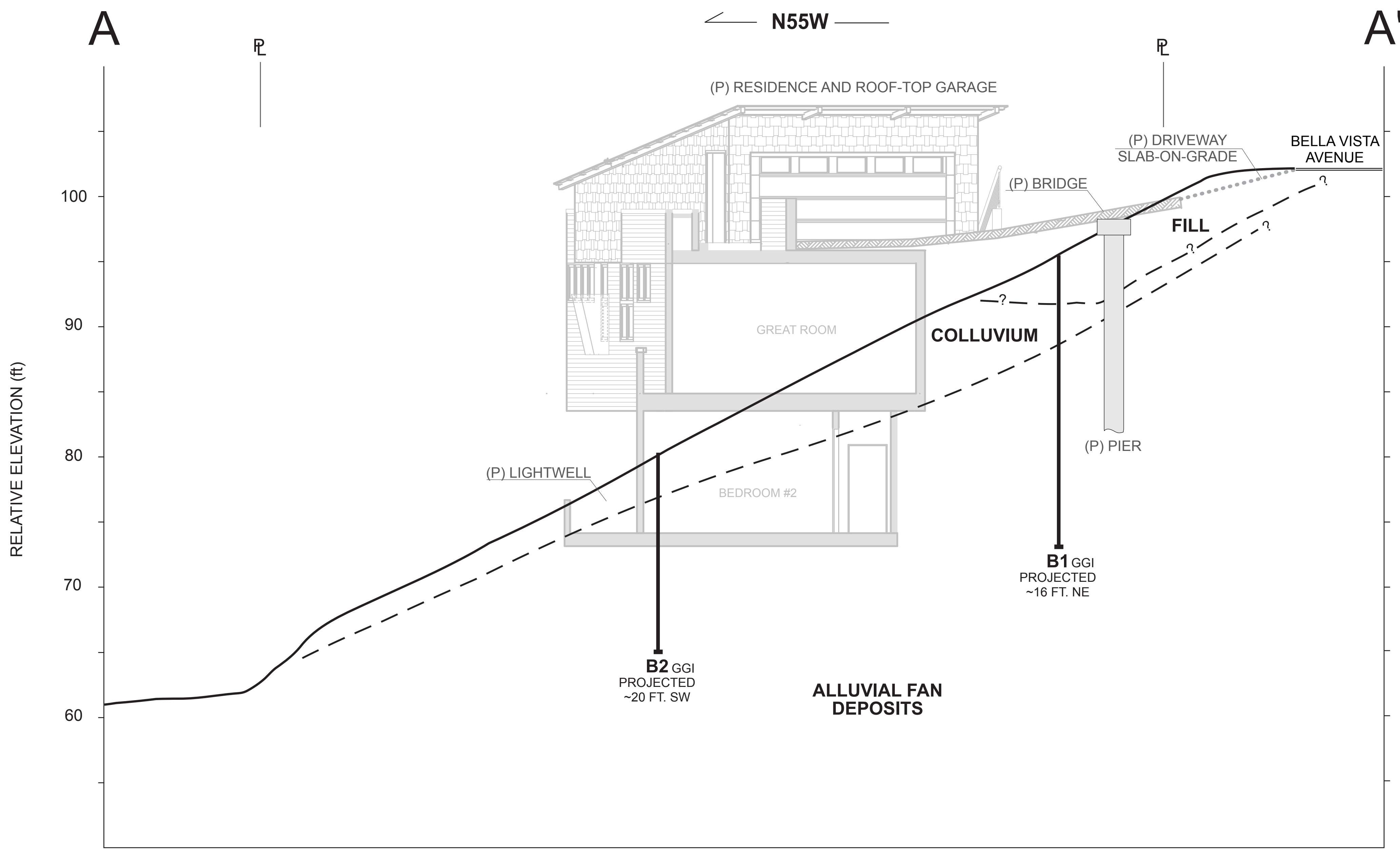
JB/CH	As Shown	15068C-01R1	June 2015	Figure 9



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

BASE: Geologic Cross-Section A-A'; Figure 4; GILPIN GEOSCIENCES, INC.; 14 March 2007
Building Section 2; Sheet A-4.2; 7 November 2014

UPDATED GEOLOGIC CROSS-SECTION A-A'				
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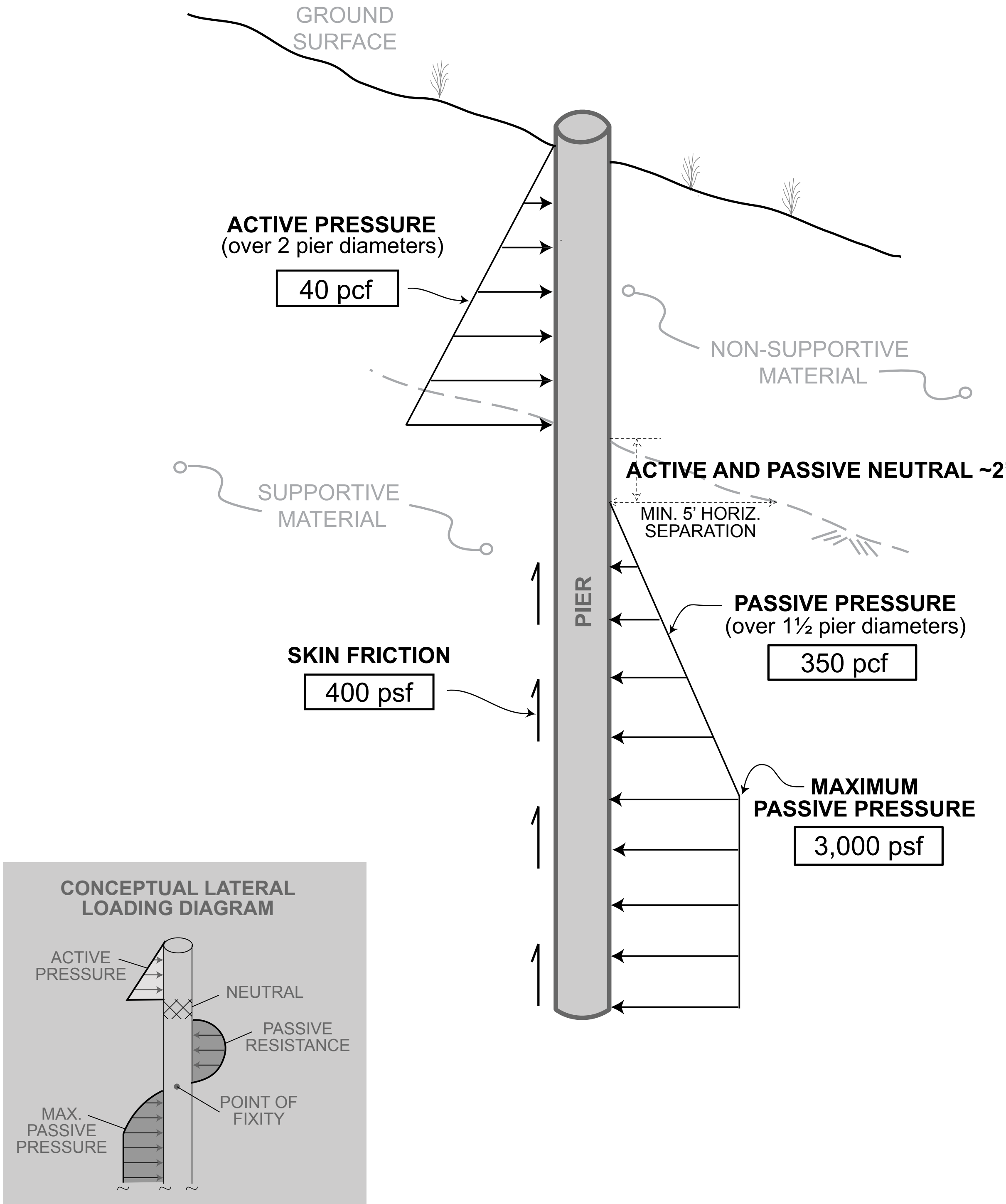
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	1"= 10'	15068C-01R1	June 2015	Figure 10



CONCEPTUAL PIER PRESSURE DIAGRAM

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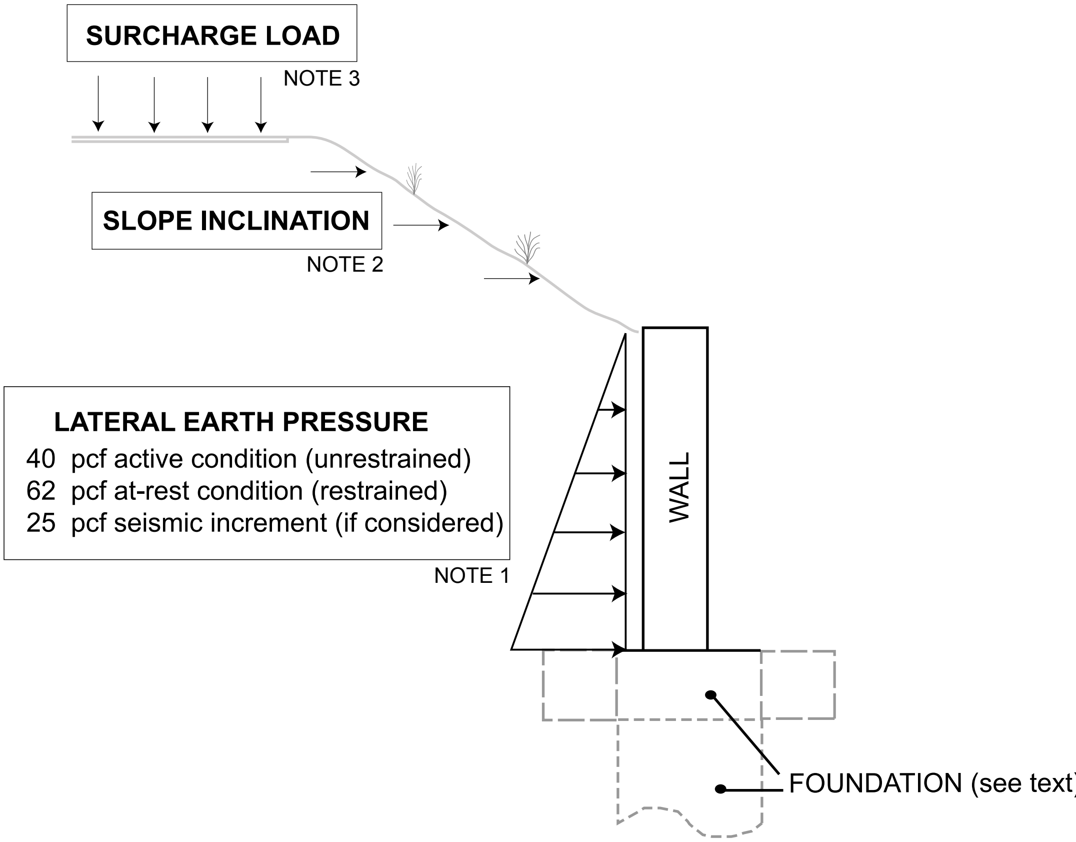
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	Not Applicable	15068C-01R1	June 2015	Figure 11



Note 1: Lateral earth pressures are shown for drained retaining walls. Contact us to provide additional recommendations if undrained walls are planned.

Note 2: Add an additional equivalent fluid pressure increment to the active and at-rest condition for sloping backfill above the wall:

+5 pcf for slope inclinations up to 4:1 (horizontal to vertical)
+8 pcf for slope inclinations between 3:1 and 4:1
+12 pcf for slope inclinations between 2:1 and 3:1

Note 3: Additional lateral load equal to 1/3 (unrestrained) or 1/2 (restrained) the anticipated surcharge load.

CONCEPTUAL RETAINING WALL PRESSURE DIAGRAM

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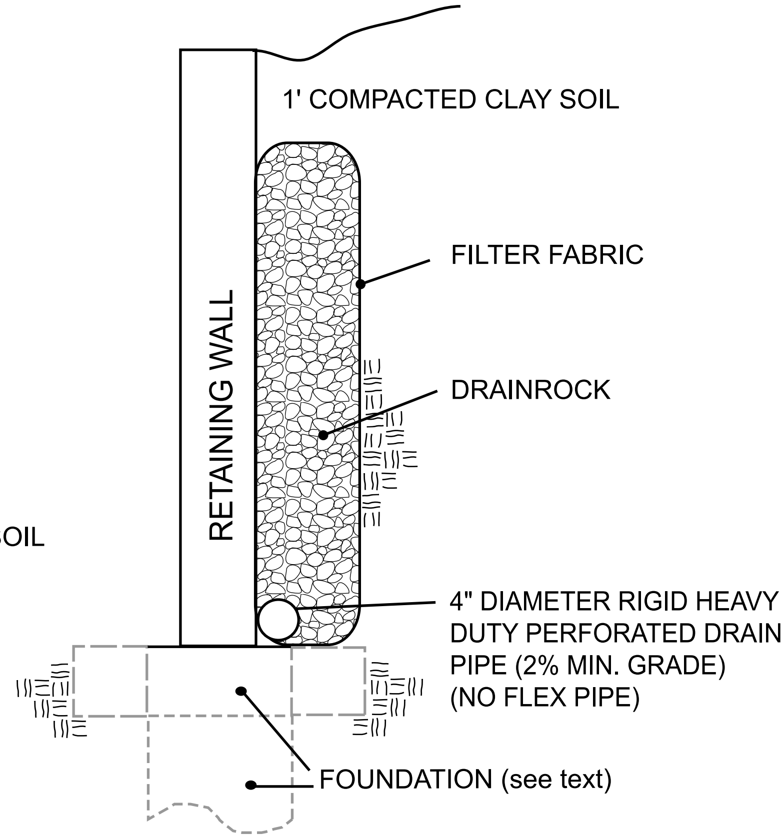
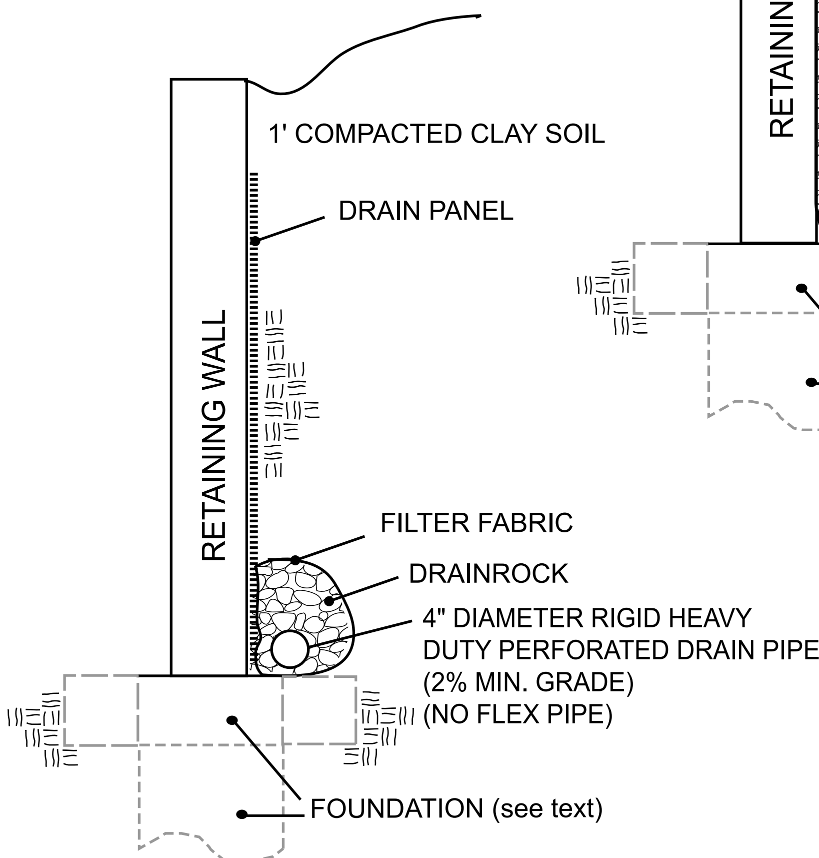
JB/CH	Not Applicable	15068C-01R1	June 2015	Figure 12





CONCEPTUAL RETAINING WALL BACKDRAIN DIAGRAM

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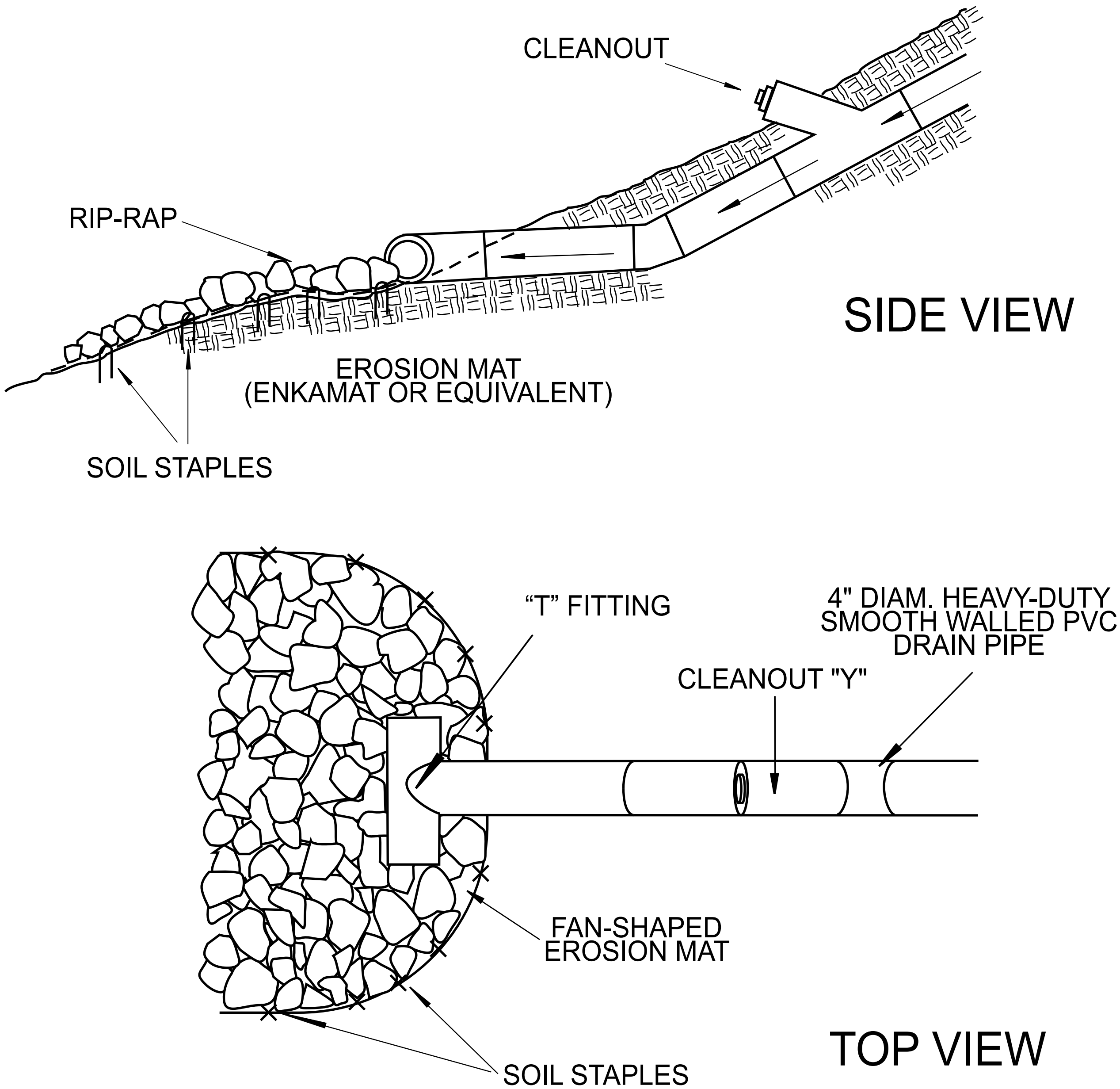
JB/CH	Not Applicable	15068C-01R1	June 2015	Figure 13



CONCEPTUAL ENERGY DISSIPATER DIAGRAM

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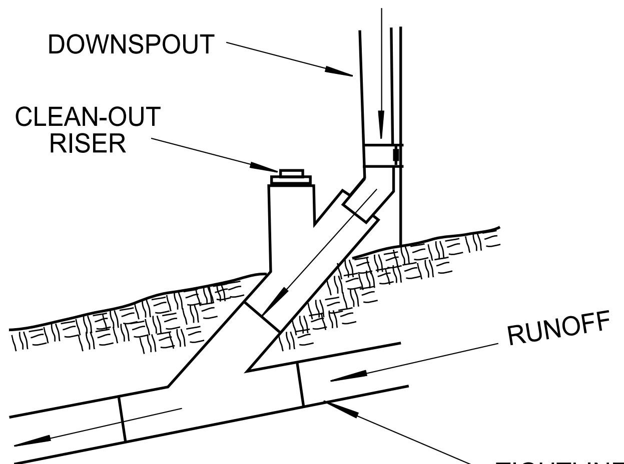
JB/CH	Not Applicable	15068C-01R1	June 2015	Figure 14



CONCEPTUAL DOWNSPOUT CLEAN-OUT DIAGRAM

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DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JB/CH	Not Applicable	15068C-01R1	June 2015	Figure 15

APPENDIX I

BORING LOGS AND LABORATORY TEST DATA
FROM PRIOR STUDIES

Logged By: J.B.	Exploratory Boring Log	Hole No. B-1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unfonf. Comp. Strenght, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "0" Degree	Sample Number	Depth in Feet	Boring Log	Job No. 98-1179-S1 DESCRIPTION
									DARK SANDY SILT WITH LARGE ROCK, STIFF, MOIST
121.3	12.5	56		1.3	22	1-1	5		VERY HARD TO DRILL
							10		
							15		
							20		
									Boring terminated @ 20'

Remarks:

Figure 4 – Logs of Test Borings

ADCO ENGR.

EXPLORATORY BORING LOG

No. B-B-1

PROJECT: **BELLA VISTA DEVELOPMENT** | DATE: 5/25/2005 | LOGGED BY: AMO

DRILL COMPANY: Powered Hand Augur | BORING DIA.: 8 1/4" | BORING ELEV.: ---

GROUNDWATER DEPTH: Not Encountered | SAMPLER: **L=3" O.D.; M=2" O.D.; *= SPT; B=BULK; S=SLOUGH**

NOTES: Sunny at 92 F, Boring at 4 feet from the edge of A/C pavement

DESCRIPTION	USCS SOIL TYPE	DEPTH (feet)	SAMPLE	BLOWS PER FOOT	POCKET PEN. (tsf)	DRY DENSITY (pcf)	WATER CONTENT (%)	FINES (%)	SANDS (%)	GRAVELS (%)	LIQUID LIMIT	PLASTIC LIMIT	DIRECT SHEAR: FRIC. ANG. f (deg.)	DIRECT SHEAR: COHESION, c (ksf)
Sandy Gravel mixed with Top Soil, Roots, and Organic Materials		1												
		2												
		3	B				10.4	8.0	32	40				
Brown Sandy Gravel with fines and cobbles		4												
Shear strength did not drop significantly upon saturation in field.		5	B											
Permeability is estimated to be 10 $^{-3}$ cm/sec (field test)		6												
Boring terminated at 6 feet		7												
		8												
		9												
		10												
		11												
		12												
		13												
		14												
		15												
		16												
		17												
		18												
		19												
		20												
		21												
		22												
		23												
		24												
		25												

PROJECT NO.: BL-01-05 Ali Oskoorouchi, Ph.D., P.E., G.E.

Log of Boring B-1

PAGE 1 OF 1

PROJECT: **Bella Vista Development**
339-341 Bella Vista Avenue, Los Gatos

Boring location: 17 feet west of and 7 feet downslope of Bella Vista Ave.
Logged by: Rick Ford
Date started: 02-20-07 | Date finished: 02-20-07
Drilling method: Minute Man/Portable Rig 4" solid stem/Cathead
Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Manual
Sampler: Sprague & Henwood split-barrel, SPT, CAL 2"

Ground Surface Elevation: 94.0 feet [2]

DEPTH (feet)	Sampler Type	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Gravels %	Sands %	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–4			FILL	GRAVELLY CLAYEY SAND (SC), brown, loose to medium dense, slightly moist, sub-angular to rounded pebbles to cobbles, roots to .25 inches dia. (FILL)					
~3								7.4	109
3–4	S&H	13			34.6	49.5	15.9	6.5	116
4–7			Qc	light yellowish brown, medium dense, dry, (COLLUVIUM)				5.4	
4.5–5.5	SPT	19							
7				Harder drilling					
8	SPT	58	Qpf	SANDY SILTY GRAVEL (GC), light brown and brown mottled, sand matrix, medium dense to dense, dry, clasts consist of reddish-orange to buff weathered sandstone and gray mudstone, sub-angular to rounded pebbles to cobbles, roots to .125 inches dia.(Alluvium)	35.9	45.9	18.2	4.9	
10	SPT	60							
12				sub-rounded pebbles in cuttings					
13–14.5	SPT	53							
18	CAL	60/6"							
19–20	SPT	65							
20	CAL	50/6"			67.3	27.4	5.3	5.9	
21	SPT	94							
22	SPT	87		Refusal in boulder (?)	40.6	48.2	11.2	6.1	

Boring Terminated at 22.5 feet
No ground water encountered at time of drilling
Boring backfilled with soil.

1. Blow counts converted to approximate SPT N-values.
2. Approximate elevation from SMP Engineers, 2006.

PROJECT: **Bella Vista Development**
339-341 Bella Vista Avenue, Los Gatos

Log of Boring B-2

PAGE 1 OF 1

Boring location: App. 58 feet west of and 20 feet downslope of Bella Vista Ave.	Logged by: Rick Ford
Date started: 02-20-07	Date finished: 02-20-07
Drilling method: Minute Man/Portable Rig (no auger) continuous drive	
Hammer weight/drop: 140 lbs./ 30 inches	Hammer type: Manual
Sampler: Sprague & Henwood split-barrel, SPT,	

Ground Surface Elevation: 81.0 feet [2]

DEPTH (feet)	Sampler Type	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Gravels %	Sands %	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–3.5			Qc	GRAVELLY CLAYEY SAND (SC), brown, loose to medium dense, moist, sub-angular to rounded pebbles to cobbles, organic material, roots to .25 inches dia. (COLLUVIUM)					
3					43.9	38.7	17.4	4.4	
3.5–14.5			Qpf	SANDY SILTY GRAVEL (GC), light yellowish brown and brown mottled, medium dense to dense, dry, clasts consist of reddish-orange to buff weathered sandstone and gray mudstone, sub-angular to rounded pebbles to cobbles, (ALLUVIUM)					
5–6.5	S&H	24			36.8	47.3	15.9	5.4	108
6								4.7	107
10–11.5	SPT	63			46.3	43.2	10.5	4.9	
14–14.5	SPT	117		Refusal in boulder (?)					

Boring Terminated at 14.5 feet
No ground water encountered at time of drilling
Boring backfilled with soil.

1. Blow counts converted to approximate SPT N-values.
2. Approximate elevation from SMP Engineers, 2006.

PROJECT: **Bella Vista Development**
339-341 Bella Vista Avenue, Los Gatos

Log of Boring B-3

Boring location: App. 35 feet west of and 10 feet downslope of Bella Vista Ave.

Logged by: Rick Ford

Date started: 02-21-07 | Date finished: 02-21-07

Drilling method: Minute Man/Portable Rig (no auger) continuous drive

Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Manual

Sampler: Sprague & Henwood split-barrel, SPT,

MATERIAL DESCRIPTION — Ground Surface Elevation: 91.0 feet [2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Gravels %	Sand %	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–3			Qc	GRAVELLY CLAYEY SAND/SANDY CLAYEY GRAVEL (SC/GC), brown, loose to medium dense, slightly moist, sub-angular to rounded pebbles to cobbles, organic material, roots to .125 inches dia. (COLLUVIUM)					
3	S&H	37			60.5	31.9	7.6	4.0	
4–6		26	Qpf	SANDY SILTY GRAVEL (GC), light yellowish brown and brown mottled, medium dense to dense, dry, clasts consist of reddish-orange to buff weathered sandstone and gray mudstone, sub-angular to rounded pebbles to cobbles, (ALLUVIUM)	58.7	32.5	8.8	7.3	
8					31.1	49.9	19.0	6.7	
9	SPT	72							
10	SPT	57/6"							
11				Pebbly lense					
14	SPT	80		Refusal in boulder (?)					

Boring Terminated at 14.5 feet
No ground water encountered at time of drilling
Boring backfilled with soil.

1. Blow counts converted to approximate SPT N-values.
2. Approximate elevation from SMP Engineers, 2006.

PROJECT: **Bella Vista Development**
339-341 Bella Vista Avenue, Los Gatos

Log of Boring B-4

Boring location: Gravel path on west side of property (see site plan)

Logged by: Rick Ford

Date started: 02-20-07 | Date finished: 02-20-07

Drilling method: Minute Man/Portable Rig 4" solid stem/Cathead

Hammer weight/drop: 140 lbs./ 30 inches | Hammer type: Manual

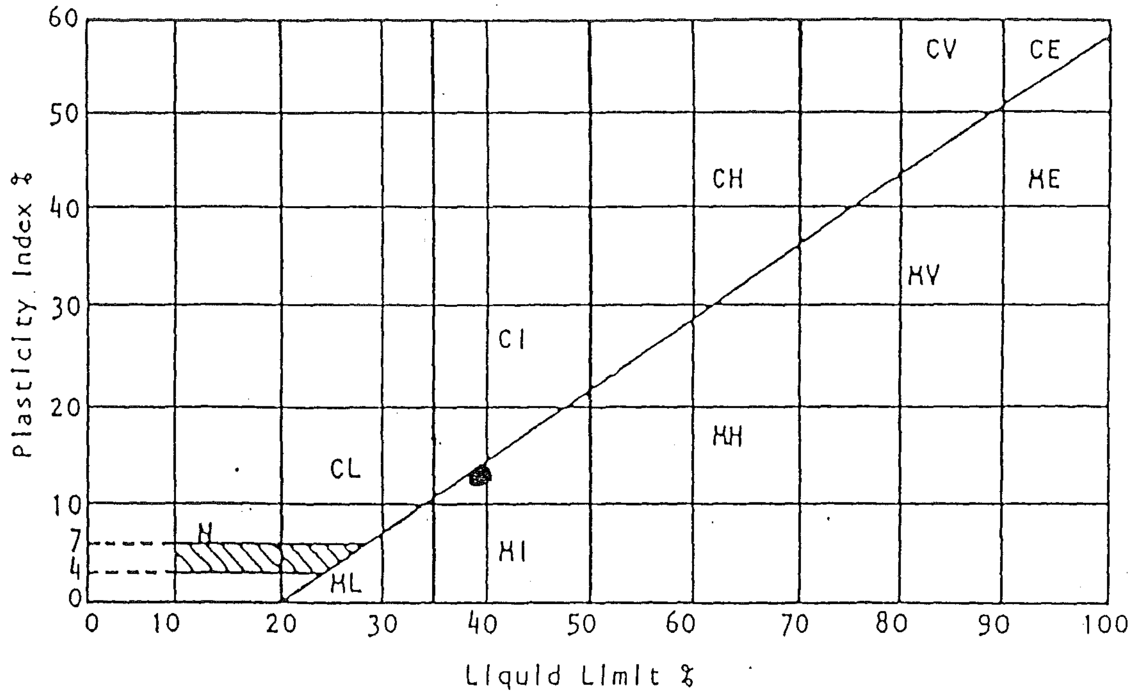
Sampler: Sprague & Henwood split-barrel, SPT,

Ground Surface Elevation: 67.0 feet [2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Gravels %	Sands %	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				Asphalt					
1			FILL	SANDY CLAY (CL), brown with orange mottling, stiff, moist, organic material. (FILL)					
2									
3			Qpf	GRAVELLY CLAYEY SAND / SANDY SILTY GRAVEL (SC/GC), light-brown, brown, reddish-brown mottled, medium dense, moist, clasts consist of reddish-orange to buff weathered sandstone and gray mudstone, sub-angular to rounded pebbles to cobbles, roots to .125 inches dia. (ALLUVIUM)					
4	S&H	25							
5					64.8	30.1	5.1	9.3	
6	SPT	36		more clayey					
7									
8	SPT				49.3	38.7	12.0	7.5	
9		66	Qpf						
10	SPT	44			50.6	34.9	14.5	7.7	
11				pebble lense					
12	SPT	29							
13	SPT	16							
14									
15	SPT	50		sandy					
16	SPT	54/6"							
17	SPT	53/6"		inrease in moisture and cementation					
18									
19	SPT	88			34.8	50.6	14.6	9.5	
20	SPT	115/6"		Refusal in boulder (?)					
21–30				Boring Terminated at 20 feet No ground water encountered at time of drilling Boring backfilled with soil.					

1. Blow counts converted to approximate SPT N-values.
2. Approximate elevation from SMP Engineers, 2006.

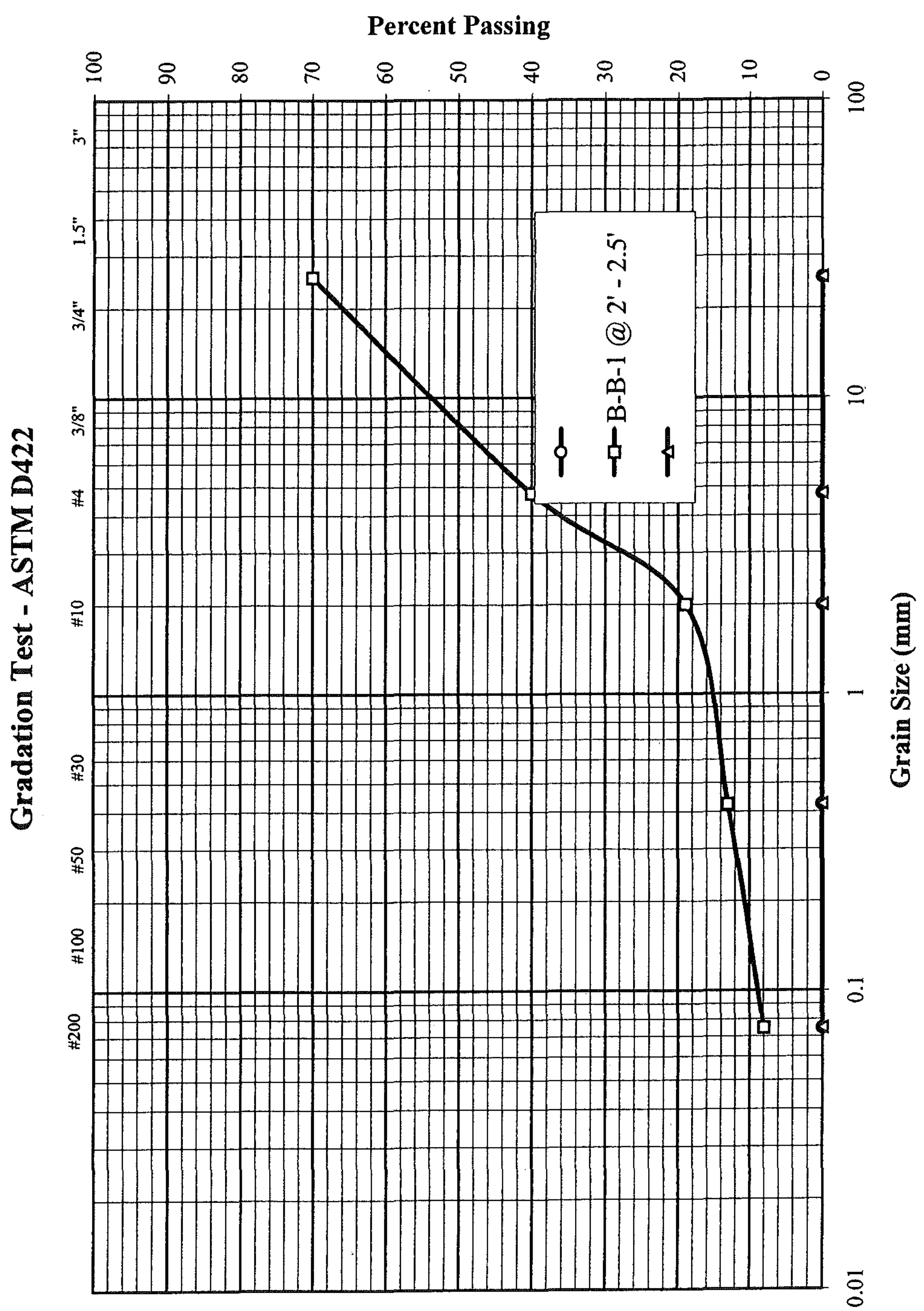
PLASTICITY CHART



PLASTICITY DATA

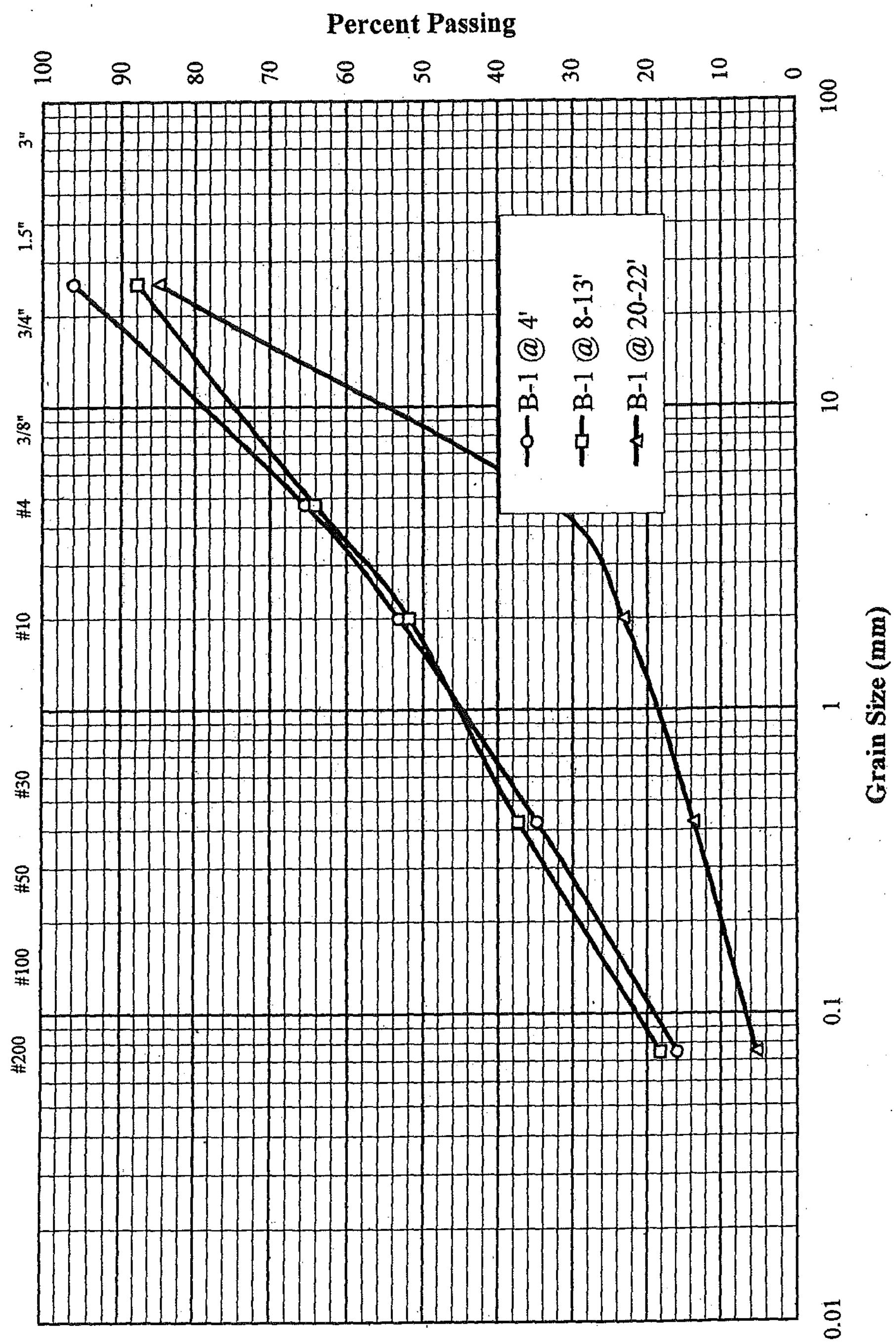
Key Symbol	Hole No.	Depth Ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol
●	Bag Sample	1-2	38	14	MI

Figure 5 Pl



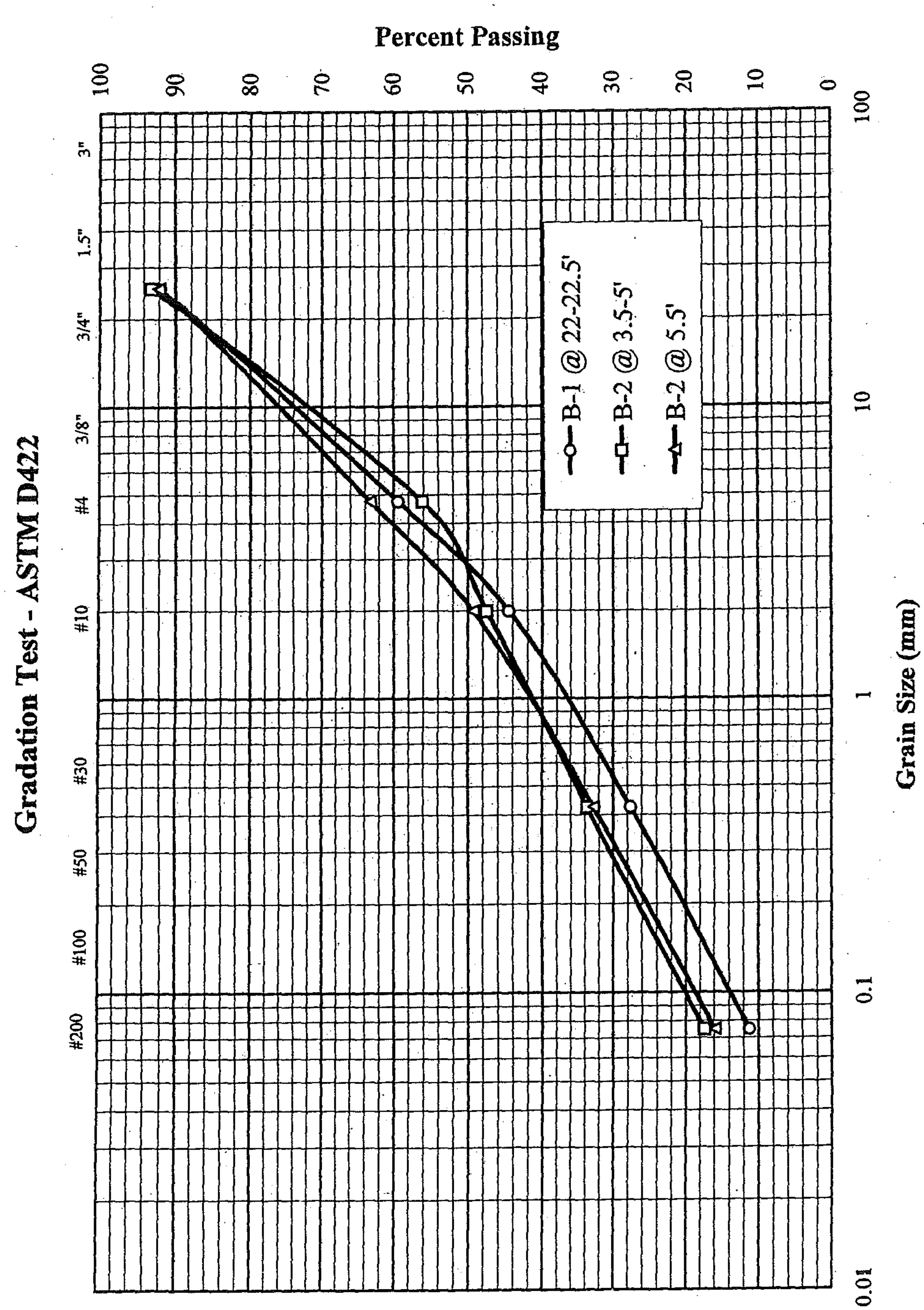
Ali M. Oskoorouchi, Ph.D., P.E., G.E. P.O. Box 3494 Freedom, California 95019	**Gradation Test ASTM D1140** Bella Vista Development Los Gatos, California	Figure No. 4 Project No. BL_01-05 Date 06/28/2005

Gradation Test - ASTM D422



Note: Some gravel cobbles and boulders are broken during drilling and it impacted the grain size distribution

Ali M. Oskoorouchi, Ph.D., P.E., G.E. P.O. Box 66245 Scotts Valley, California 95067	**Gradation Test ASTM D1140** Bella Vista Development Los Gatos, California	Figure No. 4_1 Project No. BL-01-07 Date 03/10/2007



Note: Some gravel cobbles and boulders are broken during drilling and it impacted the grain size distribution

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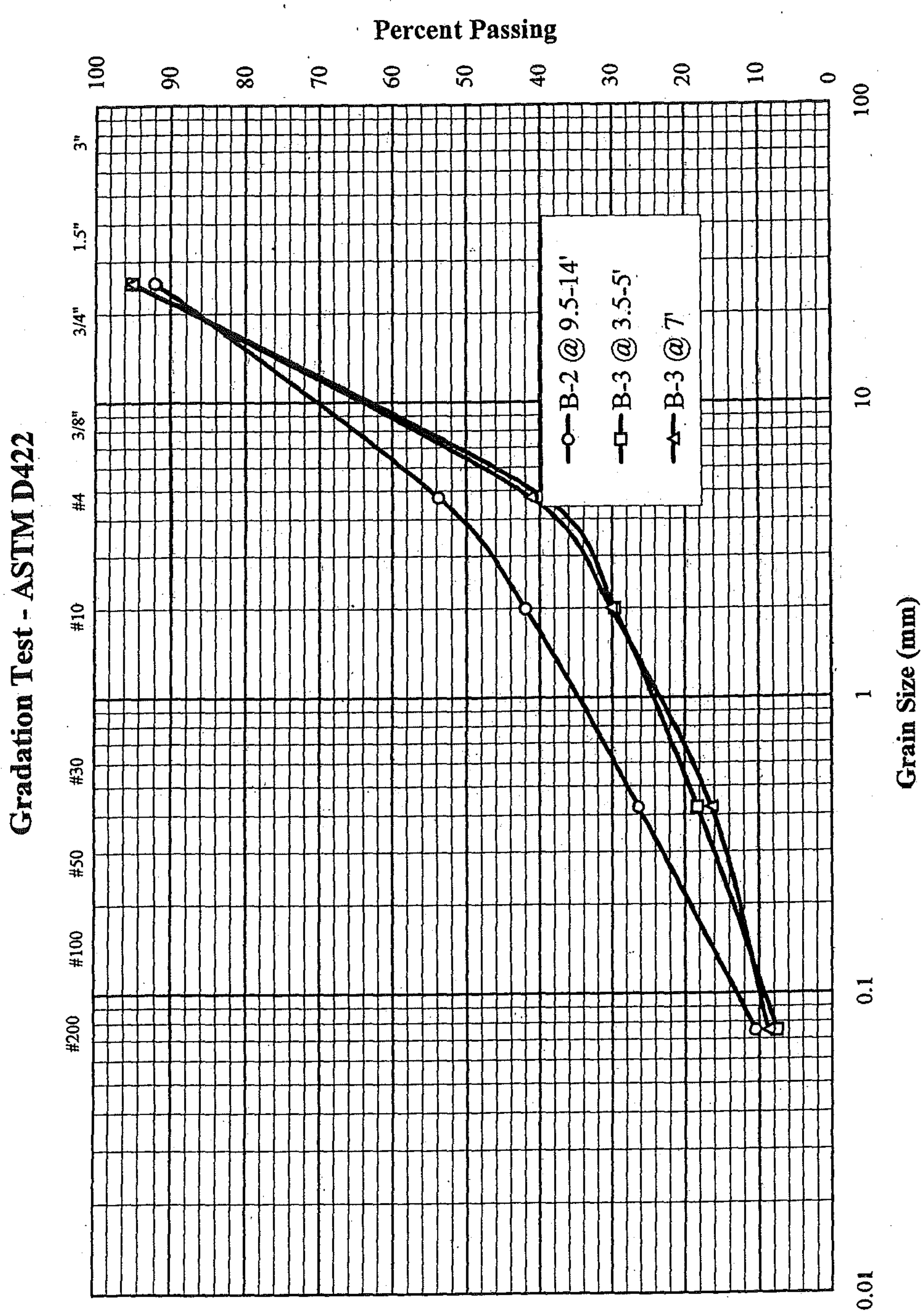

Note: Some gravel cobbles and boulders are broken during drilling and it impacted the grain size distribution

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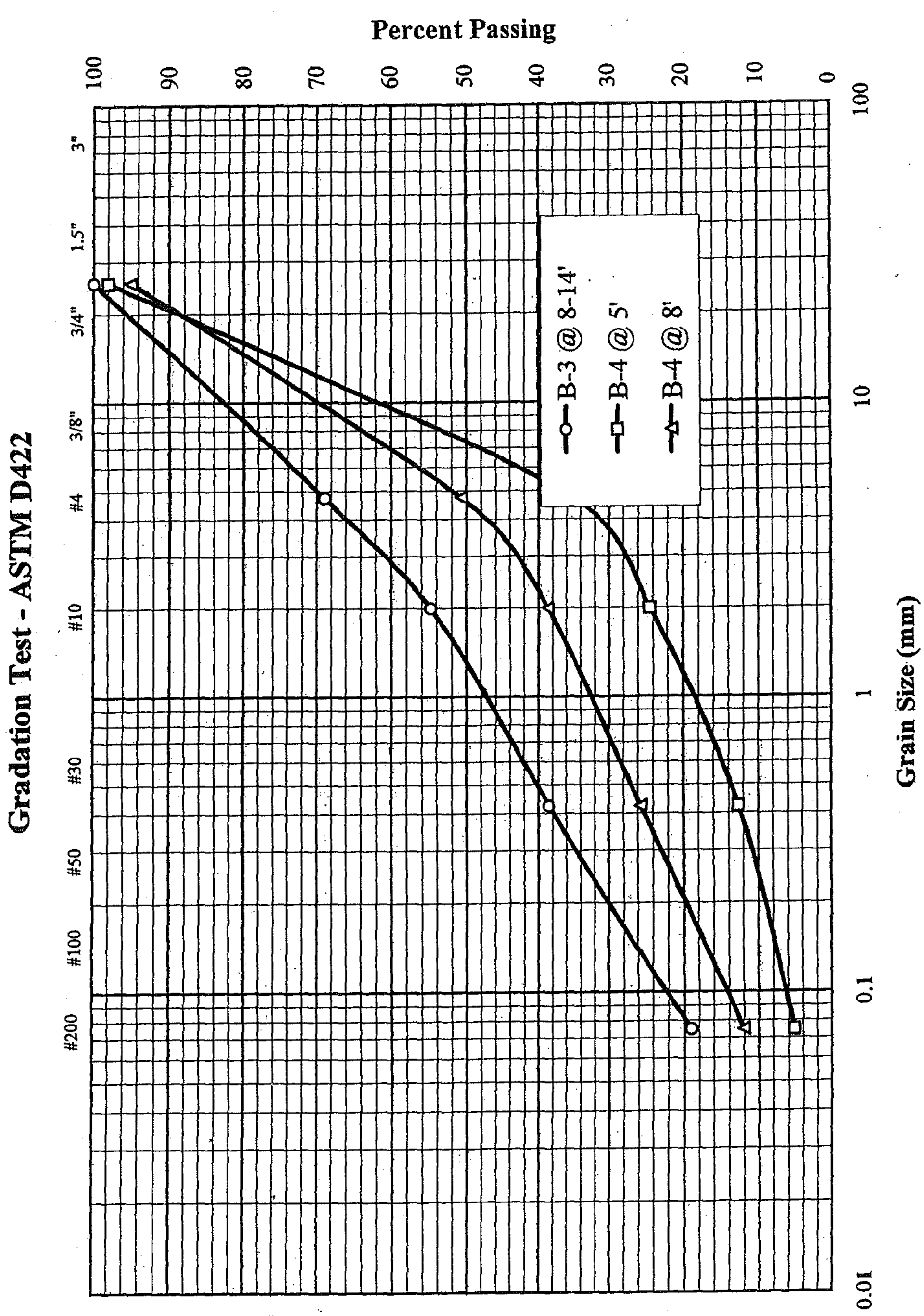
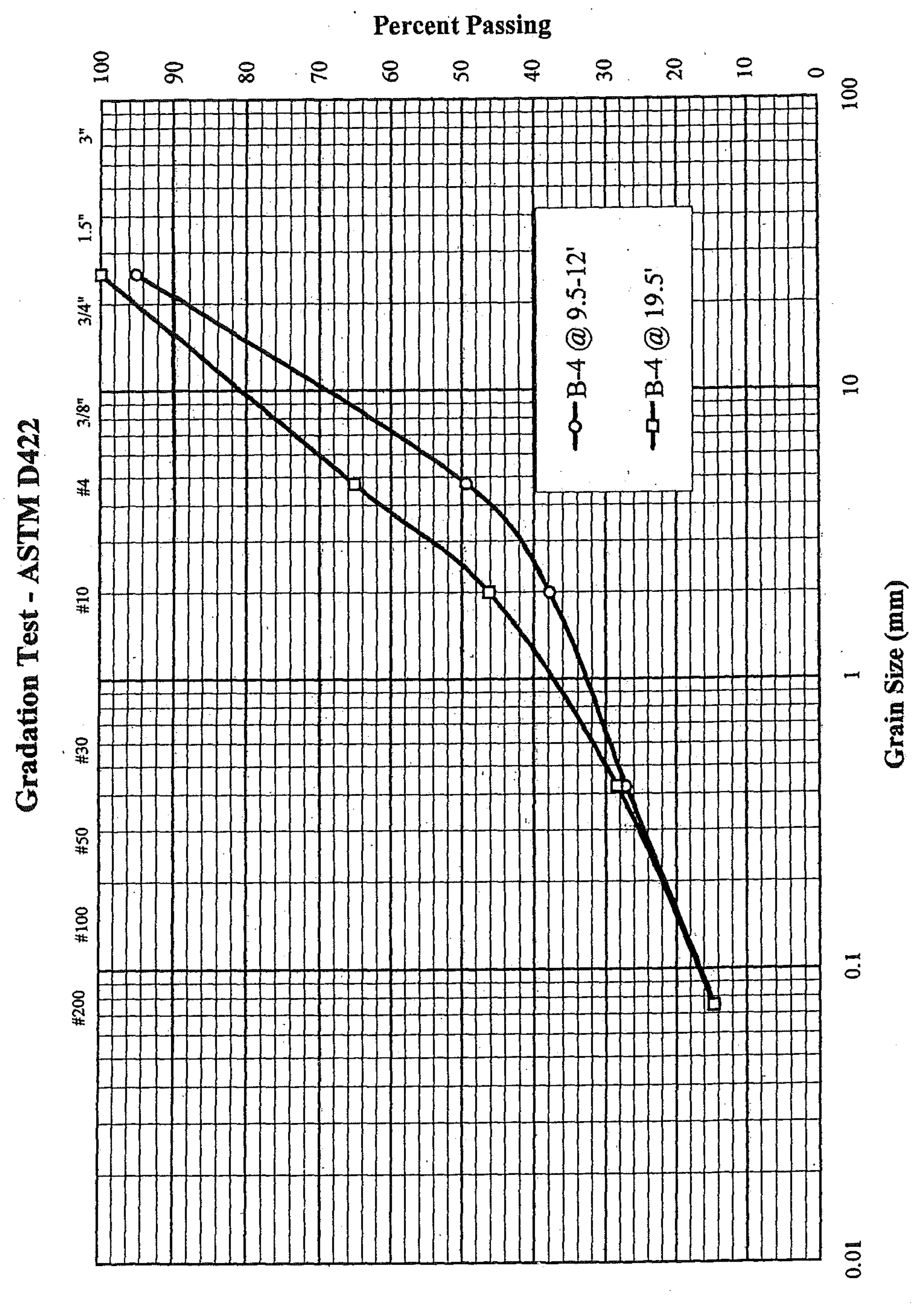


Note: Some gravel cobbles and boulders are broken during drilling and it impacted the grain size distribution

Ali M. Oskoorouchi, Ph.D., P.E., G.E. **P.O. Box 66245** **Scotts Valley, California 95067**	**Gradation Test ASTM D1140** Bella Vista Development Los Gatos, California	Figure No. 4_4 Project No. BL-01-07 Date 03/10/2007



Note: Some gravel cobbles and boulders are broken during drilling and it impacted the grain size distribution

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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.

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