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October 3, 2017
Project No.: 20181527.001A – Task 20

Mr. Jim Mosier
Project Manager
Project Development Unit
County of San Mateo (County)
1402 Maple Street
Redwood City, California 94063
Jim.mosier@vanir.com

SUBJECT: Addendum to Geotechnical Investigation Report
County of San Mateo Government Center
Lathrop House Relocation Project
Redwood City, California

Dear Mr. Mosier:

At your request, this letter serves as an addendum to our existing geotechnical investigation report for the Lathrop House Relocation project at the County Government Center in Redwood City, California. This letter provides our alternative geotechnical recommendations for foundation design of the subject project after recent communications with you and the design team.

Kleinfelder provided our geotechnical recommendations for the design and construction of the subject project in a report titled *Geotechnical Investigation Report, County of San Mateo Government Center, Lathrop House Relocation Project, Redwood City, California,* dated September 13, 2017 (Project No. 20181527.001A). In that report, we noted that the surficial soils have a high expansion potential and may undergo shrink and swell due to soil moisture content changes, resulting in building distresses. As a result, we recommended that over-excavation be conducted during site grading by excavating the uppermost three feet of the on-site soils within the building footprint and extending laterally to at least 5 feet beyond the building footprint, where physically possible. The idea of over-excavation was to moisture-condition the onsite clays to reduce the potential of excessive swelling/shrinking of the recommended shallow foundation system. Also, varying near-surface materials are expected at the site. The recommended over-excavation would also provide more uniform foundation support.

We now understand that the over-excavation requirement is considered relatively expensive, and an alternative design is desired. We have discussed various options with the design team and the design team has selected the following alternative to be used in the final design.

Instead of conducting over-excavation and designing the foundation to have a footing embedment depth of 18 inches below the lowest adjacent grade, we recommend that no over-excavation be conducted but extending the footing embedment depth to at least 30 inches below the lowest adjacent grade. With this alternative design, differential settlement/heaving between the exterior and interior footings is still expected, and is estimated to be on the order of 2½ inches. During construction, we recommend that the footing subgrade be kept moist prior to concrete placement to pre-swell the soil at the footing subgrade level. Also, during footing excavation, any soft or loose spots should be over-excavated and replaced with engineered fill. All other geotechnical recommendations presented in our geotechnical report of September 13, 2017, remain valid.

CLOSING

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, expressed or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

Kleinfelder appreciates the opportunity to provide continued geotechnical engineering services during this project. If there are any questions concerning the information presented in this letter, please contact the undersigned at this office at your convenience.

Respectfully submitted,

KLEINFELDER, INC.

Omar Khan
Project Geologist

Edward Mak, PE, GE #2212
Geotechnical Engineer

Reviewed by:

Dr. Zia Zafir, PE, GE
Senior Principal Geotechnical/Seismic Engineer

GEOTECHNICAL INVESTIGATION REPORT
COUNTY OF SAN MATEO GOVERNMENT CENTER
LATHROP HOUSE RELOCATION PROJECT
REDWOOD CITY, CALIFORNIA

KLEINFELDER PROJECT No.: 20181527.001A

SEPTEMBER 13, 2017

September 13, 2017
Project No. 20181527.001A

Mr. Jim Mosier
Project Director/Associate
Vanir Construction Management, Inc.
180 Montgomery Street, Suite 700
San Francisco, California 94104
jim.mosier@vanir.com

SUBJECT: Geotechnical Investigation Report
County of San Mateo (County) Government Center
Lathrop House Relocation Project
Redwood City, California

Dear Mr. Mosier:

Kleinfelder is pleased to present our geotechnical investigation report for the proposed Lathrop House Relocation project at the County of San Mateo (County) Government Center in Redwood City, California. Our services were provided in general accordance with our proposal dated June 27, 2017, revised July 31, 2017 (Proposal No. MW170318.001P/PLE17P61836). The attached report summarizes the results of our field investigation, laboratory testing, and engineering analyses. The report also provides geotechnical recommendations for site earthwork, design and construction of foundations, and flatwork for this project.

Based on our field investigation and laboratory testing, it is our opinion that the site is suitable for the planned relocation of the Lathrop House provided that the recommendations presented in this report are followed. Due to some near-surface relatively highly expansive soils and potential varying near-surface soils, mitigation is recommended, which consists of over-excavating about 3 feet of the surface site soils and backfilling with compacted excavated soils. The planned relocation of the Lathrop House can then be supported on a shallow foundation system. Our conclusions and geotechnical recommendations for the design and construction of the proposed building are presented in Section 6 of this report.

The conclusions and recommendations presented in this report are based on limited subsurface exploration and laboratory testing programs. Consequently, variations between anticipated and actual subsurface soil conditions may be found at the project site during construction. It is recommended that Kleinfelder be retained during construction to observe earthwork operations and the installation of foundations to allow us to make changes, if needed, to our recommendations due varying subsurface conditions. Kleinfelder should also review final project plans and details to check conformance with the general intent of our conclusions and recommendations presented in this report.

We appreciate the opportunity to provide our services to you for this project. If you have questions regarding this report or need further assistance, please contact us at your convenience.

Sincerely,

KLEINFELDER, INC.

Omar Khan
Project Geologist

Edward Mak, PE, GE
Geotechnical Engineer

Reviewed by:

Dr. Zia Zafir, PE, GE
Senior Principal Geotechnical/Seismic Engineer

GEOTECHNICAL INVESTIGATION REPORT
COUNTY OF SAN MATEO GOVERNMENT CENTER
LATHROP HOUSE RELOCATION PROJECT
REDWOOD CITY, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the planned Lathrop House relocation project at the County Government Center in Redwood City, California. The approximate location of the project is shown on Figure 1 – Site Vicinity Map.

The existing historic 2-story Lathrop House is currently located at 627 Hamilton Street and the County intends to relocate the building to an area behind the County History Museum along Marshall Street, which is currently occupied by asphalt paved parking, to make room for a new County office building. We have not been provided with specific project details at this time but anticipate the house with raised wood floor will be constructed on spread footings. Structural loads are expected to be light. In the event these structural or improvement details are inconsistent with the final design criteria, our firm should be contacted prior to final design in order that we may update our recommendations as needed.

The conclusions and recommendations presented in this report are based on the subsurface soil conditions encountered at our exploration location and the provisions and requirements outlined in the Limitations section of this report. The findings, conclusions and recommendations presented herein should not be extrapolated to other areas or be used for other projects without our review.

2.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to explore and evaluate the subsurface conditions at the new site in order to develop recommendations related to the geotechnical aspects of project design and construction. The scope of our services was outlined in our Proposal (MW170318.001P/PLE16P61836) dated June 27, 2017, revised July 31, 2017, and included the following:

- A site reconnaissance to observe the surface conditions at the new location of Lathrop House
- A field investigation that consisted of drilling one boring at the new location of Lathrop House to explore the subsurface conditions
- Laboratory testing of selected soil samples obtained during the field investigation to evaluate relevant physical and engineering parameters of the subsurface soils
- Evaluation of the field and laboratory data obtained and performing engineering analyses to develop our geotechnical conclusions and recommendations
- Preparation of this report which includes:
  - Site Vicinity Map, and Site Plan showing the test boring location;
  - Description of the project;
  - Boring log and laboratory test results;
  - Discussion of general site subsurface conditions, as encountered in our soil test boring;
  - Conclusions pertaining to feasibility of the proposed development, impacts of geotechnical and geologic features on the proposed development, and geologic hazards;
  - Recommendations for design of footings including allowable soil pressures and embedment depths;
  - Seismic design parameters in accordance with 2016 California Building Code (CBC);
  - Anticipated total and differential settlements;
  - Flatwork support recommendations;
  - Discussion of liquefaction analysis and associated ground settlement potential and magnitude;
  - Recommendations for site grading, subgrade preparation, earthwork, and fill placement and compaction specifications;
  - Recommendations for surface and subsurface drainage;
  - Soil corrosivity test results; and
  - Discussion of construction considerations.

3.0 GEOLOGIC SETTING

3.1 AREA AND SITE GEOLOGY

The planned relocation site is located approximately 3 miles southwest of the western boundary of the San Francisco Bay and within the Palo Alto 7½-minute quadrangle, which encompasses mainly the Counties of San Mateo, Santa Clara, and a small portion of Santa Cruz. The quadrangle straddles the crest of the northwest-trending Santa Cruz Mountains in the Coast Ranges geomorphic province. The axis of the Santa Cruz Mountains and several broad-crested ridges are aligned roughly parallel to the northwest-trenching San Andreas fault zone, which cuts across the southwestern corner of the quadrangle, in the vicinity of Portola Valley. The flatland areas between the base of the Santa Cruz Mountains and the shoreline of San Francisco Bay are underlain by Quaternary alluvial sediment that originated from the Santa Cruz Mountains and artificial fill consisting of engineered and/or non-engineered fill. Bedrock units exposed in the quadrangle consist of Cenozoic and Mesozoic formations comprised of sandstone, siltstone, conglomerate, shale, basalt, serpentinite, and other Franciscan rocks (California Geological Survey [CGS], 2006a).

The project area has been mapped by Pampeyan (1970, 1993), Brabb and Pampeyan (1983), Brabb et al. (1998, 2000), Knudsen et al. (2000), CGS (2006a), and Witter et al. (2006). Most of these geologic maps differentiate the Quaternary deposits into Pleistocene (between 2.6 million and 11,700 years old) and Holocene (11,700 years old to modern) ages. In addition, the above-noted maps differentiate the surficial deposits based on their mode of deposition and type of alluvial geologic deposit. All the maps are in general agreement that the project site is underlain by Holocene alluvial fine grained fan deposits, as shown on Figure 3 – Area Geology Map, utilitizing the portion of Pampeyan (1993) geologic map of the area. Pampeyan (1993) describes the unit as unconsolidated, poorly sorted, plastic, organic clay and silty clay in poorly drained interfluvial basins, usually at margins of tidal marshlands. Locally contains thin well-sorted interbeds of sand and fine gravel. Whereas Witter et al. (2006) describe the unit as fine-grained alluvial fan and flood plain overbank deposits laid down in very gently sloping portions of the alluvial fan or valley floor. Slopes in these distal alluvial fan areas are generally less than or equal to 0.5 degrees and soils are clay rich. Deposits are dominated by clay and silt, with interbedded lobes of coarser alluvium (sand and occasional gravel). Liquefaction susceptibility is moderate based on shallow groundwater and the presence of lenses of fine sand and silt.

3.2 SEISMIC HAZARD ZONES

The CGS Seismic Hazards Zone maps associated with soil liquefaction and earthquake-induced landslides prepared by the CGS (2006b) for the Palo Alto Quadrangle indicates that the project site is situated within a seismic hazard zone associated with liquefaction. The explanation provided by CGS for seismic hazard zone associated with liquefaction is as follows:

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

3.3 FAULTING AND SEISMICITY

The San Francisco Bay Area is seismically dominated by the active San Andreas Fault system. This fault system movement is distributed across a complex system of generally strike-slip, right-lateral parallel and sub-parallel faults including, among others, the San Andreas, San Gregorio, Hayward, and Calaveras faults.

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart, 2007). The definition of “potentially active” varies. A generally accepted definition of “potentially active” is a fault showing evidence of displacement that is older than 11,700 years (Holocene age [USGS, 2010]) and younger than 2.6 million years (Pleistocene age [USGS, 2010]). However, “potentially active” is no longer used as a criterion for zoning by the California Geological Survey (CGS), formerly known as the Division of Mines and Geology (DMG). The terms “sufficiently active” and “well-defined” are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Zoning Act. A “sufficiently active fault” is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a “well-defined fault” is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition “inactive” generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 2.6 million years).

Based on the data provided in Hart and Bryant (1997), Bryant and Hart (2007) and CGS (2000), the project site is not situated within an Alquist-Priolo Earthquake Fault Zone (AP Zone) established by the CGS around active fault traces. The nearest zoned fault is the San Andreas, approximately 6 kilometers to the southwest of the project site (Jennings and Bryant, 2010). A major seismic event on the San Andreas or other Bay Area faults may cause significant ground shaking at the project site.

4.0 FIELD INVESTIGATION AND LABORATORY TESTING

4.1 FIELD INVESTIGATION

4.1.1 Pre-Field Activities

Prior to the start of the field investigation, Underground Service Alert (USA) was contacted to locate utilities at the boring location within public rights-of-way. We also subcontracted the services of a private utility locator who identified and marked underground utilities in the vicinity of our boring location.

As required by local ordinance, a drilling permit was obtained from San Mateo County Environmental Health Services Division.

4.1.2 Exploratory Boring

The subsurface conditions at the new Lathrop House site were explored by drilling one soil test boring (Boring KLF LHB-1) on August 14, 2017. The approximate boring location is shown on Figure 2 – Site Plan. The boring was drilled to a depth of about 51 feet below existing grade. The boring was drilled using a track-mounted drill rig utilizing 5-inch diameter solid flight augers to a depth of about 20 feet and then switched to mud rotary method using rotary drilling equipment. The boring was drilled by Pitcher Drilling Company of East Palo Alto, California. The boring location was located in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the boring were not surveyed.

A Kleinfelder professional maintained log of the boring, visually classified the soils encountered according to the Unified Soil Classification System presented on Figure A-1 in Appendix A, and obtained relatively undisturbed and bulk samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were in accordance with American Society for Testing and Materials (ASTM) Method D 2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D 2487. The undrained shear strengths of cohesive soil samples were estimated in the field using a hand-held penetrometer device. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring log. The blow counts listed on the boring log has not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Correction factors were applied to the raw blow counts to estimate the sample apparent density noted on the boring log and for engineering analyses. After the boring was completed, it was backfilled with cement grout and patched with asphalt at the surface.

Soil cuttings and drilling mud were placed in 55-gallon drums during drilling. At the completion of our field exploration, a sample of the soil cuttings was collected for analytical testing. The analytical test results indicate that the sample tested was considered non-hazardous, and the soil cuttings were disposed of at a state-licensed facility by our subcontractor.

Keys to the soil descriptions and symbols used on the boring log are presented on Figures A-1 and A-2 in Appendix A. Log of the boring is presented on Figure A-3.

4.1.3 Sampling Procedures

Soil samples were collected from the boring at depth intervals of approximately 5 feet. Samples were collected from the boring at selected depths by driving either a 2.5-inch inside-diameter (I.D.) California sampler or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler driven 18 inches (unless otherwise noted) into undisturbed soil. The samplers were driven using a 140-pound automatic hammer free-falling a distance of about 30 inches. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the logs.

The SPT sampler did not contain liners, but had space for them. The 2.5-inch I.D. California sampler contained stainless steel liners. The California sampler was in general conformance with ASTM D 3550. The SPT sampler was in general conformance with ASTM D 1586.

Soil samples obtained from the boring were packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples were returned to our laboratory for further examination and testing.

4.2 GEOTECHNICAL LABORATORY TESTING

Kleinfelder performed laboratory tests on selected samples recovered from the boring to evaluate their physical and engineering characteristics. The following laboratory tests were performed:

- Unit Weight (ASTM D 2937)
- Moisture Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)
- Particles Finer Than #200 Sieve (ASTM D 1140)

- Unconsolidated Undrained Triaxial Shear (ASTM D 2850)
- Corrosion - Soluble Sulfate Content (ASTM D 4327)
- Corrosion - Soluble Chloride Content (ASTM D 4327)
- pH (ASTM D 4972)
- Minimum Resistivity (ASTM G57)

Most of the tests were sent to our internal laboratory, while the limited corrosion analysis was performed at an accredited laboratory, CERCO Analytical of Concord, California. The results of most of the laboratory tests are included on the boring log in Appendix A. All laboratory test data are summarized in Appendix B. The soluble sulfate, soluble chloride, pH, and minimum resistivity test results are discussed in Section 6.9 of this report and results are presented in Appendix C. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the laboratory test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

5.0 SITE CONDITIONS

5.1 SITE DESCRIPTION

The planned Lathrop House relocation site is currently occupied by asphalt paved parking for the San Mateo County History Museum. The site is relatively flat. The site is bordered by Marshall Street on the north, Hamilton Avenues on the west, the museum on the south, and additional parking on the east.

5.2 SUBSURFACE CONDITIONS

The following descriptions provide a general summary of the subsurface conditions encountered during the current study performed at the site. For more thorough descriptions of the actual conditions encountered at the site, refer to the boring log in Appendix A.

The boring was drilled in an asphalt paved parking area. The asphalt pavement section thickness measured approximately 8 inches with no apparent baserock underneath the asphalt. Underneath the pavement section, medium to hard sandy lean clays and lean clays with interbedded loose to medium dense clayey sands and silty sands were encountered. The uppermost 20 feet of the clayey soils are generally very stiff. Laboratory tests conducted on a near-surface sample indicate the sample tested has a Plasticity Index of 35, suggesting it has high expansive potential. Groundwater was encountered in the boring at approximately 19½ feet below the existing ground surface during our investigation. According to CGS (2006a), the historical highest groundwater levels in the area are less than 10 feet below the ground surface.

Our interpretations of soil and groundwater conditions at the site are based on the conditions encountered in the boring, published geologic maps, and our knowledge of geologic and hydrogeologic conditions in the site vicinity. It is possible that groundwater conditions at the site could change due to variations in rainfall, groundwater withdrawal or recharge, construction activities, well pumping, or other factors not apparent at the time of our investigation. If soil or groundwater conditions exposed during construction vary from those presented in this report, Kleinfelder should be notified to evaluate whether our conclusions or recommendations should be modified.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Based on our findings, it is our professional opinion that the site should be suitable from a geotechnical engineering standpoint for support of the Lathrop House provided the recommendations contained herein are incorporated into the project design and construction.

Due to potential varying near-surface soil conditions and moderately expansive near-surface site soils, over-excavation of the surface soil is recommended. More detail discussion and recommendations regarding over-excavation is provided in Section 6.3 of this report.

Specific conclusions and recommendations regarding the geotechnical aspects of design and construction are presented in the following sections.

6.2 GEOLOGIC AND SEISMIC HAZARDS

6.2.1 Fault-Related Ground Surface Rupture and Strong Ground Shaking

The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site. In our opinion, the potential for fault-related ground surface rupture at the site is low. However, the project area can expect strong ground shaking during the life of the project due to seismic activity along the San Andreas fault or other nearby Bay Area faults.

6.2.2 Slope Stability and Landslide Potential

Since the site is relatively flat, with no topographic relief, the potential for landslide (seismically-induced or otherwise) to impact the project area is considered low to nil.

6.2.3 Quaternary Geology and Liquefaction Potential

As noted above in the Geologic Setting section of this report, the CGS (2006a) prepared a Seismic Hazard Zone Report for the Palo Alto quadrangle in which the project site is located. The CGS differentiated the age of the Quaternary deposits into Pleistocene (between 2.6 million and 11,700 years old) and Holocene (11,700 years old to modern) ages. The younger Holocene age deposits are usually less cemented, less consolidated, and are more susceptible to liquefaction and settlement. In addition, the CGS map differentiated the alluvial deposits based on the type of deposit (such as terrace deposits, alluvial fan deposits, natural levee deposits) based on their deposition mode, topographic position, and grain sizes.

The geologic unit mapped at the project site was discussed and described above. According to the Seismic Hazard Zone Report prepared by CGS, the liquefaction susceptibility for the Holocene alluvial fine grained fan deposits is moderate when highest historical groundwater is less than 10 feet or 10 to 30 feet below ground surface and low when historical groundwater is between 30 to 40 feet.

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. The site is not located within a State of California Seismic Hazard Zones map for liquefaction where areas of historical occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required (CGS, 2006b).

Evaluations of potential liquefaction susceptibility based on soil composition were made according to the criteria of Bary and Sancio (2006) (for the Cetin et al., 2004 SPT analyses) and Idriss and Boulanger (2015) for the Idriss and Boulanger (2006 and 2008) analyses.

For layers that met the compositional criteria, liquefaction triggering (factor of safety) analyses were performed using methodologies proposed by Cetin et al. (2004), and Idriss & Boulanger (2006, 2008). The analyses utilized sample blow count data from the rotary-wash boring drilled for this study. In order to perform liquefaction analysis, estimates of earthquake magnitude and peak ground acceleration (PGA_M) are needed. Using the U.S. Geological Survey (USGS) interactive deaggregation website, the modal earthquake magnitude M_W = 7.87 was estimated and used in the analysis. The peak ground acceleration (PGA_M) value for our analyses was calculated based on Equation 11.8-1 in Section 11.8.3 of the American Society of Civil Engineers (ASCE) 7-10 for the Risk-Targeted Maximum Considered Earthquake (MCE_R). The PGA_M value was calculated using the US Seismic Design Maps application assuming a Site Class D. We used the 2010 ASCE 7 (with March 2013 errata) design code reference document. The calculated PGA_M value is 0.678g for the $MCE_{G.}$

Liquefaction induced volumetric settlements were estimated using the methods of Idriss and Boulanger (2008), and Cetin et al. (2009). For liquefaction analyses the historical highest groundwater depth of 10 feet was used.

Our liquefaction analysis indicates that a silty sand layer at depths between about 28 and 31½ feet could liquefy during a major seismic event. We estimate total settlement due to liquefaction of this layer to be about 1 inch. However, since this potentially liquefiable layer is relatively deep, its impact on shallow foundations near the ground surface would not be significant.

6.2.4 Lateral Spreading

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. These phenomena typically occur adjacent to free faces such as slopes and creek channels. Since no slopes and channels are located in the vicinity of the site, and the potential for lateral spreading at the site is low.

6.2.5 Dynamic Compaction

Dynamic compaction, or seismic settlement, typically occur in unsaturated, loose granular material or uncompacted fill soils. Since the site soils generally consist clayey soils interbedded with relatively thin layers of silty sand and clayey sand, in our opinion, the potential for dynamic compaction at the site is low.

6.2.6 Flooding

The project site is situated within panel 06081C0301E, effective date October 16, 2012, of the Federal Emergency Management Agency (FEMA, 2012) Flood Insurance Rate Map for San Mateo and Santa Clara Counties, California and incorporated areas. According to this FEMA panel, the project site is situated within Zone X – areas of 0.2% annual chance flood.

6.3 EXPANSIVE SOILS AND VARYING SUBSURFACE CONDITIONS

Based on the results of an Atterberg Limits test performed on a near-surface soil sample (Boring KLF LHB-1 at 5 feet below ground surface), the surficial soils have high expansion potential (Liquid Limit of 49, Plasticity Index of 35). These surficial soils may shrink or swell as a result of soil moisture content changes, and the amounts of shrinking and swelling are expected to be moderate. It is our opinion that moisture-conditioning of the clayey soils and maintaining the moisture during site grading, and keying the exterior continuous wall footing into clayey soils could reduce the risk of building distresses due to expansive soils. Also, the site history is unknown. It is possible that the near-surface site soils could vary across the footprint of the building. It is our opinion that over-excavating and recompacting the surface soils could provide more uniform foundation support across the building footprint.

Based on these reasons, we recommend that over-excavation be conducted during site grading by excavating the uppermost three feet of the on-site soils. Where physically possible, the over-excavation should extend laterally to at least 5 feet beyond the building footprint. The excavated soils may be used as backfill of the over-excavation provided all unsuitable materials are removed and the soils are properly moisture-conditioned prior to compaction. The backfill should be compacted as engineered fill in accordance with the compaction requirements presented in this report.

Imported select material meeting the import fill requirements may be used to backfill the over-excavation. However, they should be placed in the upper portion of the over-excavation so that the new exterior continuous footings are keyed at least one foot into the onsite recompacted clayey soils. This requirement reduces the risk of excessive moisture cumulating in the granular fill below the building. If restricting the thickness of the granular fill layer is not possible, deepening the exterior continuous footings may be required.

6.4 CBC SEISMIC DESIGN CRITERIA

Considering the location of the site and the soils that were encountered during the field exploration, the site can be classified as Site Class D according to Table 20.3-1 of the ASCE 7-10. Site Class D is defined as a soil profile consisting of stiff soil with a shear wave velocity between 600 and 1,200 feet/second, standard penetration test (SPT) blow counts (N-value) between 15 and 50 blows per foot, or undrained shear strength between 1,000 and 2,000 pounds per square foot in the top 100 feet. The design code reference document of 2010 ASCE 7 (with March 2013 errata) was used.

The site is located approximately at the following coordinates:

- Latitude: 37.487951 degrees
- Longitude: -122.229983 degrees

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_S and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 5-1. Corresponding site modified (S_{MS} and S_{M1}) and design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M, risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 5-1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps[1].

Table 5-1
Ground Motion Parameters Based on 2016 CBC

Parameter	Value	Reference
S_S	1.705g	2016 CBC Section 1613.3.1
S_1	0.787g	2016 CBC Section 1613.3.1
F_a	1.000	2016 CBC Table 1613.3.3(1)
F_v	1.500	2016 CBC Table 1613.3.3(2)
S_{MS}	1.705g	2016 CBC Section 1613.3.3
S_{M1}	1.180g	2016 CBC Section 1613.3.3
S_{DS}	1.137g	2016 CBC Section 1613.4.4
S_{D1}	0.787g	2016 CBC Section 1613.4.4
PGA	0.678g	ASCE 7-10 Figure 22-7
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.678g	ASCE 7-10 Section 11.8.3
C_{RS}	0.981	ASCE 7-10 Figure 22-17
C_{R1}	0.931	ASCE 7-10 Figure 22-18
T_L	12 seconds	ASCE 7-10 Figure 22-12
Seismic Design Category	D	2016 CBC Section 1613.3.5

6.5 GENERAL EARTHWORK

We recommend that Kleinfelder be retained to provide observation and testing services during earthwork and foundation construction. This will allow us the opportunity to compare conditions exposed during construction with those inferred from our investigation and, if necessary, to expedite supplemental recommendations if warranted by the exposed subsurface conditions. We also recommend that, prior to construction, Kleinfelder be retained to review foundation plans and

[1] http://geohazards.usgs.gov/designmaps/us/

specifications to verify conformance with our recommendations. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings prior to the completion of design and start of construction.

6.5.1 Site Preparation

Site preparation will include demolishing the existing asphalt pavement. Prior to the start of construction, all obstructions, debris and deleterious material, including any existing structures such as foundations, pavements, concrete slabs, buried irrigation lines, wells or utility lines to be abandoned, etc., should be removed from the construction areas. Stumps and primary roots of any trees and brush should be grubbed.

Depressions, voids, and holes (including excavations from removal of underground improvements) that extend below the proposed finished grades should be cleaned and backfilled with engineered fill compacted to the requirements given in the Section 6.5.5 of this report. All clearing and backfill work should be performed under the observation of a representative from Kleinfelder.

6.5.2 Subgrade Preparation

After site clearing, as discussed in Section 6.3 of this report, we recommend that over-excavation be conducted. Over-excavation should be conducted in accordance with the requirements presented in Section 6.3 of this report.

The bottom of the over-excavation as well as all subgrade areas that will receive engineered fill for support of structures should be scarified to a depth of 12 inches, uniformly moisture-conditioned to a moisture content of at least 2 percent above the optimum moisture content, and compacted as engineered fill to between 88 and 92 percent relative compaction (ASTM D 1557). Overexcavation of disturbed soil, scarification and compaction of the exposed subgrade, and replacement with engineered fill may be required to sufficiently densify all disturbed soil.

Following rough grading, construction and trenching activities often loosen or otherwise disturb the subgrade soils. On occasion, this disturbance can lead to isolated movement of the subgrade soils following construction and cracking of overlying slabs and pavement. Accordingly, loose/disturbed areas should be repaired and trench backfill should be properly compacted prior to placement of concrete.

6.5.3 Temporary Excavations

Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. The contractor should be aware that slope heights, slope inclinations, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Flatter slopes and/or trench shields may be required if loose, cohesionless soils and/or water are encountered along the slope face. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a lateral distance equal to one-third the slope height from the top of any excavation. During wet weather, earthen berms or other methods should be used to prevent runoff water from entering all excavations. All runoff water, seepage, and/or groundwater encountered within excavations should be collected and disposed of outside the construction limits.

6.5.4 Fill Materials

The native soils encountered in our borings, minus debris, rock particles larger than 3 inches in maximum dimension, and deleterious materials, should be suitable for use as engineered fill in the proposed building areas. All import fill soils should be nearly free of organic or other deleterious debris, essentially non-plastic, and contain rock particles less than 3 inches in maximum dimension. In general, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of cobbles, rock fragments, and/or clay are acceptable for use as import fill. All imported fill materials to be used for engineered fill should be sampled and tested by the project Geotechnical Engineer prior to being transported to the site. Import fill guidelines are provided below.

Table 5-2
Import Fill Guidelines

Fill Requirement		**Test Procedures**	
		ASTM[1]	**Caltrans**[2]
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D422	202
¾ inch	70-100	D422	202
No. 200	20-50	D422	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 2,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

[1]American Society for Testing and Materials Standards (latest edition)
[2]State of California, Department of Transportation, Standard Test Methods (latest edition)

Trench backfill and bedding placed within existing or future City right-of-ways should meet or exceed the requirements outlined in the current City specifications. Trench backfill or bedding placed outside existing or future right-of-ways could consist of native or imported soil that meets the requirements for fill material provided above. However, coarse-grained sand and/or gravel should be avoided for pipe bedding or trench zone backfill unless the material is fully enclosed in a geotextile filter fabric such as Mirafi 140N or an equivalent substitute. In a very moist or saturated condition, fine-grained soil can migrate into the coarse sand or gravel voids and cause "loss of ground" or differential settlement along and/or adjacent to the trenches, thereby leading to pipe joint displacement and pavement distress.

Trench backfill recommendations provided above should be considered minimum requirements only. More-stringent material specifications may be required to fulfill bedding requirements for specific types of pipe. The project Civil Engineer should develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

6.5.5 Engineered Fill

All fill soils, either native or imported, required to bring the site to final grade should be compacted as engineered fill. Onsite clayey fill should be uniformly moisture-conditioned to a moisture content at least 2 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to between 88 and 92 percent of the maximum dry density as determined by ASTM Test Method D 1557. Imported granular fill should be uniformly moisture-conditioned to a moisture content to near the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent of the maximum dry density. Additional fill lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. Discing and/or blending may be required to uniformly moisture condition soils used for engineered fill.

All trench backfill in building or other structural areas should be placed and compacted in accordance with the recommendations provided above for engineered fill. During backfill, mechanical compaction of engineered fill is recommended.

6.5.6 Wet/Unstable Subgrade Mitigation

If construction is to proceed during the winter and spring months, the moisture content of the near-surface soils may be significantly above optimum. This condition, if encountered, could seriously delay grading by causing an unstable subgrade condition. Typical remedial measures include discing and aerating the soils, mixing the soils with dryer materials, removing and replacing the soils with an approved fill material, stabilization with a geotextile fabric or grid, or mixing the soils with an approved hydrating agent such as a lime or cement product. Our firm should be consulted prior to implementing any remedial measure to observe the unstable subgrade condition and provide site-specific recommendations.

6.6 SHALLOW FOUNDATIONS

6.6.1 Allowable Bearing Pressure

The building may be supported on shallow isolated spread footings and/or continuous wall footings founded on the recompacted site soils. We recommend that a continuous exterior wall footing be used. A net allowable bearing pressure of 2,500 pounds per square foot for dead plus sustained live loading may be used to size column and continuous footings. A one-third increase in the allowable bearing pressure may be applied when considering short-term loading due to wind or seismic forces.

Footings should have a minimum width of 18 inches for continuous footings and 36 inches for isolated square footings. Spread or strip footings should be founded at least 18 inches below the lowest adjacent finished grade.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable coefficient of sliding friction of 0.3 between the foundation and the supporting subgrade may be used for design. This value includes a safety factor of at least 1.5. For allowable passive resistance, an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the side of the foundation may be used. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than ½ inch. Passive resistance in the upper 12 inches of soil should be neglected unless the area in front of the footing is protected from disturbance by concrete or pavement. The allowable friction coefficient and passive resistance may be used concurrently without reduction.

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Some minor heaving due to expansive soils is also expected. Based on the anticipated/assumed foundation dimensions and loads, we estimate the total and differential settlement/heave to be on the order of 1½ inches, provided the recommendations presented in this report are followed.

6.6.2 Construction

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project Geotechnical Engineer just prior to placing steel or concrete to verify the recommendations contained herein are implemented during construction. The structural engineer should evaluate footing configurations and reinforcement requirements to account for loading and settlement.

6.7 EXTERIOR FLATWORK

Subgrade soils underlying exterior flatwork should be scarified 12 inches, moisture conditioned, and recompacted in accordance with the compaction requirements presented in this report. The subgrade preparation should extend beyond the proposed improvements a horizontal distance of at least 2 feet. The moisture content of the subgrade soils should be maintained at least 2 percent above optimum prior to the placement of any flatwork or engineered fill.

Where exterior flatwork is anticipated to be subjected to vehicular traffic, we recommend at least 4 inches of aggregate base, compacted to a minimum of 95 percent of the maximum dry density, be used under the flatwork.

Moisture conditioning to the full 12-inch depth should be verified by the geotechnical engineer's representative. Careful control of the water/cement ratio should be performed to avoid shrinkage cracking due to excess water or poor concrete finishing or curing. Unreinforced slabs should not be built in areas where further saturation may occur following construction.

Exterior concrete slabs for pedestrian traffic should be at least 4 inches thick. Weakened plane joints should be located at intervals of about 6 feet. For large areas of hardscape, expansion joints should be placed at a minimum of 12- to 15-foot intervals.

6.8 SITE DRAINAGE

Foundation and slab performance depends greatly on proper irrigation and how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project. The ground surface around structures should be graded such that water drains rapidly away from structures without ponding. The surface gradient needed to do this depends on the landscaping type. In general, landscape area within 10 feet of buildings should slope away at gradients of at least 5 percent, per Section 1804.4 of 2016 CBC.

We recommend that landscape planters either not be located adjacent to buildings and pavement areas or be properly drained to area drains. Drought resistant plants and minimum watering are recommended for planters immediately adjacent to structures. No raised planters should be installed immediately adjacent to structures unless they are damp-proofed and have a drainpipe connected to an area drain outlet. Planters should be built such that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. Where slabs or pavement areas abut landscaped areas, the aggregate base and subgrade soil should be protected against saturation.

Vertical cut-off structures are recommended to reduce lateral seepage under slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened concrete perimeters, or equivalent, extending at least four (4) inches below the base/subgrade interface. Vertical cut-off structures should be poured neat against undisturbed native soil or compacted clayey fill. The cut-off structures should be continuous.

Roof water should be directed to fall on hardscape areas sloping to an area drain, or roof gutters and downspouts should be installed and routed to area drains. In any event, maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain landscaping plants. Should excessive irrigation, waterline breaks or unusually high rainfall occur, saturated zones and “perched” groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired. Wet utilities should also be designed to be watertight.

6.9 SOIL CORROSIVITY

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Laboratory chloride concentration, sulfate concentration, pH, oxidation reduction potential, and electrical resistivity tests were performed on a near-surface soil sample. The results of the tests are presented in Appendix C and are summarized in Table 5-3. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material.

Table 5-3
Analytical Laboratory Test Results

Boring and Depth	Material	Resistivity, ohm-cm		pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
		Saturated	In-Situ Moisture			Chloride	Sulfide	Sulfate
LHB-1 @ 6 ft.	Lean Clay	870	700	8.47	+300	37	N.D.*	59

*N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the "10-point" method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the site are corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication "Guide to Durable Concrete" (ACI 201.2R-08) provides guidelines for this assessment. The sulfate tests indicated the sample had concentrations of 59 ppm. The results of sulfate test indicate the potential for deterioration of concrete is mild, no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the sample had concentrations of 37. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

7.0 ADDITIONAL SERVICES

The review of final plans and specifications, and field observations and testing during construction by Kleinfelder is an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction. The actual tests and observations by Kleinfelder during construction will vary depending on type of project and soil conditions. The tests and observations would be additional services provided by our firm. The costs for these services are not included in our current fee arrangements.

As a minimum, our construction services should include observation and testing during site preparation, grading, and placement of engineered fill and observation of foundation excavations prior to placement of reinforcing steel. Many of our clients find it helpful to have concrete compressive tests performed for each building even though this information may not be required by any agency. It may also be helpful to perform a floor level and crack survey of all slab-on-grade floors prior to the application of any floor covering. The floor level survey can be readily performed by the client or as an additional service provided by Kleinfelder using a manometer device.

8.0 LIMITATIONS

The conclusions and recommendations of this report are provided for the design and construction of the Lathrop House Relocation project at the County Government Center in Redwood City, California, as described in the text of this report. The conclusions and recommendations in this report are invalid if:

- The assumed structural or grading details change
- The report is used for adjacent or other property
- Any other change is implemented which materially alters the project from that proposed at the time this report was prepared

The scope of services was limited to the drilling of one soil test boring in area accessible to our drill rig. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our subsurface exploration including one boring drilled to a maximum depth of about 51½ feet; groundwater level measurements in the test boring during our field exploration; laboratory testing of natural moisture content, in-place density, plasticity, and shear strength tests; and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more-detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involve greater expense, our clients participate in determining levels of service which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated building loads and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed and the conclusions of this report are modified or approved in writing by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to evaluate whether the recommendations of this report are properly incorporated in the design of this project and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions are encountered. As a minimum, Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications
- Observe the site earthwork operations to assess whether the subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill
- Evaluate whether engineered fill for the structure and other improvements is placed and compacted per the project specifications
- Observe foundation bearing soils to evaluate whether conditions are as anticipated

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, installation of piles, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinions, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to evaluate those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

This report was prepared in accordance with the generally accepted standard of practice that existed in San Mateo County at the time the report was written. No warranty, expressed or implied, is made.

It is the CLIENT'S responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety.

This report may be used only by the client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two years from the date of the report. Land use, site conditions (both on- and off-site), or other factors may change over time, and additional work may be required. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else, unless specifically agreed to in advance by Kleinfelder in writing, will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

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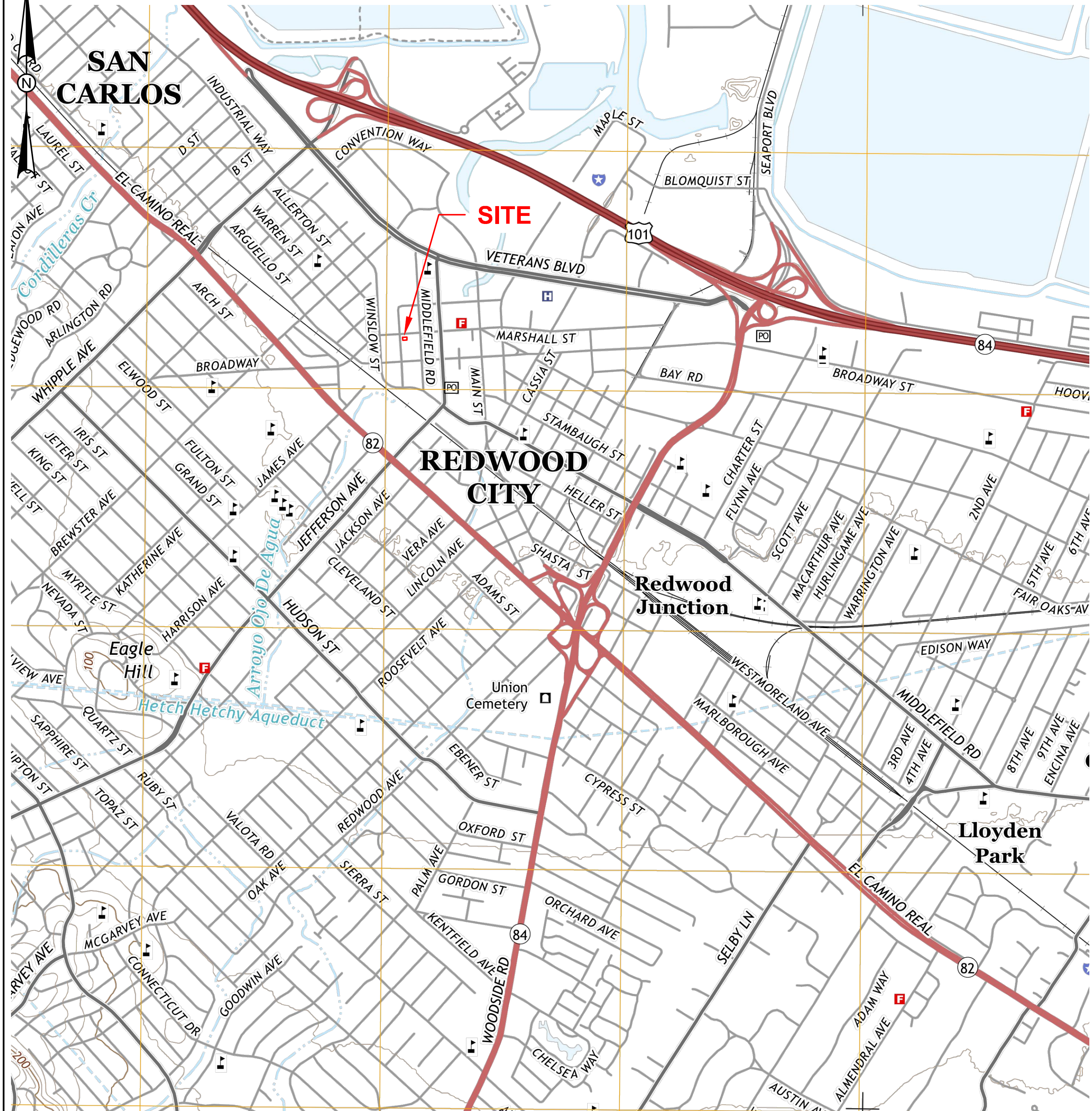
FIGURES

PLOTTED: 9/11/2017 7:28 AM BY: jeff sala



REFERENCE:
VICINITY MAP CREATED FROM DATA COMPILED FROM USGS US TOPO PALO ALTO QUADRANGLE CA. 7.5-MINUTE, 2015.

0 2000 4000

SCALE: 1" = 2000' SCALE IN FEET

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PROJECT NO.	20181527
DRAWN BY:	JDS
CHECKED BY:	OK
DATE:	09/01/2017
REVISED:	

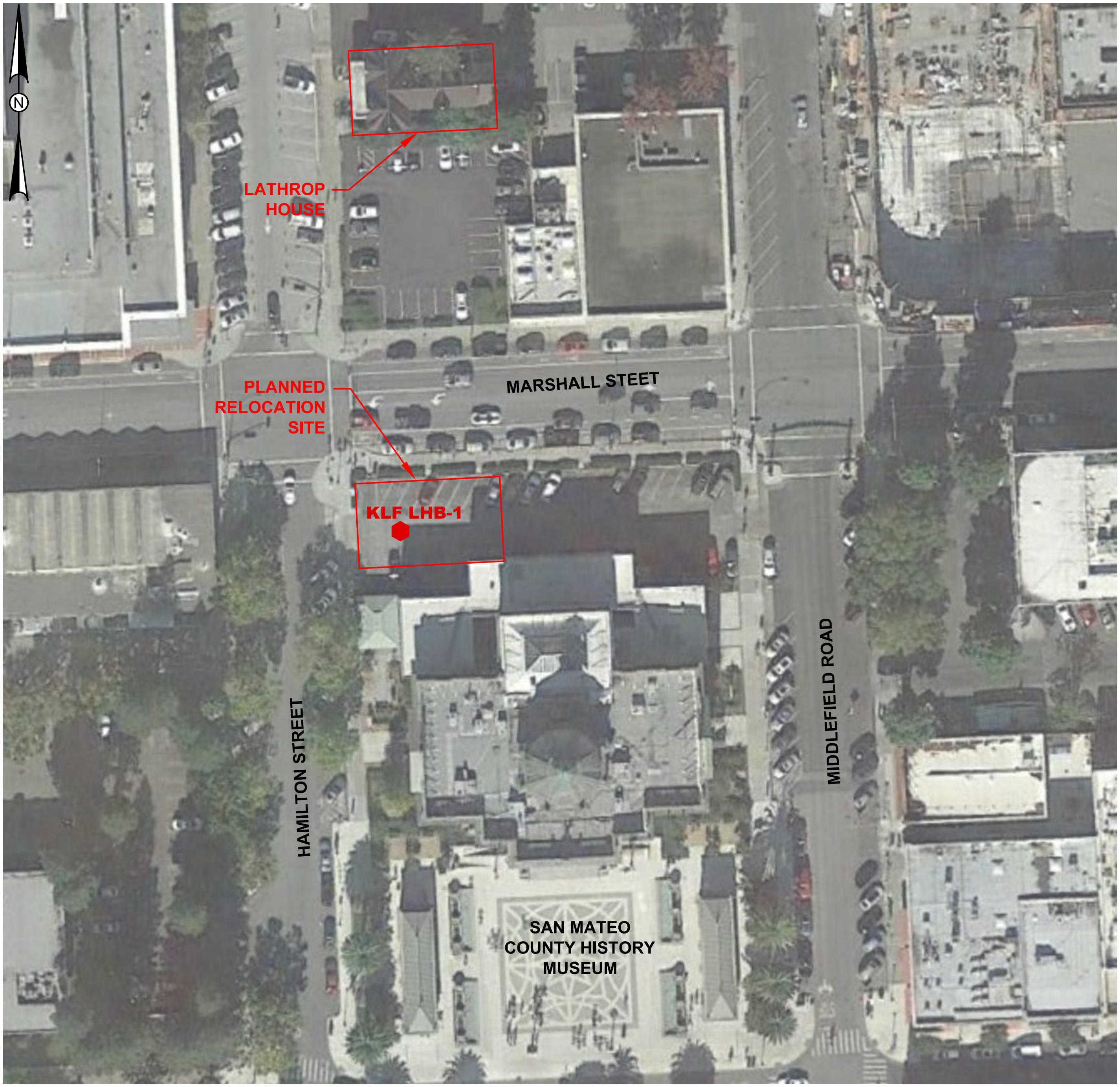
SITE VICINITY MAP

LATHROP HOUSE RELOCATION
HAMILTON STREET
REDWOOD CITY, CALIFORNIA

FIGURE

1

CAD FILE: W:\2018\20181527.001A - San Mateo Government Center_Redwood City\Lathrop House\GRAPHICs\20181527_lh.dwg



0 80 160

SCALE: 1" = 80' SCALE IN FEET

LEGEND

⬢ SOIL BORING (By Kleinfelder, 2017)

REFERENCE:
IMAGE CREATED FROM DATA COMPILED BY GOOGLE EARTH PRO., IMAGERY DATED 11-02-2016.

NOTE: All locations are approximate.

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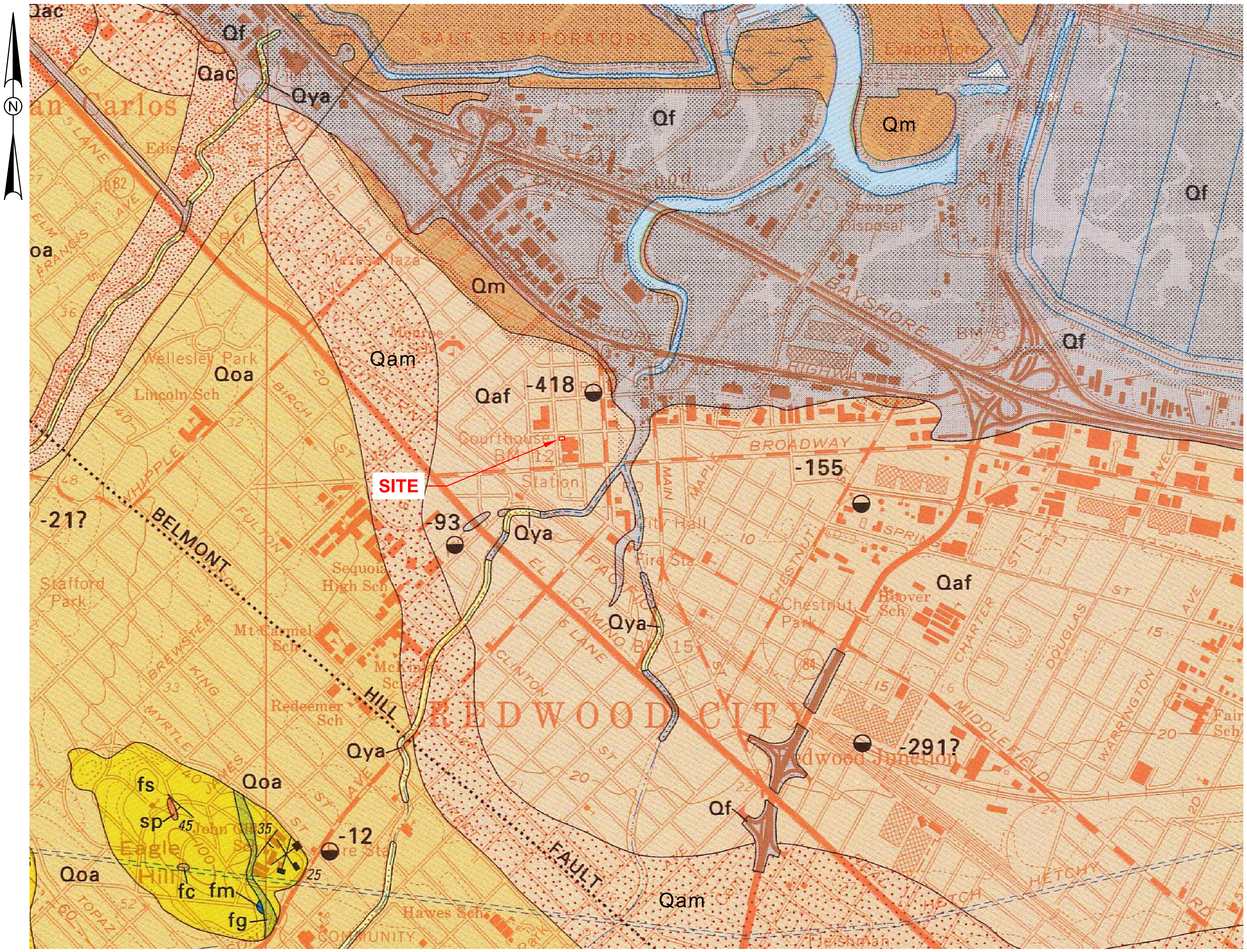
PROJECT NO. 20181527	SITE PLAN	FIGURE
DRAWN BY: JDS		
CHECKED BY: OK	LATHROP HOUSE RELOCATION HAMILTON STREET REDWOOD CITY, CALIFORNIA	2
DATE: 09/11/2017		
REVISED:		



DESCRIPTION OF MAP UNITS

Qya	Younger alluvium (Holocene)
Qf	Artificial fill (Holocene)
Qaf	Fine-grained alluvium (Holocene)
Qam	Medium-grained alluvium (Holocene)
Qac	Coarse-grained alluvium (Holocene)
Qm	Bay Mud (Holocene)
Qoa	Older alluvium (Pleistocene)
fs	Sandstone (Cretaceous and Jurassic)
fg	Greenstone (Cretaceous and Jurassic)
fc	Chert (Cretaceous and Jurassic)
fm	Metamorphic rock (Cretaceous and Jurassic)
sp	Serpentinite (Cretaceous and Jurassic)

Contact—Showing direction and amount of dip where known. Dotted where concealed; queried where uncertain

Fault—Showing direction and amount of dip where known. Dashed where approximately located; dotted where concealed; queried where uncertain. Ball and bar on downthrown side. Arrows indicate sense of relative movement. A, away; T, toward on cross-sections

Thrust fault—Sawteeth on upper plate. Dotted where concealed or inferred

Photo lineament

Major folds—Showing crestline and direction of plunge where known. Dotted where concealed; queried where doubtful

- **Anticline**
- **Syncline**
- **Overturned syncline**

Strike and dip of beds and layering in volcanic rocks—Ball indicates top known from sedimentary structures

- **Inclined, showing dip where known**
- **Vertical**
- **Horizontal**
- **Overturned**

Strike and dip of inclined foliation

Strike and dip of joints

- **Inclined**
- **Vertical**

-732 **Borehole to basement**—Showing elevation of top of basement (Franciscan Complex). Datum is mean sea level

-1059 **Borehole that did not reach basement**—Showing elevation of bottom of hole. Datum is mean sea level

Portal of adit

Quarry

Prospect pit

Approximate location of former shoreline and drainage channels now filled or concealed—Data from U.S. Coast Survey maps T-664, T-665, and T-676 (1857a,b,c), and T-2311 and T-2312 (1897a,b). Boundaries may be off as much as 100 ft on the ground because of difficulties in registering old and modern maps

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0 1200 2400

SCALE: 1" = 1200' SCALE IN FEET

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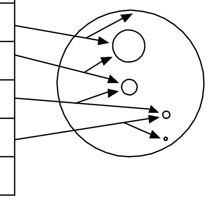
PROJECT NO. 20181527		AREA GEOLOGY MAP	PLATE
DRAWN BY	JDS		3
CHECKED BY	OK	LATHROP HOUSE BUILDING HAMILTON STREET REWOOD CITY, CALIFORNIA	
DATE:	09/01/2017		
REVISED:			

APPENDIX A

LOGS OF EXPLORATIONS

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT: Secondary Constituent is Fine Grained	AMOUNT: Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.:	20181527
DRAWN BY:	JDS
CHECKED BY:	EM
DATE:	8/31/2017
REVISED:	-

SOIL DESCRIPTION KEY

LATHROP HOUSE RELOCATION
HAMILTON STREET
REDWOOD CITY, CALIFORNIA

FIGURE
A-2

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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PROJECT NO.: 20181527	GRAPHICS KEY	FIGURE
DRAWN BY: JDS		A-1
CHECKED BY: EM	LATHROP HOUSE RELOCATION HAMILTON STREET REDWOOD CITY, CALIFORNIA	
DATE: 8/31/2017		
REVISED: -		

Date Begin - End:	8/14/2017	**Drilling Company:**	Pitcher		**BORING LOG KLF LHB-1**
Logged By:	A. Tyler	**Drill Crew:**	Willy/Conner		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track 80	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	SSA/Rotary		
Weather:	Partly Cloudy	**Exploration Diameter:**	5 in. O.D.		

Surface Condition: Asphalt

Depth (feet)	Lithologic Description (FIELD EXPLORATION)	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
0	approximately 8-inches of asphalt										core bit used
	Lean CLAY (CL): trace fine-grained sand, medium to high plasticity, light brownish gray, moist, very stiff										
5	olive brown	BC=6 9 11 PP=3.5	10"	CL					49	35	solid stem auger used Corrosivity test
10	**Sandy Lean CLAY (CL)**: fine to coarse-grained sand, low to medium plasticity, yellowish brown to brown, moist, very stiff, trace gravel	BC=8 10 15 PP=4.0	12"	CL	17.0	111.7					TXUU: c = 2.44 ksf
15	**Clayey SAND (SC)**: fine to coarse-grained sand, low plasticity, yellowish brown to olive, moist, loose, trace gravel	BC=5 6 8	11"	SC				27			
	Sandy Lean CLAY (CL): fine to medium-grained sand, low plasticity, light brownish gray, moist, very stiff, trace gravel	PP=3.0		CL	22.7	104.9					
∇											switched to mud rotary
20	**Clayey SAND (SC)**: fine to coarse-grained sand, low to medium plasticity, gray to grayish brown, wet, loose, with clay pockets, trace gravel	BC=2 3 4	16"	SC				16	27	12	
	Sandy Lean CLAY (CL): fine to medium-grained sand, low to medium plasticity, light brownish gray, moist, stiff, trace gravel										
25		BC=0 6 6 PP=1.5	12"	CL	21.3	105.7					
	Silty SAND (SM): fine to coarse-grained sand, low plasticity, light brownish gray to olive brown, moist, loose to medium dense, trace gravel										

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PROJECT NO.: 20181527	BORING LOG KLF LHB-1	FIGURE
DRAWN BY: JDS		
CHECKED BY: EM	LATHROP HOUSE RELOCATION HAMILTON STREET REDWOOD CITY, CALIFORNIA	A-3
DATE: 8/31/2017		
REVISED: -		PAGE: 1 of 2

Date Begin - End:	8/14/2017	**Drilling Company:**	Pitcher		**BORING LOG KLF LHB-1**
Logged By:	A. Tyler	**Drill Crew:**	Willy/Conner		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track 80	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	SSA/Rotary		
Weather:	Partly Cloudy	**Exploration Diameter:**	5 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Surface Condition: Asphalt

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			BC=4 9 9	7"	SM				34	NP	NP	
	Sandy Lean CLAY (CL): fine to medium-grained sand, low plasticity, dark gray, moist, medium stiff, with sand seams		BC=2 2 3		CL							
	Lean CLAY (CL): trace fine-grained sand, medium plasticity, olive yellow, moist, very stiff to hard											
35			BC=7 9 12 PP=3.3	9"	CL							
40			BC=4 10 13 PP=4	12"	CL							
45	low plasticity, light brownish gray, moist, very stiff, trace gravel		BC=6 8 10 PP=2.3	12"	CL							
50			BC=6 9 15 PP=4.0	12"	ML							
	SILT (ML): trace fine-grained sand, low plasticity, brownish gray, moist, very stiff to hard											
55												

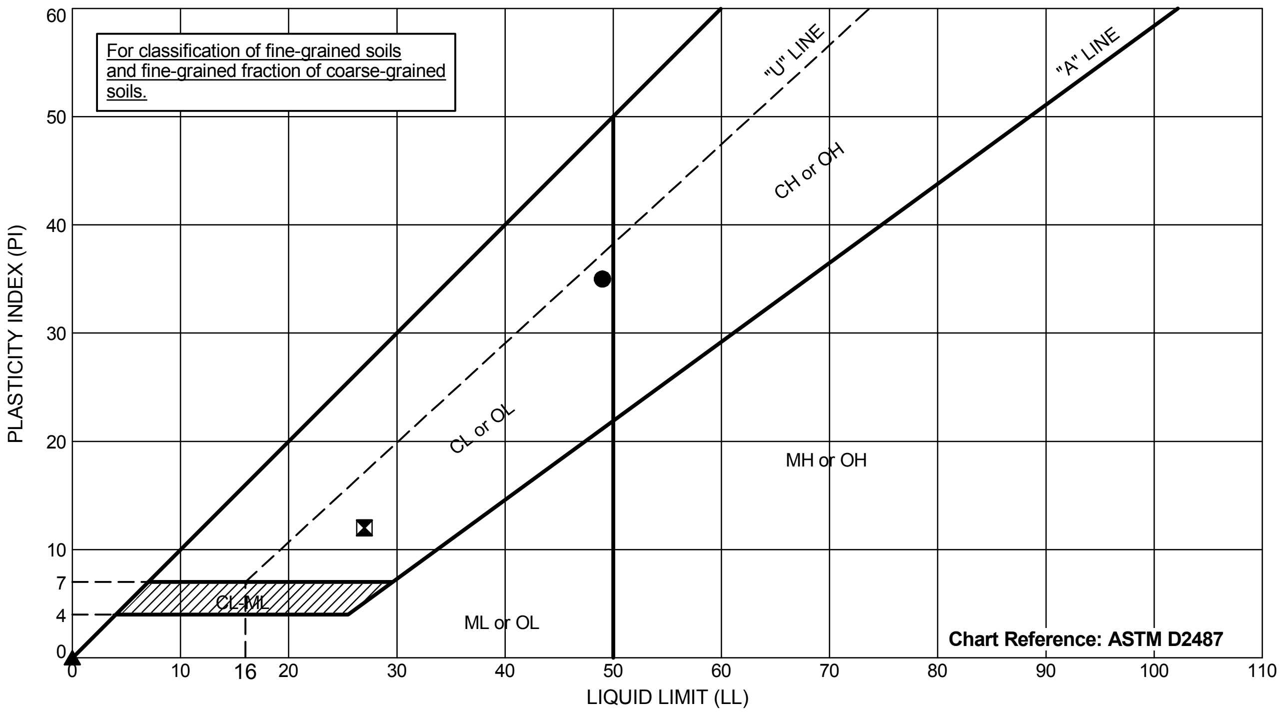
The boring was terminated at approximately 51.5 ft. below ground surface. The boring was backfilled with cement grout and patched with asphalt at surface on August 14, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 19.5 ft. below ground surface during drilling.
GENERAL NOTES:

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20181527	BORING LOG KLF LHB-1	FIGURE
	DRAWN BY: JDS		
	CHECKED BY: EM	LATHROP HOUSE RELOCATION HAMILTON STREET REDWOOD CITY, CALIFORNIA	A-3
	DATE: 8/31/2017		
	REVISED: -		PAGE: 2 of 2

APPENDIX B

LABORATORY TESTING RESULTS



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	KLF LHB-1	5.5	OLIVE BROWN LEAN CLAY (CL)	NM	49	14	35
⊠	KLF LHB-1	20	DARK GRAYISH BROWN CLAYEY SAND (SC)	16	27	15	12
▲	KLF LHB-1	30.5	OLIVE BROWN SILTY SAND (SM)	34	NP	NP	NP

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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PROJECT NO.: 20181527
DRAWN BY: JDS
CHECKED BY: EM
DATE: 8/31/2017
REVISED: -

ATTERBERG LIMITS

LATHROP HOUSE RELOCATION
HAMILTON STREET
REDWOOD CITY, CALIFORNIA

FIGURE

B-2

APPENDIX C

ANALYTICAL TEST RESULTS BY OTHERS

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20181527
Client's Project Name: Lathrop House Relocation
Date Sampled: 14-Aug-2017
Date Received: 30-Aug-2017
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 6-Sep-2017

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1708255-001	LHB-1 1B @ 6'	+300	8.47	700	870	N.D.	37	59

	Redox	pH	Resistivity (As Received)	Resistivity (100% Saturation)	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	1-Sep-2017	31-Aug-2017	31-Aug-2017	31-Aug-2017	6-Sep-2017	5-Sep-2017	5-Sep-2017

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX D

GBA INFORMATION SHEET

Important Information about This

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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