APPENDIX E

Geotechnical Study

Stevens Ferrone & Bailey
Engineering Company, Inc

Geotechnical Engineering
Engineering Geology
Storm Water Management
Construction Observation & Testing Services

November 20, 2014

GEOTECHNICAL FEASIBILITY INVESTIGATION
AND FAULT RUPTURE HAZARD ASSESSMENT
22330 MAIN STREET
HAYWARD, CALIFORNIA
SFB PROJECT NO. 648-1

Prepared For:

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1.0 INTRODUCTION

This report presents the results of our geotechnical feasibility investigation and fault rupture hazard assessment for the proposed development project at 22330 Main Street in Hayward, California. The approximate site location is shown on the Vicinity Map, Figure 1.

As shown on Figures 2 and 3, the site is located immediately south of the San Lorenzo Creek along the west side of the San Francisco East Bay Hills. We understand that the preliminary development plan consists of a four-story residential structure, three-story parking garage, a 27,000 square foot grocery store, a 7,000 square foot commercial building, and renovating an existing office building with associated parking lot. Except for the office building and parking lot, existing buildings and associated facilities will be demolished prior to new construction. The conclusions and recommendations provided in this report are based upon the information provided herein; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

As shown on Figure 4, the approximately southwestern half of the site is located within a State of California, Alquist-Priolo Earthquake Fault Zone as delineated by the California Geological Survey (CGS) for the Hayward fault. The Hayward fault shows evidence of historic ground rupture and on-going fault creep. This fault zone is based on the information and conclusions contained in the CGS Fault Evaluation Report FER-103 (Hart, 1981). CGS Note 49 (2002), *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, was followed during our fault rupture hazard evaluation.

Figure 4 also shows that the approximately eastern half of the site is located within a seismic hazards zone. This zone was established to delineate areas where historical occurrence of liquefaction has occurred, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation measures would be required.

References used during our investigation and during the preparation of this report are listed in the *References* section of this report.

2.0 SCOPE OF WORK

This geotechnical feasibility investigation and fault rupture hazard assessment included the following scope of work:

- Reviewing available published and unpublished geotechnical and geological literature, reports, and maps relevant to the site and surrounding area, including previous Alquist-Priolo Fault Reports submitted to the CGS for review;
- Reviewing stereoscopic aerial photographs of the site and surrounding area;
- Performing geotechnical and geologic reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program, including drilling two exploratory borings to a maximum depth of about 47 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing geological and geotechnical engineering analysis of the research, field, and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the primary purpose of providing preliminary geotechnical criteria for the planning and cost estimating of the project, and to provide our opinions regarding the fault rupture hazard and liquefaction potential at the site.

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed in October and November, 2014. Subsurface exploration was performed using a truck-mounted drill rig equipped with 6-inch diameter, continuous flight, solid stem augers. Two exploratory borings were drilled onsite on November 12, 2014, to a maximum depth of about 47 feet. The approximate locations of SFB's borings are shown on the Site Plan, Figure 6. Logs of SFB's borings and details regarding SFB's field investigation are included in Appendix A. The results of SFB's laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Surface Description

At the time of our investigation, the site was being used as an office complex with associated paved parking, paved access ways, and walkways. Several single- and multi-story buildings occupied the site. Some of the buildings had basements. Landscaped planter areas were also observed. Figure 6 shows the approximate locations of the various existing improvements.

3.2 Geologic and Tectonic Setting

The site is contained within the Coast Range Geomorphic Province of California, an area characterized by a series of northwestern trending ridges and valleys, dominated by the San Andreas fault system. The San Andreas fault system trends northwestward through the San Francisco Bay Area. Movement along the San Andreas fault system is distributed among several active, right-lateral faults that generally parallel the main trace of the San Andreas fault. In the East Bay, the dominant fault is the Hayward fault, which is mapped from San Pablo on the north to eastern San Jose on the south. To the east of the site, the East Bay Hills have formed from a compressional interaction between the Calaveras fault on the east side of the hills and the Hayward fault on the west side of the hills. The San Andreas fault is located approximately 18 miles southwest of the site.

As shown on Figure 2, the site is in an alluvial floodplain at the mouth of the San Lorenzo Creek which drains westward toward San Francisco Bay. Our borings indicate that the entire site is underlain by alluvial deposits. As shown on Figure 3, Holocene and Pleistocene terrace deposits have formed distinct terraces; these terraces are typically composed of well consolidated clays, silts, sands, and gravels. Figure 3 shows that bedrock in the vicinity of the site has been mapped as being part of the Franciscan Complex, a diverse group of igneous, sedimentary, and metamorphic rocks of Upper Jurassic to Cretaceous age, which are found along the eastern side of the San Andreas fault system.

The California Geological Survey (CGS) zoned the Hayward fault under the provisions of the Alquist-Priolo Special Studies Zone Act of 1972. Figure 4 shows the approximate location of the zone for the Hayward fault in the vicinity of the site. Two relatively recent, large earthquakes have been reported to have been caused by movement along the Hayward fault. In 1836, an earthquake with a roughly estimated magnitude of 6.8 is presumed to have been centered on the Hayward fault and caused ground rupture between Mission San Jose and San Pablo. In 1868, a large earthquake having an estimate magnitude of 6.8 to 7.0 occurred along the Hayward fault and caused severe damage in downtown Hayward. In 1868, ground rupture occurred along the fault in Hayward; ground rupture was documented close to the site as shown on Figure 4 (location shown on Figure 4 with the designation “1868”).

The Hayward fault is also characterized by active surface fault creep. Offsets and cracking of surficial improvement, such as curbs, gutters, roads, and walls, can be found in several areas along the main trace of the Hayward fault. Figure 4 shows solid fault lines with the designation “C” which indicate where fault creep has been observed. Creep monitoring stations have been used by the U.S. Geological survey for many years. Lienkaemper (2006) shows recently active traces of the Hayward fault on his maps, with creep data being a primary source of information.

We have observed surface fault creep in similar locations as delineated by Lienkaemper (2006). We did not observe any fault related features onsite or surrounding the site.

3.3 Stereoscopic, Aerial Photo Reviews

The Hayward fault in the area of the site exhibits geomorphic features characteristic of Holocene strike-slip movement, such as offset drainages, linear troughs, linear scarps, and closed depressions. Tonal lineaments are also commonly seen throughout the vicinity of the site. One of the most prominent fault related features in the vicinity of the site can be seen in recent aerial photos. As shown on Figures 2 and 3, the path of San Lorenzo Creek appears to have been offset right-laterally by the Hayward fault by at least 5,000 feet. Also in the vicinity of the site, lineaments are observed along the trend (both west and east sides) of Prospect Hill located to the west and northwest of the site, trends that parallel the recently active traces of the Hayward fault.

Burkland & Associates (1975) performed a very detailed study of the Hayward fault at that time for the City of Hayward. The Burkland study area is shown on Figure 5 as the site outlined as AP2820; and area which includes the project site. In their report, *Geological and Geophysical Investigation in Downtown Hayward*, dated May 8, 1975, Burkland & Associates compiled the published results of aerial photo analyses of the Hayward fault dating back to 1956. In their report, Burkland (1975) shows that tonal lineaments have been previously mapped crossing the site but Burkland shows that these lineaments are attributed to geologic contacts between younger alluvium (sediments related to infilling of a previous topographic depression, i.e., lake sediments) and older alluvium.

Earth Systems Consultants (ESC) performed an updated geological study for the City of Hayward and published their results in a report dated February 7, 1992. ESC studied photos from 1939, 1947, 1971, 1972, and 1990 and were able to map features associated with active faulting. The features are described in their report; none of the ESC mapped features indicate surface faulting traversing the project site.

Our aerial photo review of the 1939 photos did not reveal any additional features that have not already been described in the reports described above or in the CGS report FER-103. Most of the downtown area is already obscured by pavement and structures in the 1939 photos.

3.4 Previous Fault Location Studies Performed in Site Vicinity

Numerous fault location studies have been performed in the vicinity of the site. As part of most of the investigations, trenches were excavated across potential locations of fault traces. In order to better understand where the fault location studies were performed, we prepared a compilation map (attached as Figure 5) titled *Regional Fault Study Map*. Included on the map is the location of the site and the portion of the site within the AP Earthquake Fault Zone, locations of the Hayward fault traces shown on the State of California Alquist-Priolo Earthquake Fault Zone Map for the Hayward Quadrangle (2012), and the approximate locations of sites where fault location studies were performed. These fault location studies resulted in publicly available reports (AP reports) that are on file with the California Geological Survey. The AP reports used in compiling the map shown on Figure 5 are referenced on the map and are listed in the *References* section of this report. Shown on the map are the approximate locations of excavated trenches, seismic traverses, and borings. Also shown on the map are the approximate locations (yellow lines) where site specific AP fault investigations reportedly encountered an active fault trace.

In review of the AP sites shown on Figure 5, none of the trench logs showed fault traces located beyond the fault traces shown on the 2012 AP Fault Zone map except for a fault trace reported on a trench log contained in reports AP2589 (located immediately south of Foothill Boulevard) and AP270 (located at the intersection of Prospect Street and Hotel Avenue, west of the site). Trenches performed immediately to the northwest and southeast (parallel to the recently active Hayward fault traces) of the portion of the subject property located in the AP fault zone did not encounter any active fault traces. In summary, the only active fault traces reported in the available documents are located to the west of Main Street between Sunset Boulevard on the north and E Street on the south. The nearest reportedly active fault trace is located approximately 225 feet southwest of the nearest site boundary (see report AP270 for additional details).

3.5 Subsurface Conditions

The near-surface soil materials encountered by our borings at the site (below existing pavements) generally consisted of firm to stiff clayey fills extending to depths of about 2 feet, and interbedded native stiff to hard silty clays, medium dense sands, and medium dense to dense gravels that extended to depth of about 47 feet. According to the results of our laboratory testing, the near-surface more clayey fills and soils have a low plasticity and low expansion potential.

Detailed descriptions of the materials encountered in our exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using pacing or landmark references and should be considered accurate only to the degree implied by the method used.

3.6 Groundwater

Groundwater was initially encountered in our borings at depths of about 25 to 27 feet and rose to depths of about 22 and 23 feet at the end of drilling. SFB's borings was backfilled with lean cement grout in accordance with Alameda County Water District requirements prior to leaving the site. Historically, groundwater in the vicinity of the site has been measured at depths of about 15 feet[1]. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, and other factors.

3.7 Liquefaction & Lateral Spreading

Soil liquefaction is a phenomenon primarily associated with saturated, cohesionless, soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface. According to ABAG and the U.S. Geological Survey, the site is located in an area mapped as having a likelihood of liquefaction in an earthquake and has been characterized as

[1]State of California, 2013, *Seismic Hazard Zone Report for the Hayward 7.5-Minute Quadrangle*.

having liquefaction susceptibility[2,3]. According to the Seismic Hazard Zones Map of the Hayward Quadrangle (see Figure 4), part of the site is located in a seismic hazard zone due to liquefaction as designated by the State of California. As shown on Figure 7, liquefaction related ground damage has been historically reported in the vicinity of the site.

SFB performed SPT-based liquefaction analyses using procedures described by the Southern California Earthquake Center (SCEC, Martin and Lew, 1999) and research papers by Seed (2001)[4]. A peak ground acceleration having a 10% probability of being exceeded in a 50-year period (mean return time of 475 years) was used in our analyses (this resulted in an onsite peak ground acceleration of 0.7g, a design basis ground motion based on stiff soil site condition). Groundwater levels measured in the borings and the historically measured groundwater levels were used in our analyses to assess their impacts on liquefaction and ground surface damage potential.

The results of our analyses indicate that the saturated sands and medium dense gravels encountered by the onsite borings have a high potential for liquefying when subjected to a design basis earthquake event. The earthquake induced liquefaction in these sand or gravels could result in residual volumetric strains varying from about 1.6% to 2.9%. We estimate that the liquefaction of these soils if subjected to a design basis earthquake event may cause total aerial ground surface settlements of about 3 to 4 inches when using historically measured groundwater levels, with differential settlements of about 1-1/2 to 2 inches between typical building columns (distances of about 30 feet; recommended differential settlement estimates per SCEC, Martin and Lew, 1999). The actual ground surface damage will vary depending on the thickness of the overlying non-liquefiable soils and the underlying liquefiable soils[5].

To reduce the liquefaction effects on the overlying super-structure, we recommend the building foundations be designed to resist 2 inches of differential settlement of the supporting soils. This magnitude of differential settlement could occur directly below foundation supporting column loads. Similarly, settlement could occur below foundation slabs (at a distance of about 30 feet), creating a "cupping" shape of the underlying supporting subgrade.

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[2]Association of Bay Area Governments, 1980, *Liquefaction Susceptibility, San Francisco Bay Region.*
[3]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.
[4]Seed et al., 2001, *Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation, Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn, San Diego, California.*
[5]Ishihara, K., 1985, *Stability of Natural Deposits During Earthquakes*, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA Volume 1, p. 321-376, August.

In addition, underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as roadways and utilities may be observed.

As part of our analyses, we evaluated the potential for lateral spreading impacting the site. Lateral spreading occurs when soils liquefy during an earthquake event and the liquefied soils with the overlying soils move laterally to unconfined spaces (for example, the drainage channel banks), which causes significant horizontal ground displacements. Based on our review of available literature, the results of the field exploration, and results of our liquefaction analyses, it is our opinion that the potential for lateral spreading toward San Lorenzo Creek severely impacting the site development is low due to the depth of the creek, the depth of the liquefiable soils, and the distance of the site to the creek.

4.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed project from a geological and geotechnical engineering standpoint. The following are the primary geologic and geotechnical considerations for development of the site.

SURFACE FAULTING: As described previously, we did not uncover evidence that an active earthquake fault extending across the surface of the subject site. The only active fault traces reported in the documents reviewed for this study are located west of Main Street, between Sunset Boulevard on the north and E Street on the south. The nearest reportedly active fault trace is located approximately 225 feet southwest of the nearest site boundary; this trace was reported by an independent consulting geologist (see report AP270). As shown on Figures 4 and 5, the nearest active fault traces shown within the California Geological Survey's AP Fault Zone for the Hayward fault are located approximately 350 feet southwest of the site.

LIQUEFACTION: As described in Section 3.7 of this report, the saturated sands and gravels encountered in our borings have a high potential for liquefying when subjected to a design basis earthquake shaking event. We estimate that the liquefaction of these soils if subjected to a design basis earthquake event may cause total aerial ground surface settlements of about 3 to 4 inches when using historically measured groundwater levels, with differential settlements of about 1-1/2 to 2 inches between typical building columns (distances of about 30 feet; recommended differential settlement estimates per SCEC, Martin and Lew, 1999). This magnitude of settlement could also occur directly below the center of a building's mat slab foundation (or at a distance of about 30 feet), creating a "cupping" shape of the underlying supporting subgrade. In addition, underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as driveways, roadways, and utilities can occur and may require repair.

EXISTING FILL MATERIALS: As described previously, old fill materials were encountered by the borings and extended to depths of about 2 feet. Deeper fills may exist elsewhere onsite. These fills are heterogeneous, and potentially weak and compressible. In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend that these fills be completely removed and re-compacted. The over-excavation should extend to depths where competent soil is encountered. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. Where over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The removed fill materials can be used as new fill provided it is placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

ADDITIONAL RECOMMENDATIONS: Additional borings, laboratory testing, and geotechnical engineering analyses will need to be performed in order to provide detailed geotechnical design and construction criteria for the project and to confirm the preliminary recommendations provided below. The future report would include detailed drainage, earthwork, foundation, and pavement recommendations for use in the design and construction of the project. Once the future, detailed investigation is complete, we recommend SFB review the project's design and specifications to verify that the recommendations presented in the future report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications and are not retained during construction.

4.1 Preliminary Earthwork Recommendations

The site will need to be cleared of all obstructions including designated structures and their entire foundation systems, basements, fill materials, existing utilities and pipelines and their associated backfill, existing pavement, designated trees and their associated entire root systems, and debris. Wells and septic systems, if any, should be abandoned in accordance with Alameda County Environmental Health standards. From a geotechnical standpoint, any existing fill materials, backfill, clay and concrete pipes, pavements, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill.

After the completion of clearing, site preparation, and fill re-compaction, soil exposed in areas to receive improvements (such as structural fill, building foundations, driveways, exterior flatwork, and pavements) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 3 to 5 percent over optimum water content, and compacted to the requirements for structural fill.

From a geotechnical and mechanical standpoint, onsite soils and fills having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. Larger sized rock may be used as fill onsite provided it is closely monitored, placed properly to achieve compaction, and are located at depths below anticipated, future excavations; SFB should be consulted regarding the use of larger rock pieces in fill materials. If required, imported fill should have a plasticity index of 20 or less and have a significant amount of cohesive fines.

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted to at least 90 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 percent relative compaction, as determined by ASTM D1557 (latest edition). The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction.

We recommend that exterior slabs (including patios, sidewalks, and driveways) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 to 5 percent above laboratory optimum moisture (ASTM D-1557).

4.2 Preliminary Foundation Recommendations

Due to the high potential for liquefaction induced ground settlements, we recommend stiffened mat foundation slabs and/or post-tensioned slab foundations be used to support the structures. The foundations should be designed to resist the anticipated differential settlements. The slab foundations should bear entirely on properly prepared, compacted structural fill. The actual thickness of the slabs and reinforcement should be determined by a Structural Engineer.

A vapor retarder must be placed between the subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. The concrete mix design for the slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete.

4.3 Preliminary Pavement Recommendations

Based on the soil types encountered in our borings, we anticipate that flexible pavement sectinos would range from about 3 inches of asphalt concrete over 8 to 12 inches of baserock. Actual R-value testing of subgrade soils would need to be performed to determine actual pavement thicknesses. Governing agencies, however, may require thicker pavement sections. We also anticipate that concrete slabs for trash enclosures would likely consist of 6 inches of concrete overlying 6 inches of Caltrans Class 2 aggregate baserock.

5.0 CONDITIONS AND LIMITATIONS

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

We recommend SFB be retained to provide geotechnical and geological services during future investigations, designs, reviews, earthwork operations, paving operations, and foundation installation to confirm and observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered or if changes to the scope of the project, as defined in this report, are made.

This report is a document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of Bay Area Property Developers and their consultants for specific application to the proposed 22330 Main Street development project in Hayward, California, and is intended to represent our findings to Bay Area Property Developers for specific application to the 22330 Main Street project. The conclusions contained in this report are solely professional opinions. We are not responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to provide future investigations, review geological and geotechnical aspects of the construction calculations, specifications, and plans; we should also be retained to participate in prebid and preconstruction conferences to clarify the opinions, conclusions, and recommendations contained in this report and future reports.

It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore we should be consulted if it is not completely understood what the limitations to using this report are.

In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing. The opinions, conclusions, and recommendations contained in this report are based upon the description of the project as presented in the introduction section of this report.

This report does not necessarily represent all of the information that has been communicated by us to Bay Area Property Developers and their consultants during the course of this engagement and our rendering of professional services Bay Area Property Developers. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of Bay Area Property Developers to divulge information that may have been communicated to Bay Area Property Developers. We cannot accept consequences for use of segregated portions of this report.

Please refer to Appendix C for additional guidelines regarding use of this report.

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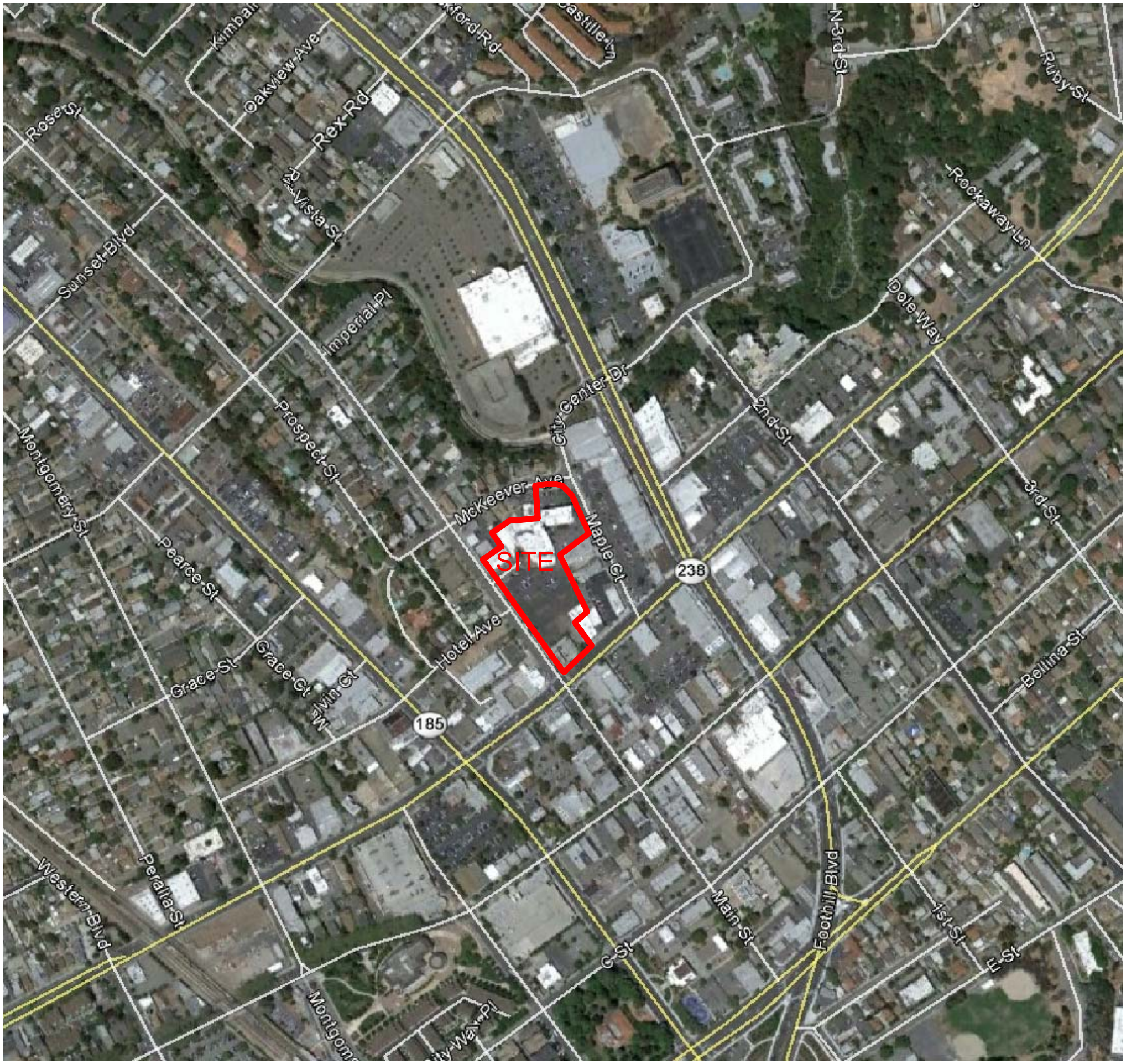
FIGURES



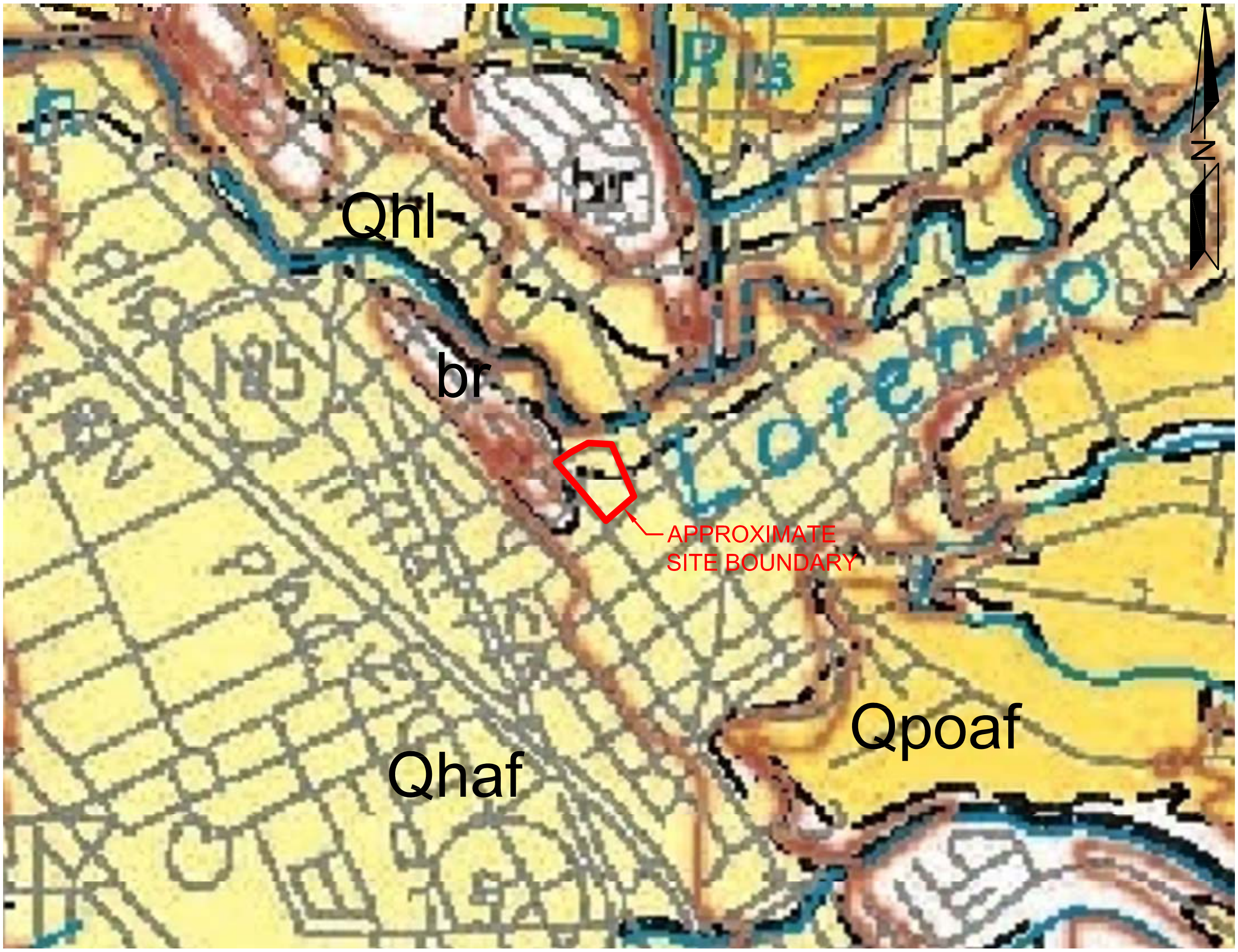
N

APPROXIMATE SCALE: 1" = 600'

0 600' 1200'

NOTE: Base map was taken from Google Earth photograph date 6/9/14.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	VICINITY MAP	FIGURE
November 2014			22330 MAIN STREET Hayward, California	1
PROJECT NO.				
648-1				



Qhl Natural Levee Deposits (Holocene)--Loose, moderately to well-sorted sandy or clayey silt grading to sandy or silty clay. These deposits are porous and permeable and provide conduits for transport of ground water. Levee deposits border stream channels, usually both banks, and slope away to flatter floodplains and basins.

Qhaf Alluvial Fan and Fluvial Deposits (Holocene)--Alluvial fan deposits are brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward, to sandy or silty clay. Near the distal fan edges, the fluvial deposits are typically brown, never reddish, medium dense sand that fines upward to sandy or silty clay.

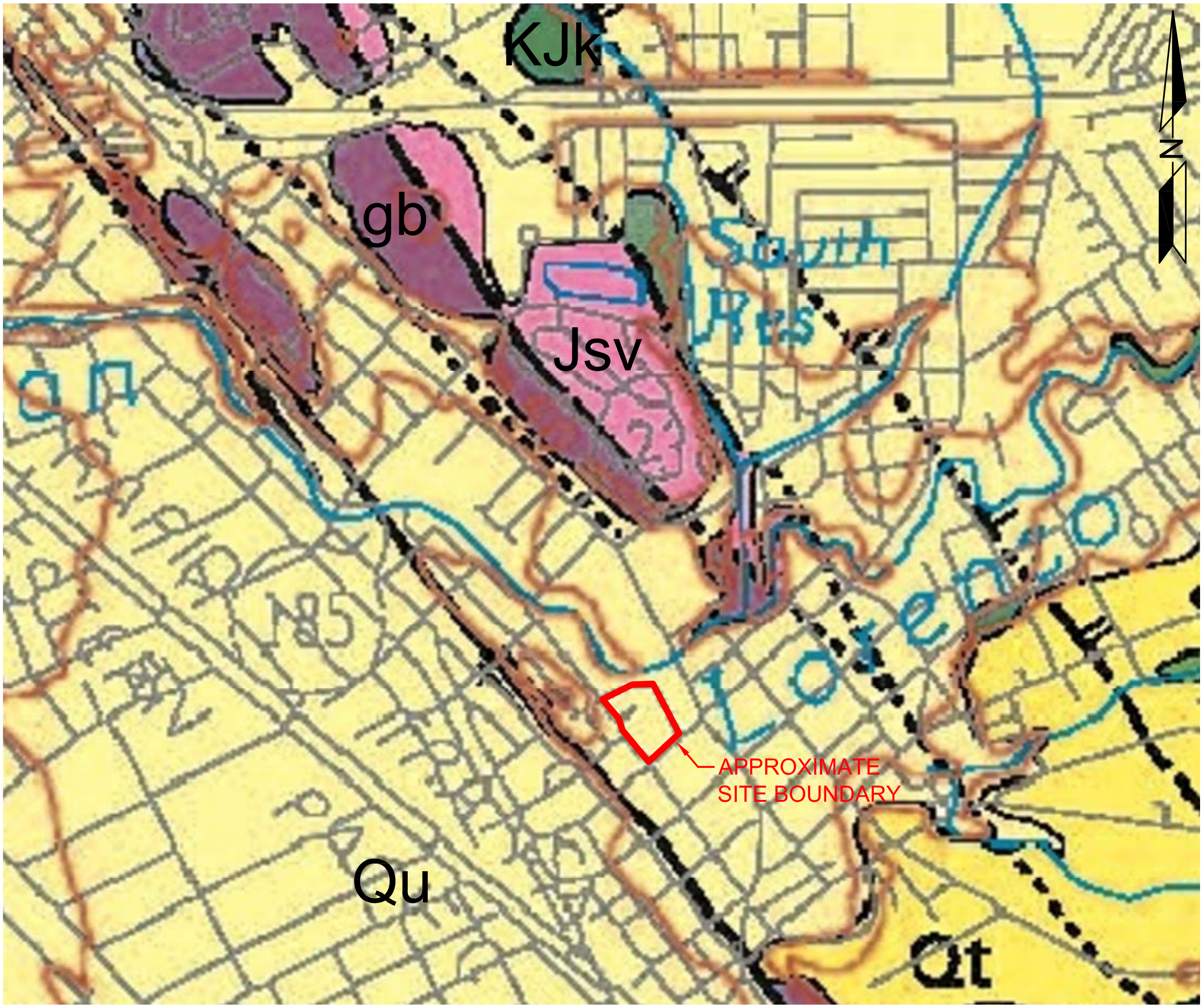
Qpoaf Older Alluvial Fan deposits (Pleistocene) -- Brown dense gravely and clayey sand or clayey gravel that fines upward to sandy clay. These deposits display various sorting qualities. All Qpoaf deposits can be related to modern stream courses. They are distinguished from younger alluvial fans and fluvial deposits by higher topographic position, greater degree of dissection, and stronger profile development. They are less permeable than younger deposits, and locally contain fresh water mollusks and extinct Pleistocene vertebrate fossils.

Br Bedrock

NOTE: Base map was taken from "Quaternary Geology Of Alameda County and Surrounding Areas, California", dated 1997.

APPROXIMATE SCALE: (FEET)
0 1,500' 3,000'

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	QUATERNARY GEOLOGY MAP	FIGURE
November 2014	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	22330 MAIN STREET Hayward, California	2
PROJECT NO.			
648-1			



Qu Surficial deposits, undivided (Holocene and Pleistocene).

Qt Terrace deposits (Holocene(?) and Pleistocene). Clay, silt, sand, gravel, and cobbles forming geomorphically distinct terraces. This unit is only differentiated in a few places.

KJk Knoxville Formation (Late Jurassic and Early Cretaceous). Mainly dark, greenish-gray silt or clay shale with thin sandstone interbeds. Locally includes thick pebble to cobble conglomerate beds in its lower part (KJkc). Locally at the base the formation contains beds of angular, volcanoclastic breccia (KJkv) derived from underlying ophiolite and silicic volcanic rocks. The depositional contact of Knoxville Formation on ophiolite and silicic volcanic rocks can be observed at several locations in Alameda County.

Jsv Keratophyre and quartz keratophyre (Late Jurassic). Highly altered intermediate and silicic volcanic and hypabyssal rocks. Feldspars are almost all replaced by albite. In some places, closely associated with (intruded into?) basalt.

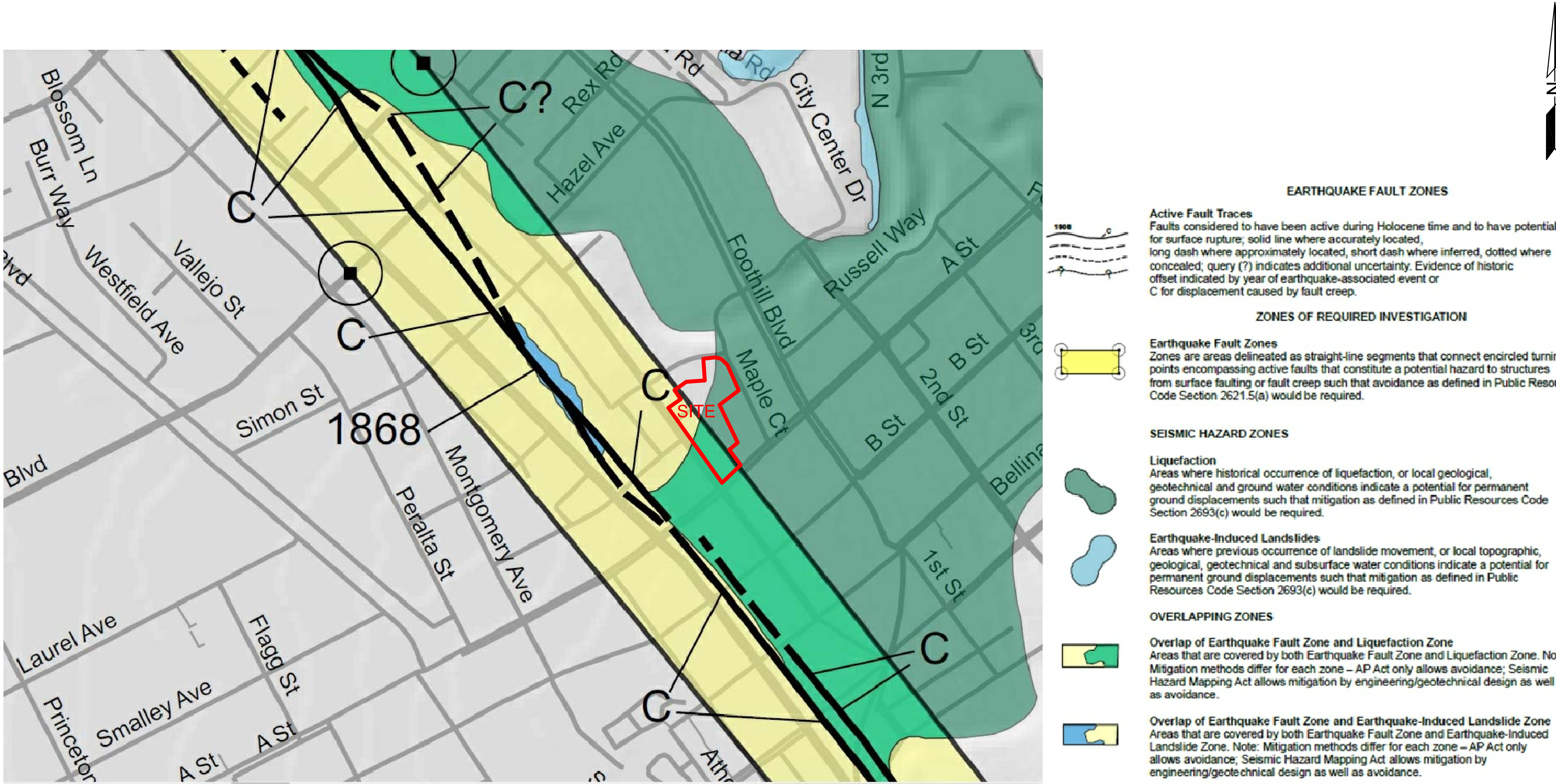
gb Gabbro

APPROXIMATE SCALE: (FEET)

0 1,500' 3,000'

NOTE: Base map was taken from "Preliminary Geologic Map Emphasizing Bedrock Formations in Alameda County, California", dated 1996.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	BEDROCK GEOLOGY MAP	FIGURE
November 2014			22330 MAIN STREET Hayward, California	3
PROJECT NO.				
648-1				



EARTHQUAKE FAULT ZONES

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

ZONES OF REQUIRED INVESTIGATION

Earthquake Fault Zones
Zones are areas delineated as straight-line segments that connect encircled turning points encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as defined in Public Resources Code Section 2621.5(a) would be required.

SEISMIC HAZARD ZONES

Liquefaction
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

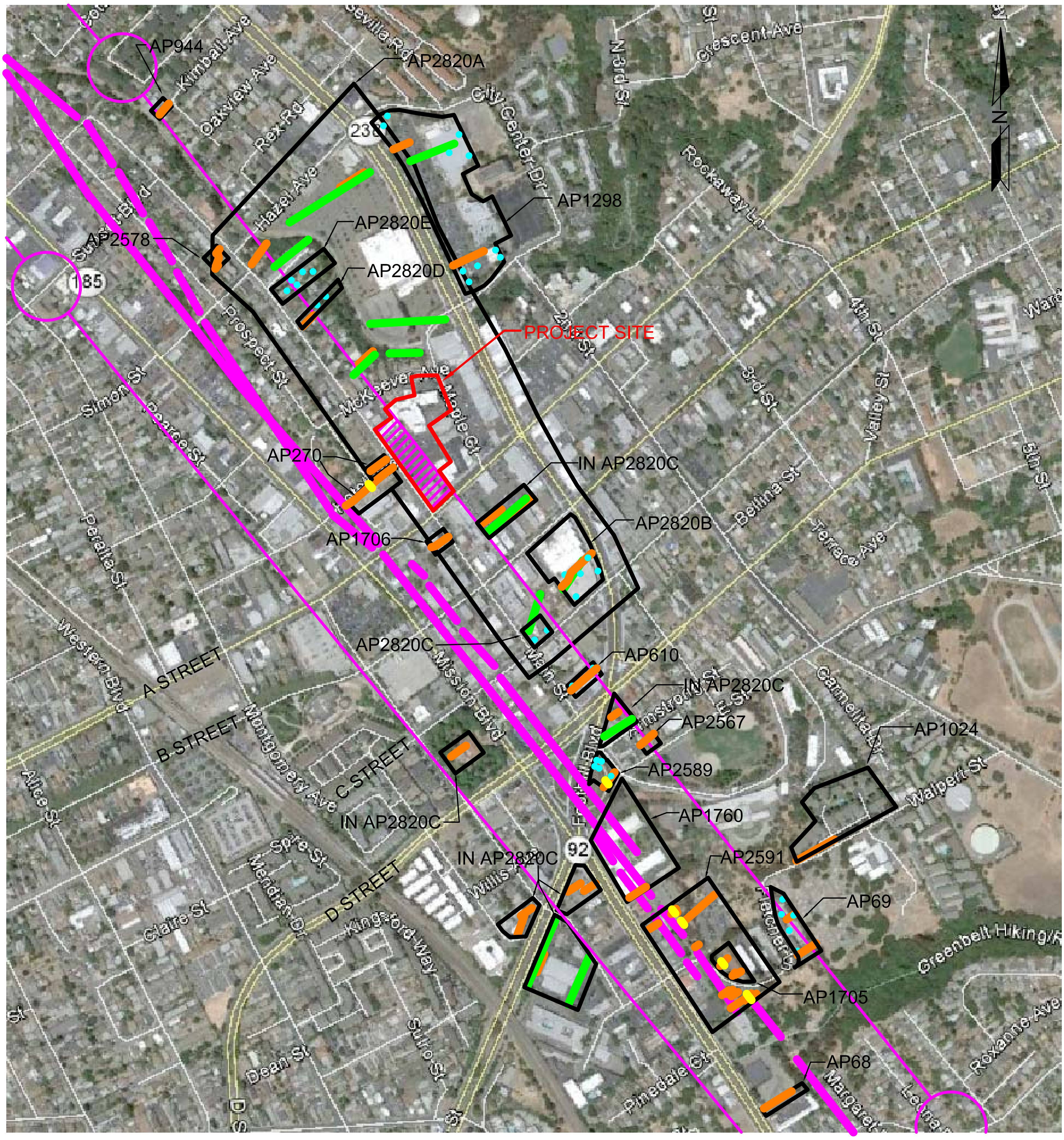
OVERLAPPING ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

NOTE: Base map was taken from "State of California: California Geological Survey Earthquake Zones of Required Investigation Hayward Quadrangle", dated 2012.

APPROXIMATE SCALE: 1" = 600' 0 600' 1200'	DATE November 2014 PROJECT NO. 648-1	Stevens Ferrone & Bailey Engineering Company, Inc 1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	EARTHQUAKE ZONES OF REQUIRED INVESTIGATION 22330 MAIN STREET Hayward, California	FIGURE 4



KEY

Alquist - Priolo Earthquake Fault Zone Boundaries (Per Alquist - Priolo Earthquake Fault Zone Map of Hayward Quadrangle Dated 9/21/12)

Approximate Location of Hayward Fault Trace (Per Alquist - Priolo Earthquake Fault Zone Map of Hayward Quadrangle Dated 9/21/12)

Note: Solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed.

Area of site within Alquist - Priolo Earthquake Fault Zone

Approximate Study Area of Previous AP Fault Reports

Approximate Boring Location as Reported by AP Fault Reports

Approximate Trench Location as Reported by AP Fault Reports

Approximate Location of Seismic Traverse as Reported by AP Fault Report

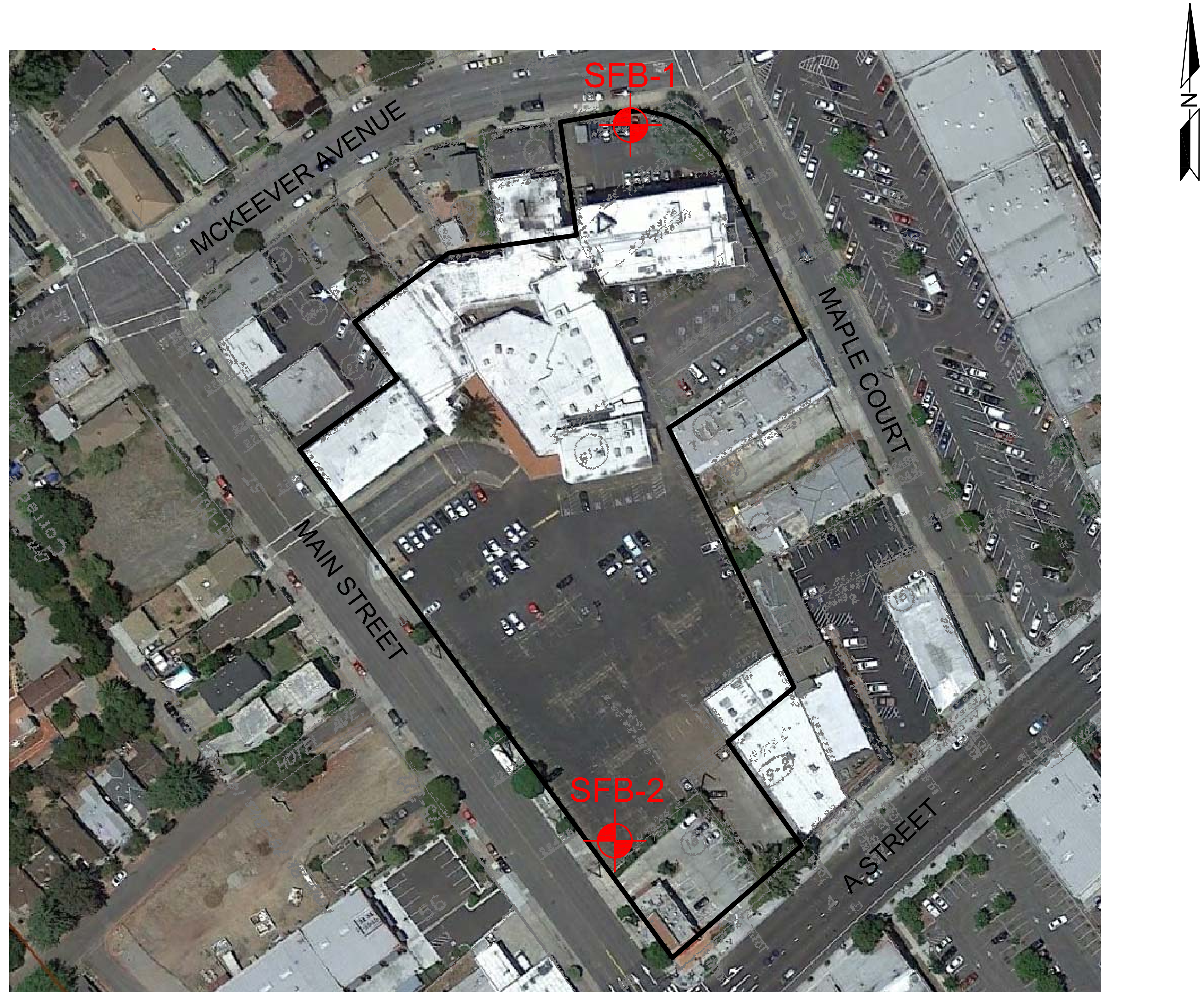
Approximate Location of Reported Fault Trace Logged in Trench

APPROXIMATE SCALE: 1" = 600'

0 600' 1,200'

NOTE: Base map was taken from Google Earth photograph date 6/9/14.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	REGIONAL FAULT STUDY MAP	FIGURE
November 2014			22330 MAIN STREET Hayward, California	5
PROJECT NO.				
648-1				



KEY

SFB-2 APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (11/12/14)

APPROXIMATE PROPOSED PROJECT LIMIT

NOTE: Base map was created by overlaying the Alameda County Assessor Map Book 428 Page 61 on Google Earth image dated 6/9/14.

APPROXIMATE SCALE: 1" = 100' (0, 100', 200')	DATE: November 2014 / PROJECT NO.: 648-1	Stevens Ferrone & Bailey Engineering Company, Inc; 1600 Willow Pass Court, Concord, CA 94520, Tel 925.688.1001, Fax 925.688.1005, www.SFandB.com	SITE PLAN: 22330 MAIN STREET, Hayward, California	FIGURE 6

Historical Ground Failures (modified from Knudsen and others, 2000)

Symbol	Description
⊠	Miscellaneous effects
◎	Sand boil
171	Number assigned to ground failure site (adapted from Youd and Hoose (1978) and Tinsley and others (1998) by Knudsen and others (2000))
B	Pre-Quaternary bedrock. See "Bedrock and Surficial Geology" in Section 1 of report for descriptions of units.
—10—	Depth to ground water, in feet (5, 10, 20 foot contours)
●	Geotechnical boreholes used in liquefaction evaluation
○	Ground-water level data provided by the California State Water Resources Control Board

APPROXIMATE SCALE: (FEET)

0 1,500' 3,000'

NOTE: Base map was taken from "State of California: Seismic Hazard Zone Report for the Hayward 7.5-Minute Quadrangle", dated 2013.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	HISTORICAL GROUND FAILURES MAP	FIGURE
November 2014			22330 MAIN STREET Hayward, California	7
PROJECT NO.				
648-1				

APPENDIX A

Preliminary Field Investigation

APPENDIX A

Preliminary Field Investigation

Our field investigation for the proposed 22330 Main Street development project in Hayward, California, consisted of surface reconnaissance and a subsurface exploration program. Reconnaissance of the site and surrounding area was performed in October and November, 2014. Subsurface exploration was performed using a truck-mounted drill rig equipped with 6-inch diameter, continuous flight, solid stem augers. Two exploratory borings were drilled on November 12, 2014. Our representative continuously logged the soils encountered in the borings in the field. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings as well as a key for the classification of the soil (Figure A-1) are included as part of this appendix.

Representative samples were obtained from our exploratory boring at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. split barrel sampler with liners, and disturbed samples were obtained using the 2-inch O.D. split spoon sampler. All samples were transmitted to our offices for evaluation and appropriate testing. Both sampler types are indicated in the “Sampler” column of the boring logs as designated in Figure A-1.

Resistance blow counts were obtained in our boring with the samplers by dropping a 140-pound safety hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of converted blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The blow counts recorded on the boring logs have been converted to equivalent SPT field blowcounts, but have not been corrected for overburden, silt content, or other factors.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description	Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines	Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			GM	Silty gravels, gravel-sand-silt mixtures				OL	Organic silts and organic silt-clays of low plasticity
			GC	Clayey gravels, gravel-sand-clay mixtures		Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines				CH	Inorganic clays of high plasticity, fat clays
			SP	Poorly-graded sands or gravelly sands, little or no fines				OH	Organic clays of medium to high plasticity
			SM	Silty sands, sand-silt mixtures					
			SC	Clayey sands, and-clay mixtures	Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

trace	<5%
some	5-15%
with	16-30%
-y	31-49%

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KEY TO EXPLORATORY BORING LOGS

22330 MAIN STREET
Hayward, CA

PROJECT NO.	DATE	FIGURE NO.
648-1	November 14	A-1

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY RAC/TC
DEPTH TO GROUND WATER 22 feet	BORING DIAMETER 4-inch	DATE DRILLED 11/12/14

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Asphalt Concrete 2" thick.			0						
Aggregate Base 7" thick.									
FILL: CLAY (CL), dark brown, silty, with sand(fine-grained), trace rootlets, dry.	stiff				13	10	82	0.7	At 1.5': Liquid Limit = 28 Plasticity Index = 12
SILT (ML), light brown, sandy(fine-grained), dry.	stiff				13				
CLAY (CL), dark brown, silty, with sand(fine- to medium-grained), trace rootlets, dry.	very stiff		5		16				
SAND (SM), light brown, fine- to medium-grained, silty, trace rootlets, dry.	medium dense								
CLAY (CL), light brown, silty, with to sandy(fine- to medium-grained), dry.	stiff		10		14				
SAND (SM), light brown, fine- to medium-grained, with to silty, dry to damp.	medium dense		15		18				At 15': Passing # 200 Sieve = 27%
Change color to brown, with clay, damp to moist.			20		24				At 20': Passing # 200 Sieve = 32%
SAND (SP-SM), grayish-brown, fine- to medium-grained, some coarse-grained, some gravel(fine to coarse, subrounded to subangular), some silt, moist.	medium dense		25		20				At 25': Passing # 200 Sieve = 12%
GRAVEL (GW-GM), grayish-brown, fine to coarse, angular to rounded, sandy(fine- to coarse-grained), some silt, wet.	medium dense		30		24				At 30': Passing # 200 Sieve = 7%

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EXPLORATORY BORING LOG

22330 MAIN STREET
Hayward, CA

PROJECT NO.	DATE	BORING NO.
648-1	**November 14**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY RAC/TC
DEPTH TO GROUND WATER 22 feet	BORING DIAMETER 4-inch	DATE DRILLED 11/12/14

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Gravel (GW-GM) Continued. Hole caved in at 35'.	medium dense		35						
CLAY (CL), mottled white gray, silty, trace sand(fine-grained), dry to damp.	hard								
			40		45				
Change color to bluish gray, damp.			45		45				
Bottom of Boring = 46.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.									
			50						
			55						
			60						
			65						

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EXPLORATORY BORING LOG

22330 MAIN STREET
Hayward, CA

PROJECT NO.	DATE	BORING NO.
648-1	**November 14**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY RAC/TC
DEPTH TO GROUND WATER 23 feet	BORING DIAMETER 4-inch	DATE DRILLED 11/12/14

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Aphalt Concrete 1" thick.			0						
Aggregate Base 11" thick.									
FILL: CLAY (CL), dark brown, silty, some gravel(fine to coarse, subrounded to angular), dry to damp.	firm				5	17	98	1.3	At 2': Liquid Limit = 32 Plasticity Index = 14
CLAY (CL), dark brown, silty, some sand(fine-grained), trace rootlets, dry to damp.	firm		5		5 5				
SILT(ML)/SAND(SM) light brown, sandy(fine- to medium-grained), trace clay, trace rootlets, dry to damp. Clayey.	very stiff		10		21				
CLAY (CL), mottled white orangish brown, with sand clasts(fine- to medium-grained), silty, damp.	medium dense		15		20				
SAND (SM), grayish-brown, fine- to medium-grained, with to silty, some clay, damp.	medium dense		20		10				At 20': Passing # 200 Sieve = 27%
CLAY (CL), mottled orangish brown, silty, with sand(fine- to coarse-grained), damp to moist.	stiff								
SAND (SM), dark grayish-brown, fine- to medium-grained, with to silty, some clay, wet.	medium dense		25		11				At 25': Passing # 200 Sieve = 23%
SAND (SC), bluish-gray, fine- to medium-grained, clayey, with silt, wet.	medium dense								
GRAVEL (GM), grayish-brown, fine to coarse, angular to rounded, sandy(fine- to coarse-grained), with silt, trace clay, wet.	dense		30		36				
CLAY (CH), bluish-gray, silty, with small rock fragments, dry to damp.	hard								

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EXPLORATORY BORING LOG

22330 MAIN STREET
Hayward, CA

PROJECT NO.	DATE	BORING NO.
648-1	**November 14**	**SFB-2**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY RAC/TC
DEPTH TO GROUND WATER 23 feet	BORING DIAMETER 4-inch	DATE DRILLED 11/12/14

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
Clay (CH) Continued.	hard		35		75				
			40						
Hole caved in at 45'.			45		50 /4"				
Bottom of Boring = 45.8 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.									
			50						
			55						
			60						
			65						

EXPLORATORY BORING LOG 648-1.GPJ STEVENS FERRONE BAILEY.GDT 11/19/14

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Concord, CA 94523
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EXPLORATORY BORING LOG

22330 MAIN STREET
Hayward, CA

PROJECT NO.	DATE	BORING NO.
648-1	**November 14**	**SFB-2**

APPENDIX B

Limited Laboratory Investigation

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Limited Laboratory Investigation

Our laboratory testing program for the proposed 22330 Main Street development project in Hayward, California was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water contents was determined on two samples of the subsurface soils. The water contents are recorded on the boring logs at the appropriate sample depths.

Dry density determination was performed on two samples of the subsurface soils to evaluate their physical properties. The results of the tests are shown on the boring logs at the appropriate sample depths.

Unconfined compression test was performed on two relatively undisturbed samples of the subsurface soils to evaluate the undrained shear strengths of these materials. Failure was taken as the peak normal stress. The results of the tests are presented on the boring logs at the appropriate sample depths.

The percent passing the #200 sieve was determined on eight samples of the subsurface soils to aid in the classification of these soils. The results of the tests are shown on the boring logs at the appropriate sample depths.

Atterberg Limit determinations were performed on one sample of the subsurface soils to determine the range of water content over which these materials exhibit plasticity. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of the test are presented on the boring logs at the appropriate sample depth.

APPENDIX C

ASFE Information

Important Information about Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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