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Geotechnical Engineering
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GEOLOGIC HAZARDS EVALUATION & GEOTECHNICAL INVESTIGATION
761 FLEMING AVENUE RESIDENTIAL DEVELOPMENT
SAN JOSE, CALIFORNIA
SFB PROJECT NO. 1084-1

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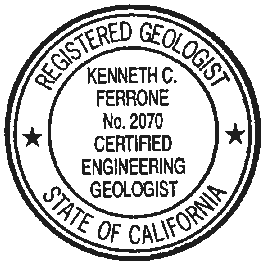


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1.0 INTRODUCTION

This report presents the results of our geologic hazards evaluation and geotechnical investigation for the proposed residential development to be located at 761 Fleming Avenue in San Jose, California. The project site location (approximate latitude 37.368314°N and longitude 121.809009°W) is shown on the **Vicinity Map, Figure 1**, and the **Site Plan and Engineering Geology Map, Figure 2**. The purpose of our investigation was to evaluate the geologic and geotechnical conditions at the site and provide recommendations regarding the geologic and geotechnical engineering aspects of the project.

According to the provided tentative map, we understand that the project will consist of developing approximately 2 acres of land for 9 residential lots. An existing home will be included within one of the lots. A private, asphalt concrete paved roadway will provide access to the lots from Fleming Avenue. Cut and fill grading will be performed and underground utilities will be provided. Several retaining walls will be utilized to create level building pads. Several bio-swales will be included in the project.

Elevations discussed in this report and shown on the exploration logs were taken from Sheet 3 of the Vesting Tentative Map prepared by HMH and dated August 7, 2024.

The conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical aerial photographs, aerial photo images, and topographic maps of the site and surrounding area;
- Reviewing previous geological and geotechnical reports prepared for residential developments surrounding the site area (Tracts 8159, 3740, and 5325) that have been included in the City of San Jose's online permit website;
- Performing reconnaissance and geologic mapping of the site and surrounding area;
- Performing four exploratory backhoe pits to a maximum depth explored of about 10 feet;
- Performing laboratory testing of soil samples retrieved from the pits;
- Performing geological and engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing our opinions regarding geologic hazards and geotechnical design and construction criteria for site earthwork, underground utility, drainage, building foundations, retaining walls, and flatwork. Toxicity potential assessment of onsite materials or groundwater (including mold) was beyond our scope of work. In addition, it was also beyond our scope of work to perform an evaluation of potential flooding and its impact on the project.

3.0 SITE INVESTIGATION

3.1 Field Exploration

Our geologic and geotechnical field exploration program for the project included performing four exploratory pits on September 26, 2024. The approximate locations of the pits are shown on the **Site Plan and Engineering Geology Map, Figure 2**. The pits were performed using a Deere 410L backhoe with a 30-inch wide bucket. Our Certified Engineering Geologist performed a reconnaissance and mapping of the site and surrounding area on September 26, 2024. The results of the mapping are shown on **Figure 2**.

Our field engineer and engineering geologist continuously logged the soils encountered in the pits. The soils were classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). Logs of the pits as well as a key for the classification of the soil (**Figure A-1**) are included in **Appendix A**. Upon completion of our field exploration, the pits were loosely backfilled and wheel-rolled. The pit backfill will require over-excavation and re-compaction during future grading operations.

The approximate locations of our pits were determined by pacing, measurements, and/or alignment from landmark references. Latitude and longitude of exploration locations shown on the exploration logs are estimated from online map data from Microsoft; actual locations were not surveyed.

Representative samples were obtained from our exploratory pits at selected depths appropriate to the investigation. All samples were transported to our geotechnical laboratory for evaluation and appropriate testing.

Previously, numerous exploratory borings were performed in areas (residential development Tracts 8159, 3740, and 5325) that surround the site. The locations of previous borings performed near the project site are shown on **Figure 2**. Logs of the previous explorations (including Site Plans showing the locations of the borings) and associated laboratory testing are provided in **Appendix C** for reference. These previous field and lab results have been incorporated into our geological and geotechnical engineering analyses for the project.

It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report. In addition, our attached exploration logs and related information show our interpretation of the subsurface conditions at the dates and locations

indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

3.2 Laboratory Testing

Our laboratory testing program for the project was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included the following testing:

- Two moisture content determinations per ASTM D2116 and D2937;
- Two Atterberg Limits determination (plastic and liquid limits) per ASTM D4318; and
- Two sieve analyses per ASTM C136, D422, and/or D1140.

All tests were performed by our geotechnical laboratory in Concord, California. The results of the testing are included on the exploration logs and plotted laboratory results are also included in **Appendix B**. The results of previous laboratory testing for developments surrounding the site are included in **Appendix C**.

Two selected onsite soil samples were tested by CERCO Analytical, Inc. in Concord, California for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498). The test results and a brief evaluation summary report prepared by CERCO regarding the onsite soils' potential for corrosion of concrete and buried metal such as utilities and reinforcing steel are included in **Appendix B**. We recommend these corrosion test results be forwarded to the project's underground contractors, pipeline designers, concrete contractors, and foundation designers and contractors.

3.3 Surface Conditions and Site Development

At the time of our investigation and as shown on **Figure 2**, the proposed new development site is located west of Fleming Avenue, south of residential development Tract 8159, east of residential development Tract 5325, and north of residential development Tract 3740. An asphalt concrete paved access road extends from Fleming Avenue and slopes downward into the proposed development site. Along this paved access road, two lots have been developed for residences, and the remaining lots are undeveloped.

Topographically, the lowest elevations within the site are located along the northeastern portion of the site (near the northeastern boundary) and appear to be aligned with the headwaters of North

Babb Creek[1]. Water was observed in the northern portions of the creek alignment within the site, with the creek flowing westward and southwestward near the northwestern boundary. The approximate alignment and flow direction of the observed creek are shown on **Figure 2**. The remaining portions of the site slope downward toward the creek. At the time of our investigation, the water observed within the creek varied in depth between 2 and 6 inches.

An existing residence is located in the southernmost corner of the site. The residence is surrounded by landscaping and incudes a pool structure north of the residence. Wood fencing surrounds this development. The paved access road from Fleming Avenue extends to the garage structure of the residence. To the northeast of the residence, we observed a soil covered, graded pad that appears to have been the location of a previous storage shed along with parking access.

Beyond the residence, graded pad, and the paved access road, the site was covered with a thick growth of both small and large diameter trees, shrubs, and ground cover. West of the paved access road within the site, in the lower elevation areas, the ground was wet with occasional water seepage observed at the ground surface. Water was observed in the areas shown on **Figure 2** and appeared to be the reason for the development of North Babb Creek. Water was observed ponding at elevations as high as about Elevation 221 feet.

As shown on **Figure 2**, a few feet of fill was observed throughout the site, with the greatest thicknesses observed along the north side of the existing residential development and also in the area of the graded pad area. The site appears to have been nominally graded over a period of many years to aid in the collection and re-direction of surface water flows toward North Babb Creek. Our review of historic topographic maps and aerial photographs indicate that the proposed development site was previously surrounded by orchards but the site remained undeveloped except for a structure(s) seen in the approximate location as the current residence.

3.4 Subsurface Conditions

Our exploratory pits generally encountered two to three feet of soft to firm, clayey fill materials with varying amounts of roots except in Pit EP-4 where the fill extended to a depth of about seven feet and gravel, cobbles and brick were encountered between depths of three and seven feet. The fill was generally dry to damp except in Pit EP-2 where water was observed seeping through the fill. These fills are weak, heterogenous, and compressible under proposed fill and improvement loads.

[1] Thompson and Sowers, 2005, *Creek & Watershed Map of Central San Jose & Vicinity*: Oakland Museum of California, Oakland, CA.

Below the fill materials, the pits encountered stiff to hard, silty clays with variable sand contents to the maximum depth explored of about 10 feet. In Pits EP-2, -3, and -4, water was encountered at depths of about 0 to 2 feet (Elevation 216 feet), 8 feet (Elevation 217 feet), and 7 feet (Elevation 222 feet), respectively. The water seepage rate into the pits was relatively high at those elevations. We would expect that soft, compressible, and weak soils exist in the areas where water seepage and ponding occur at the site. Based on our experience, we anticipate these soils extend to depths of 2 to 4 feet.

Based on the laboratory testing results of soil samples retrieved from the pits, the onsite more clayey fills and soils have a high plasticity and a high expansion potential. Detailed information of the materials encountered by our exploration are indicated on the exploration logs in **Appendix A**. Results of laboratory testing of onsite soils are included in **Appendix B**. Logs of previous explorations and laboratory testing results performed for the surrounding residential developments are provided in **Appendix C.**

3.5 Groundwater

Groundwater was encountered in Pits EP-2, -3, and -4 at depths of about 0 to 2 feet (Elevation 216 feet), 8 feet (Elevation 217 feet), and 7 feet (Elevation 222 feet), respectively. The water seepage flow rate into the pits was relatively high at those elevations. In addition, water was observed at the ground surface throughout the site at elevations below approximately 221 feet except for in the area east of the paved access road (the area of the observed graded pad). We also observed water flowing out of the ground along the northern side of the driveway before it reached the development site. Approximate locations where water seepage was observed on the ground surface are shown on **Figure 2**. Please be aware that not all seepage locations have been mapped since much of the area was covered with a very thick growth of vegetation.

Fluctuations in the seepage and groundwater level could occur due to change in seasons, variations in rainfall, and other factors. It is likely that during or after rainfall events, localized groundwater or seepage may develop at higher elevations than those noted above and will seep/flow toward lower elevations.

3.6 Hydrologic Soil Group

The surface soils at the site have been mapped by the USDA Natural Resource Conservation Services (NRCS) Web Soil Survey (WSS)[2] and categorized as Urbanland-Altamont-Alo complex, 15 to 30 percent slopes. This map unit is assigned to Hydrologic Soil Group Unranked to C and D by USDA and was estimated to very low to moderately low transmission rates (approximately 0

[2]USDA NRCS, https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed 10/8/2024.

to 0.3 inches per hour). Group C and D soils are defined as having low infiltration rates (high runoff potential) when thoroughly wet and consist mainly of clays and clay loams.

Actual field infiltration rates will depend on the in-situ soil type, moisture, relative density, gradation, and fines content of soils, whether any water impeding clay layers exist at shallow depth, and proper and regular maintenance of the infiltration facilities. If needed, we recommend field Double Ring Infiltrometer Tests (ASTM D3385) be performed at the potential infiltration depths to evaluate the field infiltration rates.

4.0 GEOLOGIC CONDITIONS AND HAZARDS

4.1 Geology and Faulting

4.1.1 Regional Geology

According to Wentworth, et al (1999)[3], the project site (below surficial fills) is underlain by Holocene alluvial fan deposits in the lower elevations and conglomerate (bedrock) of the Cretaceous Berryessa Formation in the higher elevations. The alluvial fan deposits are described as brown sand and sandy gravel grading upward to sandy and silty clay and being moderately dense to dense. The conglomerate is described as thick, indistinct beds of pebble, cobble, and less common conglomerate interfingered with coarse-grained mica-quartz-lithic wacke. Helley, et al (1994)[4] maps similar geology showing Holocene alluvial deposits being located in the lower elevations and bedrock in the higher elevations of the site. Dibblee and Minch (2005)[5] map alluvium in the lower elevations and conglomerate of the Upper Cretaceous Panoche Formation (locally called Berryessa Formation) in the upper elevations. The California Geological Survey's online data viewer[6] of the California Geology Map shows the site to be underlain by Cretaceous bedrock with areas to the northwest and west of the site being underlain by Quaternary older alluvium.

On all geology maps described above, a fault is shown along Fleming Avenue (about 320 feet northeast of the site) and a fault is shown no closer than about 1,100 feet southwest of the site. The U.S. Geological Survey (data last reviewed December 13, 2000)[7] characterizes these faults as being part of the southeast extension of the Hayward fault zone, of late Quaternary age, vertical in orientation, having right lateral slip, having an inferred location, and of good mapping certainty.

We did not uncover any published maps that show faulting through the site or within 100 feet of the site boundaries.

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[3]Wentworth, Blake, McLaughlin, and Graymer, 1999, *Preliminary Geologic Map of the San Jose 30'x60' Quadrangle, California*, USGS Open File Report 98-795.

[4] Helley, Graymer, Phelps, Showalter, and Wentworth, 1994, *Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California*, USGS Open File Report 94-231.

[5] Dibblee and Minch, 2005, *Geologic Map of the San Jose East Quadrangle, Santa Clara County, California*, Dibblee Geological Foundation, Dibblee Foundation Map DF-155.

[6] https://maps.conservation.ca.gov/cgs/dataviewer/, web site accessed 10/9/2024.

[7] U.S. Geological Survey, U.S. Quaternary Faults, USGS Geologic Hazards Science Center, Golden Colorado, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf, web site accessed 10/9/2024.

4.1.2 Site Geology

The attached **Figure 2** shows the results of our geological mapping of the site. Based on the results of our reconnaissance and exploratory pits, the site is blanketed by variable thicknesses of fill, with the greatest thicknesses of fill mapped as Qaf on **Figure 2**. Below the fill, the pits encountered alluvial soils (mapped as Qa) consisting of clays with variable silt and sand contents that extended to the maximum depth explored of 10 feet. We did not observe bedrock in our pits or exposed onsite.

Water seepage and flowing water was observed onsite in the approximate locations shown on **Figure 2**. Thick growth of vegetation was observed in areas of surface or near-surface water. Water channels were also observed onsite and are approximately shown on **Figure 2**. Creek water generally flowed northwestward.

No landslides or evidence of landsliding was observed onsite. As shown on **Figure 2**, large and deep landslide deposits (mapped as Qls) appear to be located immediately east of Fleming Avenue. We did not observe disruption due to landsliding of the access driveway that extends from Fleming Avenue down and into the site.

4.1.3 Active Faulting

The site is not located within an Alquist-Priolo Earthquake Fault Zone as designated by the State of California[8]. The nearest State designated Fault Zone is located approximately 3,700 feet northwest of the site. However, maps prepared for Santa Clara County[9] and the City of San Jose[10] show the site located within an area designated as a fault zone or Special Studies Zone (established for fault hazards), respectively.

As part of our work, we investigated the possibility that an active fault crosses the site or near the site. During our field reconnaissance, we walked the site and the surrounding area to look for features that may be indicative of active faulting. We did not observe features within the site or near the site that exhibit faulting features except for along Fleming Avenue where the geology to the east of the roadway appears to be disrupted by faulting.

We also reviewed topographic maps, historical aerial photo images, and stereoscopic photographs of the site and surrounding area. The topographic maps and historic aerial photo images we reviewed ranged in date from 1897 to 2022. Stereoscopic, aerial photos we reviewed were dated 1939, 1940, 1949, 1965, and 2002. Except for fault created features seen along the current

[8]California Department of Conservation, Earthquake Fault Zones, CGS Special Publication 42, Revised 2018.
[9] Santa Clara County Geologic Hazards Zones, 2012, Sheets 12 and 21.
[10] City of San Jose, 1983, Fault Hazards Maps, San Jose East Quadrangle.

alignment of Fleming Avenue, we did not observe fault features extending through the site or within 100 feet of the site boundaries.

Based on the results of our field work, literature review, and topographic and aerial photo review, it is our opinion that active faulting (faulting that has disrupted sediments less than about 11,000 years old) does not extend through the planned residential portion of the site nor within 100 feet of the residential portion of the site.

It should be noted that none of the previous geotechnical investigation reports that we reviewed for the nearby residential developments (Tracts 8159, 3740, and 5325) indicated that active faulting was present near or within the site.

It is our opinion that, based on the results of our investigation as described herein, the potential for known faults to rupture the ground surface at the site is low.

4.2 Seismicity

Earthquake intensities will vary throughout the region, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2016)[11] indicated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Therefore, the site will be subjected to earthquakes that cause strong ground shaking.

According to 2022 CBC/ASCE 7-16, the site modified geometric mean peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) event is estimated to be about 0.96g based on a stiff soil condition (Site Class D). The MCE peak ground acceleration generally has a 2% probability of being exceeded in 50 years (a mean return period of 2,475 years) except where deterministically capped along highly active faults.

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0)[12], the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.77g in 50 years (a ground motion based on a Site Class D, with a mean return period of 475 years).

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[11]Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, *Earthquake Outlook for the San Francisco Bay Region 2014–2043*, USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).
[12]USGS Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed 10/2/2024.

The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying soils.

4.3 Landsliding

The State of California has designated all or portion of the site as being within a landslide hazard zone[13]. The California Geological Survey (CGS)[14] shows massive landslides being located to the east of Fleming Avenue. The closest landslide to the site shown by CGS is at the approximate location shown on **Figure 2** just east of Fleming Avenue. This landslide is categorized by the CGS as a definite, dormant, mature rock slide exhibiting historic, recent, or active movement and with a depth of greater than 50 feet. Nilsen (1975)[15] does not show landslide deposits within the site or in the vicinity of the site (he only shows a tongue of alluvium extending into the site at the lower elevations).

As described previously, we did not observe landslides or evidence of landsliding onsite. In addition, we did not observe disruption due to landsliding of the access driveway that extends from Fleming Avenue down and into the site. It is our opinion that, based on the results of our geologic literature review, geologic reconnaissance, and exploratory pits, the potential for landsliding at the site is low provided the recommendations contained in this report are implemented in the design and construction of the project. It is likely that the close proximity of the paved driveway to the mapped landslide at Fleming Avenue is the reason for the site to be mapped as being within a landslide hazard zone.

4.4 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts.

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[13] State of California, California Geological Survey, Earthquake Zones of Required Investigation, https://maps.conservation.ca.gov/cgs/EQZApp/app/, accessed 10/9/2024.

[14] California Geological Survey, Data Viewer, https://maps.conservation.ca.gov/cgs/dataviewer/, California Landslide Inventory, accessed 10/9/2024.

[15] Nilsen, Tor H., 1975, *Preliminary Photointerpretation Map of Landslide and Other Surficial Deposits of the San Jose East 7-1/2' Quadrangle, Santa Clara County, California*, USGS Open File Map 75-277-49.

The State of California does not map the site as being within a liquefaction hazard zone[16]. Based on our review of available literature and the results of our field work, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction is low.

4.5 Expansive Soils

The site is underlain by clayey fills and soils having a high plasticity and high expansion potential. Therefore, the potential for an expansive soil hazard at the site is high. Provided the recommendations contained in this report are implemented in the design and construction of the project, the potential for expansive soils causing severe distress on the completed project will be low.

4.6 Compressible Soils

Man-made fills generally blanket the site surface and soft compressible native soils are located in the lower elevations of the site in areas impacted by water flows and seepage. These fills and soils are heterogenous, weak, and compressible. In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, retaining walls, exterior flatwork, and pavements), we recommend these fills and soft soils be completely removed and re-compacted in accordance with the recommendations presented in this report. The over-excavation should extend to depths where competent native soils are encountered.

4.7 Corrosive Soils

Two soil samples were retrieved from the borings, tested for corrosion properties by CERCO Analytical, Inc., and a brief evaluation summary report was prepared by CERCO which is included in **Appendix B**. According to the report, the onsite near-surface soils should be considered as corrosive based on resistivity and redox potential measurements. We recommend these corrosion test results be forwarded to the project's underground contractors, pipeline designers, concrete contractors, and foundation designers and contractors, and appropriate corrosion protection measures should be implemented for the project improvements if necessary.

[16] State of California, California Geological Survey, Earthquake Zones of Required Investigation, https://maps.conservation.ca.gov/cgs/EQZApp/app/, accessed 10/9/2024.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed project from a geologic and geotechnical engineering standpoint. The conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to reduce soil or foundation related issues. The following are the primary geologic and geotechnical considerations for development of the site.

SURFACE AND SUBSURFACE WATER: The North Babb Creek extends through the site. The portion of the creek southeast of the paved driveway was dry at the time of our investigation, however the portion of the creek located northwest and west of the paved driveway contained surface water as much as about 2 to 6 inches in depth. It can be assumed that during rainy months, the extent (length, width, and depth) of the creek will be larger than what we observed during our late September 2024 field work.

At the time of our field work, we observed seepage and ponded water in the approximately northwestern half of the site at elevations of about 222 feet and lower. It is anticipated that during rainy months, this seepage elevation may be higher. In addition, the observed seepage within the exploratory pits was occurring at a relatively high rate resulting in water quickly ponding within the pits.

In order to reduce the potential for erosion and saturation of proposed fill materials and supporting native subgrade which can lead to ground surface settlement and damage, we recommend North Babb Creek be appropriately controlled and contained within a piping system and that an extensive subdrain system be installed throughout the development. We recommend subdrains be installed below all fill materials, at the perimeter of the planned lots, and below pavement subgrade including the pavement subgrade that extends to the development area from Fleming Avenue. The purpose this subdrainage system is to lower the water seepage elevation to depths that would reduce the potential for negative impacts to the development and access roadway. Subdrain water should be collected in solid pipes that discharge to the North Babb Creek piping system.

The actual location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction. We recommend SFB prepare a Geotechnical Improvement Plan once the grading and improvement plans are finalized. The plan would show the approximately locations of subdrains and over-excavation elevations.

We recommend grading and underground utility contractors closely review this report and inspect the site to determine how best to perform the grading in wet conditions and install below ground

utilities with respect to the relatively high groundwater conditions. Extensive dewatering (and possibly shoring for utility installation) may be needed in areas where groundwater exists.

Prior to design and construction, and since it appears that North Babb Creek is a tributary, it is highly recommended that potential jurisdictional agencies (including Army Corp of Engineers, Fish & Wildlife, Flood Control, and City of San Jose) be contacted to determine what work can be performed within and along the creek.

WEAK FILLS AND SOILS: As described previously, the site is generally blanketed by man-made fills. Our pits encountered fills ranging from about 2 feet to 7 feet deep. In addition, we expect soft soils to exist in areas exposed to seepage, ponded water, and creek flows. These fills and soils are weak and compressible under the planned improvements.

In order to reduce the potential for damaging total and differential settlement of overlying improvements (such as new fills, building foundations, retaining walls, exterior flatwork, and pavements), we recommend these weak fills and soils be completely removed and re-compacted. Deeper removal will be needed in areas where thicker weak fills and soils are encountered during grading. The over-excavation should extend to depths and widths where competent soils are encountered.

Where the over-excavation limits abut adjacent, existing structures or improvements, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent structures or improvements are not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations.

The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation. The removed fills and soils can be used as new fill provided they are placed and compacted in accordance with the recommendations presented in this report.

EXPANSION POTENTIAL: The onsite expansive clayey soils will be subjected to volume changes during seasonal fluctuations in moisture content. To reduce the potential for post-construction distress to the proposed structures resulting from swelling and shrinkage of these materials, we recommend that the proposed buildings be supported on post-tensioned slab foundations that are designed to reduce the impact of the onsite expansive soils. In order to reduce the potential for differential foundation movements, we also recommend at least 3-feet of well-blended, moisture conditioned, engineered fills be provided below the post-tensioned slab foundations. This 3-foot engineered fill layer should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways). It should be noted that special design considerations will be required for exterior slabs.

EROSION AND SLOPE MAINTENANCE: Drainage and erosion control measures should be maintained during and after construction. Short-term and long-term erosion control are critical for the stability of any exposed cut and fill slopes, and may be necessary for the natural slopes in order to reduce sediment accumulation in the drainage systems. We recommend all exposed cut and fill slopes be seeded or planted with appropriately designed erosion resistant vegetation and fertilizer. The vegetation should be appropriately irrigated in order to establish and maintain growth. Over-watering must be avoided in order to reduce surficial instability and erosion. Vegetation should be deeply rooted to aid in the interlocking of the near-surface soils. Additional seeding and planting may be necessary in localized areas if the initial seeding or planting is unsuccessful. After seeding, fertilizing, and planting, staked erosion control blankets might be necessary to further stabilize the surficial soils.

Additional erosion control measures will need to be designed and implemented prior to the rainy season based upon the site's configuration. The measures could include straw wattles, silt fencing, hay bales, sediment collection basins, and filtration systems. Silt fencing should be designed for the site's soil type. Storm water discharge and release points from silt fencing should be designed to reduce erosion. In areas exposed to winter rains, we recommend an erosion control plan be prepared and implemented at least two weeks prior to forecasted rains. The erosion control measures will require inspection, modification, and re-mediation during the rainy season in order to comply with regulatory requirements.

CORROSION POTENTIAL: Two selected onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities and reinforcing steel. The results of these tests and brief evaluation summary of the results are included in **Appendix B**. We recommend these test results and brief evaluation summary be forwarded to your concrete contractors, underground contractors, pipeline designers, and foundation designers and contractors so they can design and install corrosion protection measures.

Please be aware that we are not corrosion protection experts; we recommend corrosion protection measures be designed and constructed so that all concrete and metal, including foundation reinforcement, are protected against corrosion. We also recommend additional testing be performed if the test results are deemed insufficient by the designers and installers of the corrosion protection. Landscaping soils typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete; landscaping soils commonly are in contact with foundations. Consideration should be given to testing the corrosion potential characteristics of proposed landscaping soils and other types of imported or modified soils in order to design and provide protection against corrosion for the foundation and pipelines.

ADDITIONAL RECOMMENDATIONS: Detailed earthwork, underground utility, drainage, building foundation, retaining wall, and flatwork recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations.

It is the responsibility of the contractors to provide safe working conditions at the site at all times. We recommend all OSHA regulations be followed, and excavation safety be ensured at all times. It is beyond our scope of work to provide excavation safety designs.

5.1 Earthwork

5.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions, including designated existing structures and their entire foundation systems, existing utilities and pipelines and their associated backfill, designated trees and shrubs and their entire root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 5.1.5, *Fill Material***, and compacted to the requirements in **Section 5.1.6, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems, if they exist, should be abandoned in accordance with City and County standards.

From a geotechnical standpoint, any existing trench backfill materials, clay or concrete pipes, pavements, baserock, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill material in **Section 5.1.5, *Fill Material***. We recommend fill materials composed of broken up concrete or asphalt concrete not be located within 3 feet of the ground surface in yard areas. Consideration should be given to placing these materials below pavements, directly under building footprints, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB. Crushed concrete materials from concrete demolition can be re-used onsite as aggregate base or subbase if they meet current Caltrans specifications for aggregate base or subbase based on laboratory testing results.

Portions of the site containing surface vegetation should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by SFB at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

5.1.2 Weak Fill and Soil Re-Compaction

As described previously, the site is generally blanketed by man-made fills. Our pits encountered fills ranging from about 2 feet to 7 feet deep. In addition, we expect soft soils to exist in areas exposed to seepage, ponded water, and creek flows. These fills and soils are weak and compressible under the planned improvements. In addition, the exploratory pits shown on **Figure 2** were loosely backfilled and wheel-rolled.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, retaining walls, exterior flatwork, and pavements), we recommend these weak fills, weak soils, and exploratory pit backfill be completely removed and re-compacted. Deeper removal will be needed in areas where thicker weak fills and soils are encountered during grading. The over-excavation should extend to depths and widths where competent soils are encountered. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork and pavement wherever possible.

Where the over-excavation limits abut adjacent existing structures or improvements, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent structures or improvements are not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation.

The removed fill and soil materials may be used as new fill onsite provided they satisfy the recommendations provided in **Section 5.1.5, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 5.1.6, *Compaction***.

Dewatering and/or collecting water within a piping system may be needed prior to performing over-excavation and re-compaction of the fills and weak soils. We recommend grading contractors review this report and inspect the site as part of the bid process.

5.1.3 Building Pads

To reduce the potential for post-construction distress to the proposed structures resulting from swelling and shrinkage of the clayey materials, we recommend at least 3-feet of well-blended,

moisture conditioned, engineered fill be provided below building foundations and exterior slabs. This 3-foot thick fill layer should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways).

5.1.4 Subgrade Preparation

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

If completed building pads and pavement subgrades are allowed to remain exposed to sun, wind or rain for an extended period of time, are heavily disturbed by vehicle traffic or animal borrowing, or experience vegetation growth, the exposed pads and subgrades may need to be reconditioned (moisture conditioned and/or scarified and recompacted) prior to foundation or pavement construction. SFB should be consulted on the need for pad and subgrade reconditioning.

5.1.5 Fill Material

From a geotechnical and mechanical standpoint, onsite soil and fill materials having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. Larger sized rock may be used as fill onsite provided it is closely monitored, placed properly to achieve compaction, and are located at depths below anticipated, future excavations; SFB should be consulted regarding the use of larger rock pieces in fill materials. Imported fill should have a plasticity index of 30 or less and have a significant amount of cohesive fines.

In addition to the mechanical property specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for the onsite soils, a pH of between approximately 6.0 and 8.5, a total water-soluble chloride concentration less than 300 ppm, and a total water-soluble sulfate concentration less than 500 ppm. We recommend import samples be submitted for corrosion and geotechnical testing at least two weeks prior to being brought onsite.

5.1.6 Compaction

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted between 88 to 92 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 90 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 3 to 5 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be

compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in un-compacted thickness.

5.1.7 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in un-compacted thickness. Thicker lifts can be used provided the method of compaction is approved by SFB and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only. Jetting is not permitted.

Onsite trench backfill should be compacted to at least 95 percent relative compaction. Imported sand trench backfill should be also compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction. To reduce piping and settlement of overlying improvements, we recommend rock bedding and rock backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of rock bedding and rock backfill.

Sand or gravel backfilled trench laterals that extend toward driveways, exterior slabs-on-grade, or under the building foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trench laterals should be located below the edge of pavement or slabs, and under the perimeter of the foundation. The plug should be at least 24 inches thick, extend across the entire width of the trench, and extend from the bottom of the trench to the top of the sand or gravel backfill.

Where the bottoms of trenches are sloped steeper than 5 percent, we recommend a low permeability plug composed of low strength concrete or sand/cement slurry be installed in the utility trenches every 50 feet on-center. The plug will reduce piping/consolidation from water seepage that may cause roadway and trench surface settlement. The plug should be at least 12 inches thick and extend to within 1 foot of the finished ground surface or to the base of the pavement section.

5.1.8 Exterior Flatwork

We recommend that exterior slabs (including such as driveways, sidewalks, patios, exterior flatwork, etc.) be placed directly on the properly compacted fills. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, pipe, and/or heave upward. Prior to placing concrete,

subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 to 5 percent above laboratory optimum moisture (ASTM D-1557).

The soils at the site will be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 18 inches on center in both directions should be installed. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slab. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of smooth dowels at all expansion and cold joints will reduce differential slab movements; the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. The dowels must be centered within the slabs. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slab are properly reinforced.

We do not recommend the use of flatwork having permeable joints (such as pavers or tiles with sand or gravel infilled joints) unless the underlying soil subgrade is protected against water seepage or ponding. If not protected, the underlying subgrade will heave, settle, and/or pipe and cause damage to the overlying improvements.

5.1.9 Construction During Wet Conditions

If construction proceeds during or shortly after wet weather conditions, or occurs in areas impacted by the existing groundwater, the moisture content of onsite soils will likely be significantly above optimum. Consequently, subgrade preparation, placement and/or reworking of onsite soil or fills as engineered fill might not be possible. Alternative wet construction recommendations can be provided by our representative in the field at the time of construction, if appropriate. All the drainage measures recommended in this report should be implemented and maintained during and after construction, especially during wet conditions.

5.1.10 Surface Drainage, Irrigation, and Landscaping

Ponding of surface water must not be allowed on pavements, adjacent to foundations, at the top or bottom of slopes, and at the top or adjacent to retaining walls. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including walkways and the

driveway. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

Surface drainage should be designed in accordance with the latest edition of the California Building Code to direct surface water away from the foundations and toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge the collected water into appropriate water collection facilities.

In order to reduce differential foundation movements, landscaping (where used) should be placed uniformly adjacent to foundations and exterior slabs. We recommend trees be no closer to structures or exterior slabs than half the mature height of the tree; in no case should tree roots be allowed to extend near or below foundations or exterior slabs.

Drainage inlets should be provided within enclosed planter areas and collected water should be discharged onto pavement, into drainage swales, or into storm water collection systems. In order to reduce the potential for water seepage, consideration should be given to lining planting areas and collecting the accumulated water in subdrain pipes that discharge to appropriate collection facilities. The drainage should be designed and constructed so that the moisture content of the soils surrounding the foundations do not become elevated and no ponding of water occurs. The inlets should be kept free of debris and be lower in elevation than the adjacent ground surface.

We recommend regular maintenance of the drainage systems be performed, including maintenance prior to rainstorms. The inspection should include checking drainage patterns to make sure they are performing properly, making sure drainage systems and inlets are functional and not clogged, and checking that erosion control measures are adequate for anticipated storm events. Immediate repairs should be performed if any of these measures appears to be inadequate.

Irrigation should be performed in a uniform, systematic manner as equally as possible on all sides of the foundations and exterior slabs to maintain moist soil conditions. Over-watering must be avoided. To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and low flow watering systems be used. All irrigation systems should be regularly inspected for leakage.

5.1.11 Subsurface Drainage

We recommend subdrains be installed below all fill materials, at the perimeter of the planned lots, and below pavement subgrade. Subdrain water should be collected in solid pipes that discharge to the North Babb Creek piping system. The location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction.

Where used, subdrains should consist of a 4-inch diameter, rigid perforated pipe (perforations down) surrounded by free draining, uniformly graded, 1/2- to 3/4-inch crushed gravel wrapped in filter fabric such as Mirafi 140N or equivalent. The pipe should be underlain by about 1/2 to 1 inch of gravel, and on the sides by at least 4 inches of gravel. The filter fabric should overlap approximately 12 inches or more at joints. Subdrains should be connected to a solid, rigid, collector pipe with a minimum diameter of 4 inches. Subdrain pipes should consist of rigid PVC SDR-35 or PVC A-2000 (or equal). Collector pipes should be connected to appropriate discharge facilities such as storm drains. Subdrain clean-outs should be provided. The clean-out locations should be based upon the reach of the rotary cleaning systems and the restrictions of pipe bends. Caltrans Class 2 permeable material may be used in lieu of gravel and filter fabric.

Where used, subdrain trenches should be at least 12 inches wide and about 4 feet deep below adjacent ground surface. If a subdrain trench extends to the ground surface and is not covered with concrete lined ditch or concrete flatwork, we recommend the subdrain trench be covered with a 12-inch thick cap consisting of native soil compacted to at least 90 percent relative compaction.

5.1.12 Setbacks

We recommend the project Civil Engineer determine improvement and building setbacks based upon the recommendations provided in this report, the surrounding existing conditions, the California Building Code, and local ordinances, and any other restrictions. SFB should be consulted on the need for setbacks or deepening of foundations where planned improvements will be located near existing structures.

5.1.13 Storm Water Treatment Facilities

To satisfy local and state permit requirements, most new development projects must control pollutant sources and reduce, detain, retain, and/or treat specified amounts of storm water runoff. The intent of these types of storm water treatment facilities is to conserve and incorporate on-site natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes. These facilities include bio-retention swales and basins, porous paver and pavement, water detention basins, and any proprietary underground storage and treatment systems.

In general, we recommend the portion of the storm water treatment facilities that are within 10 feet of structure foundations and improvements (such as building foundations, exterior flatwork, and pavements) be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage and distress to the adjacent structures and improvements. The lining can consist of a relatively impermeable membrane such as STEGO Wrap 15-mil or equivalent. The

membrane should be lapped and sealed in accordance with the manufacturer's specifications, including taping joints where pipes penetrate the membrane.

Soil filter/bio-mix materials within basins and swales will consolidate over time causing long-term ground surface settlement. Additional filling within the basins and swales over time will be needed to maintain design surface elevations. The soil filter/bio-mix materials, infiltration testing and procedures, and associated compaction requirements should be specified by the Civil Engineer and shown in detail on the grading and improvement plans.

Soil filter/bio-mix materials provide little to no lateral restraint of excavation side walls. Sidewalls of bio-retention swale and basin excavations (excavations made prior to the installation of the soil filter/bio-mix) steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause distresses to adjacent improvements such as foundations, utilities, pavements, driveways, walkways, and curbs and gutters. The magnitude and rate of movement depend upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 or flatter excavation sidewall slopes be used for bio-retention swales and basins, sidewalks be setback at least 3 feet from the top of slopes, and creep sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of slopes. If the above sidewall slope and setback distance cannot be met, considerations should be given to using below-grade concrete sidewalls that are designed and constructed as retaining walls. Alternatively, deepened sidewalk slab edge or roadway curbs can be used and designed to resist lateral earth pressures and act as a retaining wall. SFB should be consulted to evaluate the need for sidewall restraint when swales or basins are planned. We also recommend SFB observe and document the installation of liners, subdrain pipes, and soil filter/bio-mix materials during construction for conformance to the recommendations in this report and the development's plans and specifications.

Where used, proprietary underground storage and treatment systems should be installed and maintained in accordance with the manufacturer's specifications. In addition, the manufacturer should be consulted for vertical and lateral bearing capacities and anticipated deformations of these systems if they will also support exterior slabs and pavements that are subjected to vehicular traffic.

5.1.14 Future Maintenance

In order to reduce water related issues, we recommend regular inspection and maintenance of the site be performed, including maintenance prior to rainstorms. Inspections should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made.

Maintenance should include the re-compaction of loosened soils, collapsing and infilling holes with compacted soils or low strength sand/cement grout, removal and control of digging animals, modifying storm water drainage patterns to allow for sheet flow into drainage inlets or ditches rather than concentrated flow or ponding, removal of debris within drainage ditches and inlets, and immediately repairing any erosion or soil flow.

Differential movement of exterior slabs can occur over time as a result of numerous factors. We recommend development owners perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

5.1.15 Additional Recommendations

We recommend that the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors, and we recommend they be also included in disclosure statements given to the owners and their maintenance groups.

5.2 Foundation Support

5.2.1 Post-Tensioned Slab

The proposed residential buildings can be supported on a post-tensioned slab foundation that is designed for the expansion potential of onsite soils. The slab foundation should bear entirely on at least 3 feet of well-blended, moisture conditioned, properly prepared engineered fills. In no case should a slab foundation bear upon fills with differential expansion characteristics. Recommendations for building pad preparation are described previously in **Section 5.1.3, *Building Pads***, and **Section 5.1.4, *Subgrade Preparation***. Prior to the concrete pour, we recommend the moisture content of subgrade materials be conditioned to approximately 3 to 5 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support. We also recommend SFB review the foundation drawings and specifications prior to submittal to verify that the recommendations provided in this report have been used and properly interpreted in the design of the slabs.

The thickness of post-tensioned slab should be determined by the Structural Engineer; however, we recommend the post-tensioned slabs be at least 10 inches thick. In addition, deflection of the slab foundations should be designed not to exceed the values recommended in the most recent PTI design manual. An allowable bearing pressure of 1,500 pounds per square foot can be used for localized point and line loads. We estimate the maximum total static settlement of PT-slab

foundations under the recommended allowable bearing pressure to be on the order of 1 inch or less. Differential static settlement is estimated to be approximately 1/2 inch or less.

Lateral loads, such as derived from earthquakes and wind, can be resisted by friction between the post-tensioned slab foundation bottom and the supporting subgrade. A friction coefficient of 0.25 is considered applicable.

At least 10 feet of cover should be provided between the outer face of slabs and un-retained slope faces, as measured laterally between slope faces and the slabs. Where less than 10 feet of cover exists, deepening of the edge of slabs may be necessary in order to achieve 10 feet of cover for buildings located near tops of slopes. Where slabs are located adjacent to utility trenches, the slab bearing surface should bear below an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the slab reinforcing could be increased to span the area defined above assuming no soil support is provided.

A vapor retarder must be placed between subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil Class A or equivalent provided the equivalent satisfies the following criteria: a permeance as tested before and after mandatory conditioning of less than 0.01 Perms and strength of Class A as determined by ASTM E 1745 (latest edition), and a thickness of at least 15 mils. Installation of the vapor retarder should conform to the latest edition of ASTM E 1643 (latest edition) and the manufacturers requirements, including lapping and all joints at least 6 inches and sealing with Stego Tape or equal in accordance with the manufacturer's specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer's instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry. Any water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor which will be trapped under impermeable flooring. The concrete mix design for slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete. If a higher water/cement ratio is being considered, we recommend higher vapor transmission be taken into account in the design and construction of the buildings. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The results of sulfate and chloride testing of onsite soil samples are included in **Appendix B**; the foundation designer should determine if additional testing is needed. In

addition, we recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

During the curing process, concrete slabs will shrink in volume resulting in cracks developing in the slab. Curing of concrete can take many months (or possibly longer) to complete. These concrete cracks may be visible on the surface of the slab during and after the curing process. In order to reduce the potential for crack propagation through overlying brittle surfaces such as tile or stone flooring, we recommend appropriate crack isolation measures be used between the concrete slab and flooring to reduce the potential for slab cracks to propagate into these brittle flooring surfaces.

An experienced Structural Engineer should design the post-tensioned slabs to resist the differential soil movement. The preliminary soil design parameters presented below were generated using the procedures presented in the 3rd edition of the PTI design manual (2008)[17], PTI standard requirements (2019)[18], and a PTI preferred computer program, VOLFLO (Version 1.5 Build 120704), was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted. Modifications to the design values below would need to be made in writing. Please refer to **Section 5.1.10*, Surface Drainage, Irrigation, and Landscaping***, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

SWELLING MODE

	Center Lift	Edge Lift
Edge Moisture Variation Distance (e_m)	9.0 feet	5.0 feet
Differential Soil Movement (y_m)	1.0 inch	1.5 inch

[17]Post-Tensioning Institute, 2008, Design of Post-Tensioned Slabs-On-Ground (PTI DC10.1-08), Third Edition.
[18]Post-Tensioning Institute, 2019, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-19).

5.2.5 Retaining Walls

If segmental block walls with geogrid will be used at the site, SFB should be contacted to provide block wall and geogrid designs and specifications.

Any walls that retain soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway surcharging, earthquake loading, and hydrostatic pressure if wall back-drainage is not provided. The global stability of the walls should also be evaluated where the walls will be located on slopes or where multi-tiered walls will be used.

If walls are allowed to deflect or rotate (unrestrained walls), they can be designed to resist active pressures. If no movement is allowed at the top of walls (restrained walls), at-rest pressures should be used in wall design. The recommended active and at-rest lateral earth pressures under both drained and undrained conditions are provided in the table below.

LATERAL EARTH PRESSURES FOR RETAINING STRUCTURES

Wall Condition	Backfill Condition	Drained Equivalent Fluid Pressure (pcf)	Undrained Equivalent Fluid Pressure (pcf)	Incremental Seismic Pressure (pcf)
Unrestrained (Active Pressure)	Level	50	85	46
Restrained (At-Rest Pressure)		75	100	92*

*Note: For restrained walls, use the static active pressure and seismic increment in the seismic design.

For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that retain buildings, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher than 6 feet (as required by the 2022 CBC), we recommend the walls be designed to also resist an incremental seismic lateral earth pressure listed in the above table, using a triangular fluid pressure distribution (not inverted). This seismic induced earth pressure is in addition to the active pressures listed above. The seismic lateral earth pressures were estimated based on the half of the peak ground acceleration from a Maximum Considered Earthquake (MCE) earthquake per ASCE 7-16/2022 CBC (0.5 x PGA_M). Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Any surcharge loads located within an imaginary 1:1 (horizontal to vertical) plane projected upward from the base of the walls will increase the lateral earth pressures on the wall. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure (rectangular distribution) equal to one-third (0.33) and one-half (0.5) the anticipated surcharge load for unrestrained and restrained walls, respectively. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (250 psf). We should be consulted to provide load contributions from other particular surcharges located behind walls if needed.

It should be noted the lateral earth pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

The above recommended drained lateral earth pressures assume walls are fully back drained to prevent the build-up of hydrostatic pressures. If drainage behind the wall is omitted, the wall should be designed for undrained condition. Wall back-drainage can be accomplished by using 1/2- to 3/4-inch crushed, uniformly graded gravel entirely wrapped in filter fabric, such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 12 inches wide and extend from the base of the wall to within about 1 foot of the finished grade at the top (Class 2 permeable material per Caltrans Specification Section 68 may be used in lieu of gravel and filter fabric). The upper 1 foot of cover backfill should consist of relatively impervious material.

Where wall back-drainage is used, a 4-inch diameter, perforated, PVC SDR-35 pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to suitable discharge facilities. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. Weepholes can be spaced at about 10 to 15 feet apart. As an alternative to using gravel, pre-fabricated drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal).

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in **Section 5.1.5, *Fill Material***, and **Section 5.1.6, *Compaction***.

Retaining walls can be supported on drilled, cast-in-place, straight shaft friction piers that develop their load carrying capacity in the materials underlying the site. The piers should have a minimum diameter of 12 inches and a center-to-center spacing of at least three times the shaft diameter. We recommend that piers be at least 6 feet long. Pier reinforcing should be based on structural requirements, but in no case should less than two #4 bars for the entire length of the pier be used.

The actual design depth of the piers should be determined using an allowable skin friction of 500 pounds per square foot (psf) for dead plus live loads, with a one-third increase for all loads including wind or seismic. Eighty percent of the skin friction value can be used to resist uplift. Lateral load resistance can be developed in passive resistance for pier foundations. We recommend an allowable soil passive resistance (which includes a factor of safety of 1.5) equal to an equivalent fluid weighing 300 pounds per cubic foot be used for pier foundations. This value can be used up to a maximum value of 3,600 psf. The passive resistance can be applied against twice the projected diameter of pier shaft if the piers are spaced center-on-center at least 3 times of the pier shaft diameter.

The upper 3 feet of pier embedment should be neglected in the vertical and passive resistance design as measured from finished grade unless it is confined by a pavement or concrete slab. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face or above an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom of any adjacent walls or utility trenches should also be ignored in both the vertical bearing and passive resistance designs.

The bottoms of pier excavations should be relatively dry and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavation should be removed prior to placing concrete. We recommend that the excavation of all piers be performed under the direct observation of SFB to confirm that the pier foundations are founded in suitable materials and constructed in accordance with the recommendations presented herein. Preliminarily, we recommend concrete pour of pier excavations be performed within 24 hours of excavation and prior to any rainstorms. Where caving or high groundwater conditions exist, additional measures such as using dewatering, casing, slurry, tremie methods, and/or pouring concrete immediately after excavating may be necessary. SFB should be consulted for additional measures for pier construction as needed during construction.

As an alternative to using pier foundations to support the walls, footings may be used. Please contact SFB for footing foundation recommendations if footings will be used to support the walls.

5.3 Seismic Design Criteria

Based on the site geology and subsurface soil conditions encountered at the site, we recommend the site be characterized as Site Class "D", a "stiff soil" profile. For seismic designs using the 2022 CBC and ASCE 7-16, we recommend the following seismic design parameters be used. These parameters were calculated using the ASCE 7 Hazard Tool online program[19], and are based on the site being located at approximate latitude 37.368389°N and longitude 121.809067°W. We assumed the proposed project structures are categorized as Risk Category II, and the *Exception Number (2) of ASCE 7-16 Section 11.4.8 – Site Specific Ground Procedure* will be taken by the Structural Engineer for the project. We should be contacted if any of these assumptions are incorrect or a site-specific ground motion hazard analysis is required.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	D
S_S	2.078
S_1	0.8
F_a	1
F_v	See Section 11.4.8 of ASCE 7-16*
S_{MS}	2.078
S_{M1}	See Section 11.4.8 of ASCE 7-16*
S_{DS}	1.385
S_{D1}	See Section 11.4.8 of ASCE 7-16*
SDC	See Section 11.4.8 of ASCE 7-16*
PGA	0.874
F_{PGA}	1.1
PGA_M	0.961
T_L	12

*Note: The values of F_v, S_{M1}, S_{D1}, and Seismic Design Category (SDC) should be determined by the Structural Engineer based on the ASCE 7-16 Section 11.4.8 requirements.

5.4 Pavements

Based on the results of borings and laboratory testing, we recommend that an R-value of 5 be used in preliminary asphalt concrete pavement design. We recommend additional R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design.

[19]ASCE, https://asce7hazardtool.online/, accessed 10/2/2024.

Pavement subgrade completely composed of sandy and gravelly fills will result in higher R-values and thinner pavement sections.

We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices for residential developments. The project's Civil Engineer or appropriate public agency should determine actual traffic indices. The pavement thicknesses shown below are SFB's recommended minimum values; governing agencies may require pavement thicknesses greater than those shown.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 5			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	9.0	12.0
T.I. = 5.0 (access ways/courts)	3.0	11.0	14.0
T.I. = 6.0 (primary roadways)	3.0	14.0	17.0

If the pavements are planned to be placed prior to or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. If the pavement sections will be used for construction access by heavy trucks or construction equipment (especially fork lifts with outriggers), SFB should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use and heavier loads. If requested, SFB can provide recommendations for a phased placement of the asphalt concrete to reduce the potential for mechanical scars caused by construction traffic in the finished grade. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.

We recommend the pavement materials and construction conform to Caltrans Standard Specifications. Pavement aggregate base and asphalt concrete should be compacted to at least 95 percent relative compaction as determined by ASTM D1557 or Caltrans Test Method 375. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans.

We recommend regular maintenance of the asphalt concrete be performed at approximately five-year intervals. Maintenance may include sand slurry sealing, crack filling, and chip seals as necessary. If regular maintenance is not performed, the asphalt concrete layer could experience premature degradation requiring more extensive repairs.

6.0 CONDITIONS AND LIMITATIONS

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

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

This report is a design document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of VIAM Fleming Partners, LLC and their consultants for specific application to the proposed residential development project located in San Jose, California, and is intended to represent our design recommendations to VIAM Fleming Partners, LLC for specific application to the proposed project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of VIAM Fleming Partners, LLC to transmit the information and recommendations of this report to those designing and constructing the project. We will not be responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to review geological and geotechnical aspects of construction calculations, specifications, and plans; we should also be retained to participate in pre-bid and pre-construction conferences to clarify the opinions, conclusions, and recommendations contained in this report.

It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the

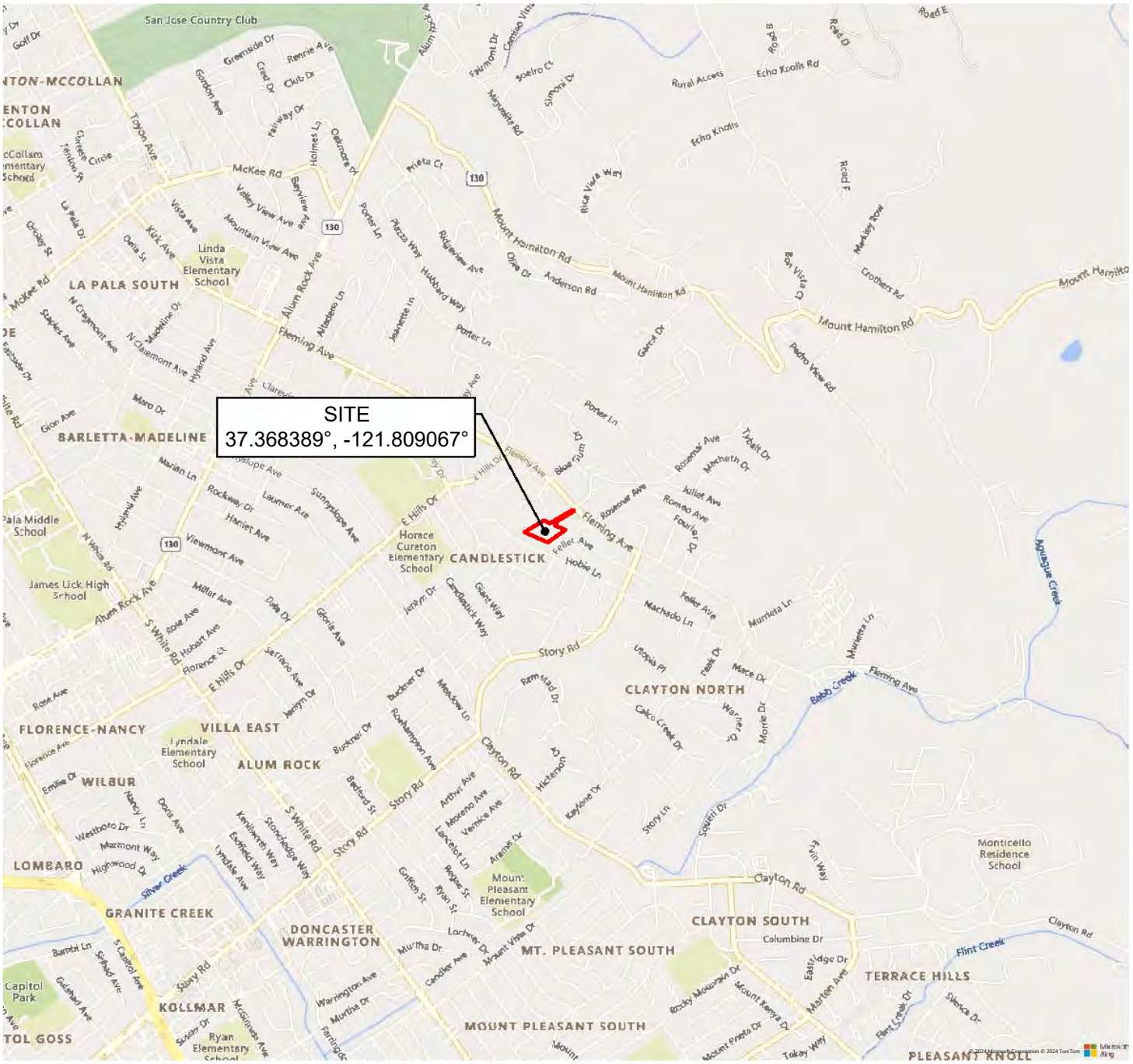
validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore, we should be consulted if the limitations to using this are not completely understood.

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

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

Please refer to **Appendix D** for Geoprofessional Business Association (GBA) guidelines regarding use of this report.

FIGURES



N

SCALE: 1" = 2,000'

0 2,000' 4,000'

NOTE: Basemap from Microsoft/TomTom 2024.

DATE		VICINITY MAP	FIGURE
October 2024	Stevens Ferrone & Bailey Engineering Company, Inc *1600 Willow Pass Court* *Concord, CA 94520* *Tel 925.688.1001* *Fax 925.688.1005* *www.SFandB.com*	761 FLEMING AVENUE San Jose, California	1
PROJECT NO.			
1084-1			

1084-1, 761 Fleming Avenue, San Jose - October 2024.dwg
Printed: 10/11/2024



KEY

Qaf	Artificial Fill (Estimated More Than 2 Feet Thick
Qa	Alluvium
Qls	Approximate Westerly Limit of Dormant Mature (Probable) Landslide Per CGS
	Observed Surficial Seepage and Water Ponding (Other Locations Likely)
	Channelized Water With Flow Direction; Water Channel Approximately 2" to 6" Deep (Other Locations Likely)
	Exploratory Pit by SFB (9/26/2024)
	Exploratory Boring by Engeotech (3/11/1988)
	Test Boring by Applied Soil Mechanics (1/4/1973)
	Test Boring by Peninsula Testing And Controls (7/8/1960)
	Approximately Project Limits

NOTES: All Locations and Limits Are Approximate.

NOTE: Basemap from the project vesting tentative tract map Sheets 4.0 prepared by HMH and dated 8/7/2024. Aerial photo imagery from Microsoft 2024.

SCALE: 1" = 60'

0 60' 120'

DATE	PROJECT NO.
October 2024	1084-1

Stevens Ferrone & Bailey Engineering Company, Inc
1600 Willow Pass Court
Concord, CA 94520
Tel 925.688.1001
Fax 925.688.1005
www.SFandB.com

SITE PLAN AND ENGINEERING GEOLOGY MAP

761 FLEMING AVENUE

San Jose, California

FIGURE 2

1084-1, 761 Fleming Avenue, San Jose - October 2024.dwg
Printed: 10/1/2024

APPENDIX A

Field Exploration

Stevens Ferrone Bailey Engineering

1600 Willow Pass Court
Concord, CA 94520
Tel: (925) 688-1001

KEY TO FIELD EXPLORATION LOGS

PROJECT:

761 Fleming Ave
San Jose, California

PROJECT NO: **1084-1**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487 & D2488)

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (More than 50% retained on #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures, trace or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, trace or no fines
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands, gravelly sands, trace or no fines
			SP	Poorly-graded sands, gravelly sands, trace or no fines
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 50% passes #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands, clayey silts of low to medium plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS (Liquid Limit 50 or more)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SIEVE SIZE

12"	3"	3/4"	#4	#10	#40	#200

BOULDERS	COBBLES	GRAVELS		SANDS			SILTS AND CLAYS
		Coarse	Fine	Coarse	Medium	Fine	

304.8 mm	76.2 mm	19.0 mm	4.75 mm	2.00 mm	0.425 mm	0.075 mm

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT*	UCS (KSF)**
Very Soft	0 - 2	0 - 1/2
Soft	2 - 4	1/2 - 1
Firm	4 - 8	1 - 2
Stiff	8 - 16	2 - 4
Very Stiff	16 - 32	4 - 8
Hard	Over 32	Over 8

*Number of blows for a 140-pound hammer falling 30 inches to drive a 2" O.D. (1-3/8" I.D.) split spoon sampler.

**UCS: Unconfined Compressive Strength.

SYMBOLS AND NOTES

- Standard Penetration Test (SPT) Sampler (2" O.D. Split Barrel)
- Modified California Sampler (3" O.D. Split Barrel)
- California Sampler (2.5" O.D. Split Barrel)
- Shelby Tube
- Bulk Sample
- Core Barrel (See Log Notes)
- Groundwater Level During Drilling
- Groundwater Level at End of Drilling

INCREASING VISUAL MOISTURE CONTENT

Wet
Moist
Dry

CONSTITUENT PERCENTAGE

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

Stevens Ferrone Bailey Engineering
1600 Willow Pass Court
Concord, CA 94520
Tel: (925) 688-1001

EXPLORATORY PIT EP-1

PROJECT NO: 1084-1	SURFACE ELEVATION: 222 feet
LOGGED / CHECKED BY: K. Ferrone/ C. Becker	DATE STARTED: 9/26/2024
EXCAVATOR TYPE: Deere 410L	DATE FINISHED: 9/26/2024
BUCKET WIDTH: 30 inches	DEPTH TO INITIAL WATER: Not Encountered
PIT LENGTH: 9 feet	DEPTH TO FINAL WATER: Not Encountered
PIT WIDTH: 3 feet	
PIT LOCATION: See Site Plan, Figure 2 (37.368410°;-121.808374°)	

PROJECT:

761 FLEMING AVE

San Jose, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	POCKET PEN. (TSF)	TORVANE (TSF)	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: CLAY (CL), mottled brown and gray, silty, with sand (fine- to medium-grained), dry.	firm		0							
CLAY (CH): brown, silty, with sand (fine- to medium-grained), dry to damp. Sample at 4 feet	stiff to hard				>4.5					
CLAY (CH): blackish gray, silty, with sand (fine- to coarse-grained), trace gravel (fine, subrounded), damp. Sample at 6 feet	very stiff		5		2.75		20.7			At 6 Feet: Liquid Limit = 56% Plasticity Index = 39 Passing #200 Sieve = 73.0%
Bottom of Pit = 10 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			10							
			15							

EXPLORATORY PIT LOG 1084-1 EP-1.lda8 STEVENS FERRONE & BAILEY 10/9/2024

Stevens Ferrone Bailey Engineering
1600 Willow Pass Court
Concord, CA 94520
Tel: (925) 688-1001

EXPLORATORY PIT EP-2

PROJECT NO: 1084-1	SURFACE ELEVATION: 216 feet
LOGGED / CHECKED BY: K. Ferrone/ C. Becker	DATE STARTED: 9/26/2024
EXCAVATOR TYPE: Deere 410L	DATE FINISHED: 9/26/2024
BUCKET WIDTH: 30 inches	DEPTH TO INITIAL WATER: Seepage
PIT LENGTH: 6 feet	DEPTH TO FINAL WATER: Seepage
PIT WIDTH: 3 feet	
PIT LOCATION: See Site Plan, Figure 2 (37.368662°;-121.808886°)	

PROJECT:

761 FLEMING AVE

San Jose, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	POCKET PEN. (TSF)	TORVANE (TSF)	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: CLAY (CL), mottled brown, silty, moist to wet.	soft to firm		0							Seepage from 0 to 2 feet (surface water seepage).
CLAY (CL): dark brown, silty, few sand (fine- to medium-grained), damp to moist.	stiff				1.5					
			5		1.8					
Bottom of Pit = 8 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			10							
			15							

Stevens Ferrone Bailey Engineering
1600 Willow Pass Court
Concord, CA 94520
Tel: (925) 688-1001

EXPLORATORY PIT EP-3

PROJECT NO: 1084-1	SURFACE ELEVATION: 225 feet
LOGGED / CHECKED BY: K. Ferrone/ C. Becker	DATE STARTED: 9/26/2024
EXCAVATOR TYPE: Deere 410L	DATE FINISHED: 9/26/2024
BUCKET WIDTH: 30 inches	DEPTH TO INITIAL WATER: 8 feet
PIT LENGTH: 6 feet	DEPTH TO FINAL WATER: 8 feet
PIT WIDTH: 3 feet	
PIT LOCATION: See Site Plan, Figure 2 (37.368392°;-121.809184°)	

PROJECT:

761 FLEMING AVE

San Jose, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	POCKET PEN. (TSF)	TORVANE (TSF)	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: CLAY (CL), brown, silty, with sand (fine-grained), damp, abuundant roots.	soft		0							
CLAY (CL): brown, silty, few sand (fine-grained), damp.	stiff				1.2					
CLAY (CL): gray, silty, trace sand (fine-grained), trace gravel (fine, subrounded), moist.	stiff									
Sample at 5 feet			5		1.5					At 5 Feet: Liquid Limit = 40% Plasticity Index = 27 Passing #200 Sieve= 55.9%
Sample at 6 feet										
Water flowing into pit at 8 feet.										
Bottom of Pit = 9 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			10							
			15							

EXPLORATORY PIT LOG 1084-1 EP-3.ldat8 STEVENS FERRONE & BAILEY 10/9/2024

Stevens Ferrone Bailey Engineering
1600 Willow Pass Court
Concord, CA 94520
Tel: (925) 688-1001

EXPLORATORY PIT EP-4

PROJECT NO: 1084-1	SURFACE ELEVATION: 229 feet
LOGGED / CHECKED BY: K. Ferrone/ C. Becker	DATE STARTED: 9/26/2024
EXCAVATOR TYPE: Deere 410L	DATE FINISHED: 9/26/2024
BUCKET WIDTH: 30 inches	DEPTH TO INITIAL WATER: 7 feet
PIT LENGTH: 6 feet	DEPTH TO FINAL WATER: 7 feet
PIT WIDTH: 3 feet	
PIT LOCATION: See Site Plan, Figure 2 (37.368242°;-121.809317°)	

PROJECT:

761 FLEMING AVE

San Jose, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	POCKET PEN. (TSF)	TORVANE (TSF)	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: CLAY (CL), brown, silty, few sand (fine-grained), dry to damp, abundant roots.	soft		0							
FILL: CLAY (CL): dark brown, silty, few sand (fine-grained), damp.	firm									
FILL: Gravel (GP), rounded cobbles and gravel, blocks of brick, damp.	loose		5							
CLAY (CL): gray, silty, trace sand (fine-grained), trace gravel (fine, subrounded), moist to wet. Water flowing into pit at 7 feet.	stiff				1.2					
Bottom of Pit = 9 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			10 15							

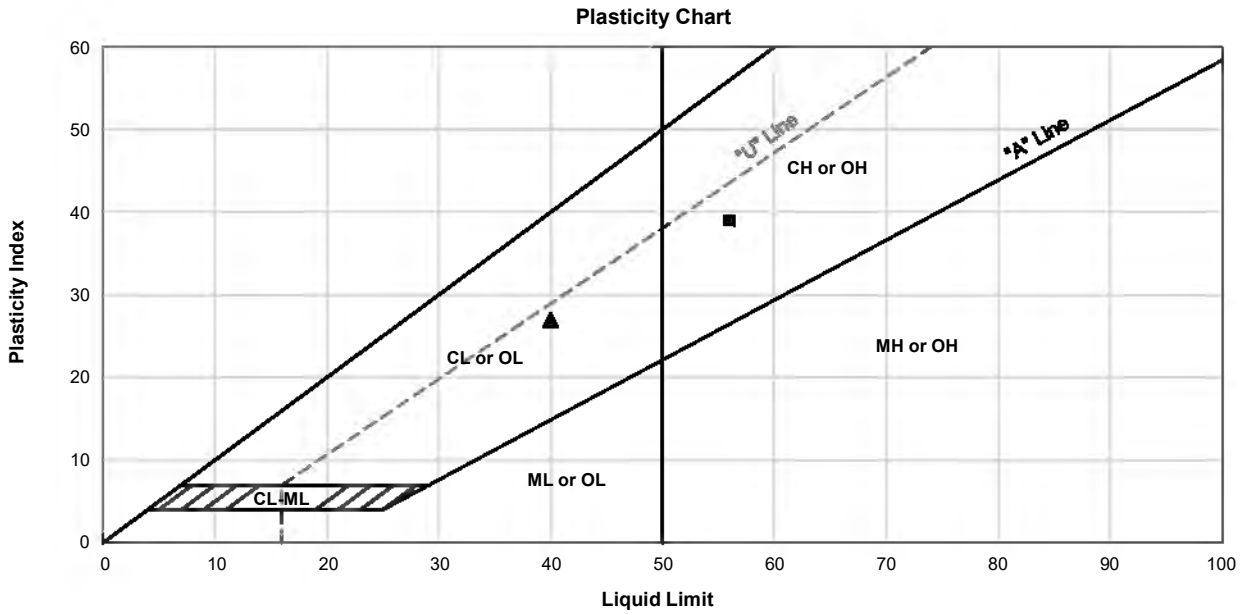
APPENDIX B

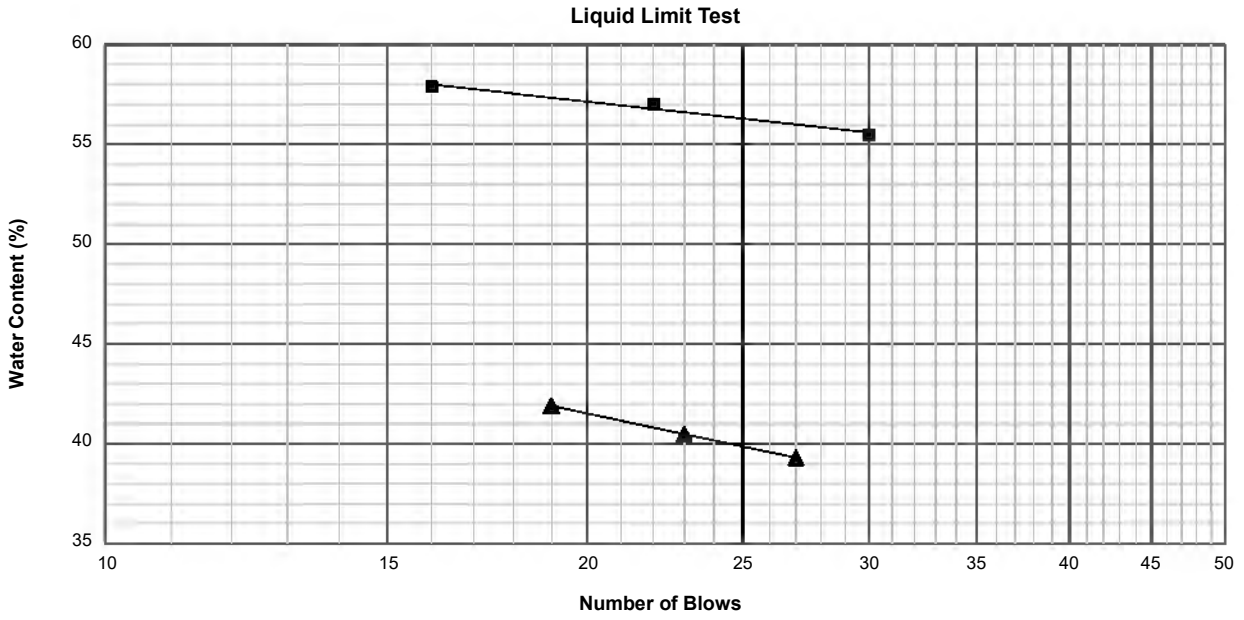
Laboratory Testing

Stevens Ferrone & Bailey Engineering Company, Inc

ATTERBERG LIMITS

ASTM D4318





	Symbol	Sample Source	Sample Description	LL	PL	PI	%≤ #200	USCS
Test Results	■	EP-1 at 6 ft.	Black silty CLAY with sand	56	17	39	73.0	CH
	▲	EP-3 at 6 ft.	Lt. gray brown sandy silty CLAY some siltstones	40	13	27	55.9	CL

Project Info

Project Number: 1084-1

Test Report Date: 10/3/2024

Project Name: Fleming

Project Location: San Jose, CA

Tested by: R. Tuazon

Checked by: T. Chen

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

30 September, 2024

Job No. 2409049
Cust. No. 11486

Mr. Ken Ferrone
Stevens, Ferrone & Bailey
1600 Willow Pass Court
Concord, CA 94520

Subject: Project No.: 1084-1
Project Name: Fleming
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Ferrone:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on September 27, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.002 is classified as "corrosive". Sample No. 001 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are none detected and 17 mg/kg. They are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations are none detected and 33 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 8.27 and 8.42, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are both 200-mV and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

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925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Stevens, Ferrone & Bailey Engineering
Client's Project No.: 1084-1
Client's Project Name: Fleming
Date Sampled: 26-Sep-24
Date Received: 27-Sep-24
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 30-Sep-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2409049-001	EP-1 @ 4'	200	8.27	-	2,500	-	N.D.	N.D.
2409049-002	EP-3 @ 5'	200	8.42	-	890	-	17	33

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reportring Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Sep-2024	30-Sep-2024	-	27-Sep-2024	-	30-Sep-2024	30-Sep-2024

Julia Clauson
Chemist

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

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

APPENDIX C

Previous Exploration Logs

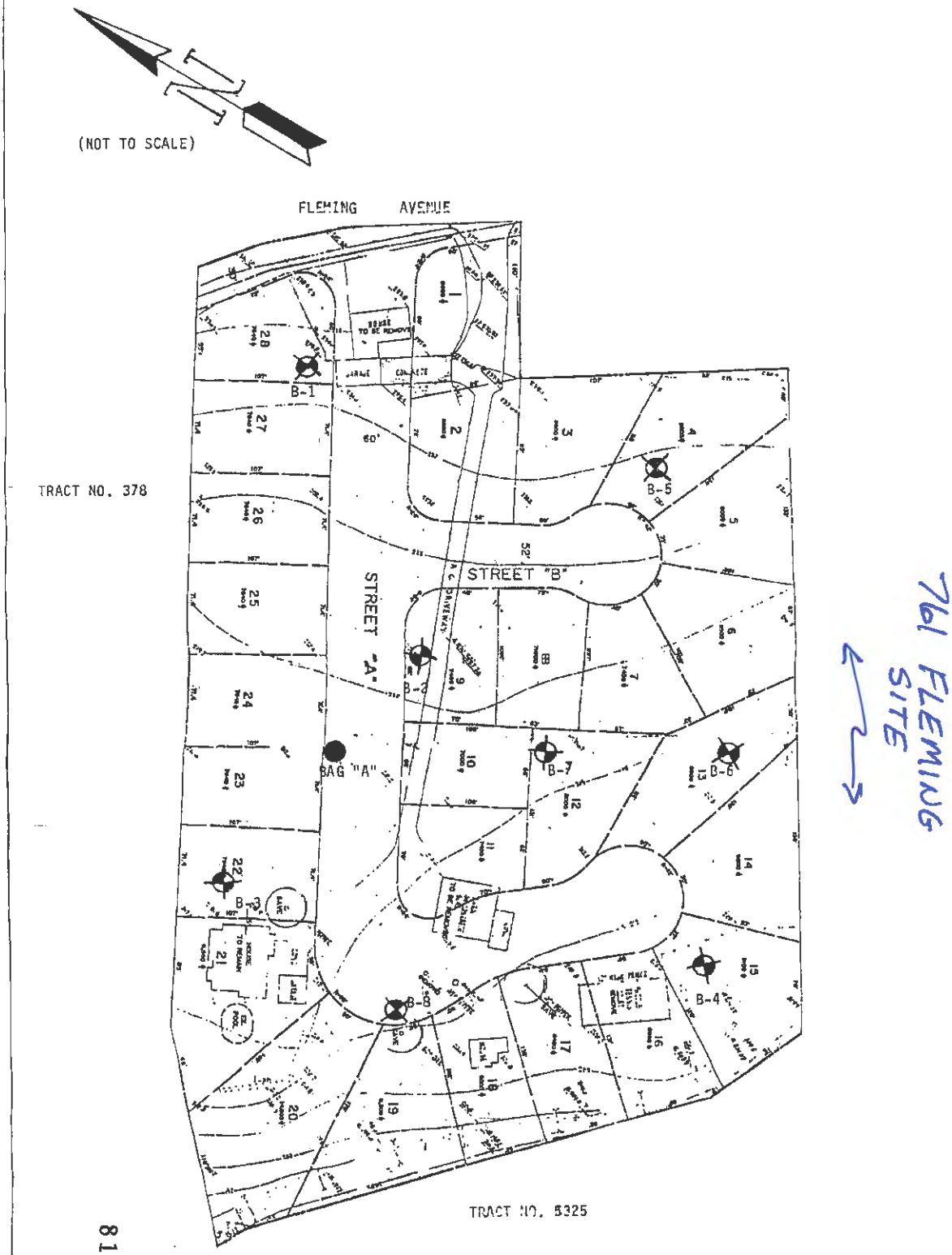


Figure 1 - Site Plan

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

BORING NO. B-1

DATE DRILLED 3-11-1988

LOGGED BY M.H.

IN PLACE							
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
					3		Dark brown silty Clay; Wet, stiff
102.0	22.1	28	3.7	1-1	3-4		
					5		Light brown silty Clay; Wet, very stiff Minor gravels
103.2	21.0	32		1-2	10-11		
					15		Light brown sandy Clay; Wet, very stiff
105.1	17.6	36		1-3	18-19		8159
					20		Similar material very stiff and slow drilling
105.8	17.2	34		1-4	26-27		Reddish brown sandy Clay; Wet, very stiff
					30		

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FIGURE NO. 2 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

DATE DRILLED 3-11-1988

BORING NO. B-1 (Contd.)

LOGGED BY M.H.

IN PLACE							
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
							Reddish brown sandy Clay; Wet, very stiff
							Boring terminated @ 40'

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FIGURE NO. 2 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

DATE DRILLED 3-11-1988

BORING NO. B-2

LOGGED BY M.H.

IN PLACE							
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
							Brown silty Clay; Wet, stiff
101.0	22.8	26		2-1	4-5		Light brown silty Clay; Wet, stiff
104.2	18.4	29		2-2	11-12		Reddish brown sandy Clay; Wet, very stiff Minor gravels
							Boring terminated @ 20'

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FIGURE NO. 3 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1 DATE DRILLED 3-11-1988

BORING NO. B-3 LOGGED BY M.H.

IN PLACE							
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
99.6	20.6	22		3-1	2-3		Dark brown silty Clay; Wet, stiff
							Light brown silty Clay; Wet, stiff
103.8	18.3	27		3-2	8-9		Brown sandy Clay; Wet, stiff
							Similar material more sand and minor gravels Borwn sandy Clay; Wet, stiff
106.0	18.0	30		3-3	15-16		
							Boring terminated @20'

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FIGURE NO. 4 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1 DATE DRILLED 3-11-1988

BORING NO. B-4 LOGGED BY M.H.

IN PLACE: DRY DENSITY P.C.F.	IN PLACE: MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
99.4	23.1	16		4-1	4-5		Dark brown silty Clay; Wet, medium stiff
103.8	19.8	21		4-2	9-10		Blue silty Clay; Wet, medium stiff
							Dark brown silty Clay; Wet, medium stiff
106.0	19.4	23		4-3	17-18		Grey silty Clay; Wet, stiff
							Boring terminated @ 20'

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SOIL AND GEOLOGICAL ENGINEERS

FIGURE NO. 5 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

DATE DRILLED 3-11-1988

BORING NO. B-5

LOGGED BY M.H.

IN PLACE							
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.	PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
							Brown silty Clay; Wet, stiff
100.8	21.3	25	3.5	5-1	3-4		Light brown silty Clay; Wet, stiff
104.8	18.2	31		5-2	12-13		Light brown sandy Clay; Wet, very stiff
105.7	18.9	37		5-3	19-20		Reddish brown sandy Clay; Wet, very stiff
							Boring terminated @ 25'

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SOIL AND GEOLOGICAL ENGINEERS

FIGURE NO. 6 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

BORING NO. B-6

DATE DRILLED 3-11-1988

LOGGED BY M.H.

IN PLACE		PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.						
99.0	18.6	21		6-1	2-3		Brown silty Clay; Wet, stiff
103.4	23.6	30		6-2	8-9		Light brown silty Clay; Wet, stiff
104.5	19.3	34		6-3	15-16		Reddish brown silty Clay; Wet, very stiff
							Grey sandy Clay; Wet, very stiff Minor gravels
							Boring terminated @ 20'

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FIGURE NO. 7 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

BORING NO. B-7

DATE DRILLED 3-11-1988

LOGGED BY M.H.

IN PLACE		PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.						
							Brown silty Clay; Wet, stiff
102.2	22.6	22		7-1	4-5		Light brown silty Clay; Wet, stiff
104.6	21.8	33		7-2	12-13		Reddish brown silty Clay; Wet, very stiff
							Boring terminated @ 20'

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SOIL AND GEOLOGICAL ENGINEERS

FIGURE NO. 8 - LOG OF TEST BORING

EXPLORATORY BORING LOG

JOB NO. ET8-0444-S1

DATE DRILLED 3-11-1988

BORING NO. B-8

LOGGED BY M.H.

IN PLACE		PENETRATION RESISTANCE BLOWS/FT.	UNCONFINED COMP. STRENGTH K.S.F.	SAMPLE NO.	DEPTH IN FT.	BORING LOG	DESCRIPTION & REMARKS
DRY DENSITY P.C.F.	MOISTURE CONTENT % DRY WT.						
98.7	21.4	29		8-1	3-4		Brown silty Clay; Wet, stiff
103.6	22.1	36		8-2	10-11		Light brown silty Clay; Wet, stiff to very stiff
106.1	19.2	38		8-3	19-20		Light brown sandy Clay; Wet, very stiff
							Reddish brown snady Clay; Wet, very stiff
							Boring terminated @ 25'

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FIGURE NO. 9 - LOG OF TEST BORING

TABLE I

SUMMARY OF MOISTURE, DENSITY,
UNCONFINED COMPRESSION AND DIRECT SHEAR TESTING

Sample No.	Depth Ft.	In-Place Conditions		Unconfined Compressive Strength k.s.f.	Direct Shear Testing	
		Moisture Content % Dry Wt.	Dry Density p.c.f.		Angle of Internal Friction Degrees	Unit Cohesion p.s.f.
1-1	3-4	22.1	102.0	3.7		
1-2	10-11	21.0	103.2			
1-3	18-19	17.6	105.1			
1-4	26-27	17.2	105.8			
2-1	4-5	22.8	101.0		19	970
2-2	11-12	18.4	104.2			
3-1	2-3	20.6	99.6			
3-2	8-9	18.3	103.8			
3-3	15-16	18.0	106.0			
4-1	4-5	23.1	99.4		20	940
4-2	9-10	19.8	103.8			
4-3	17-18	19.4	106.0			
5-1	3-4	21.3	100.8	3.5		
5-2	12-13	18.2	104.8			
5-3	19-20	18.9	105.7			

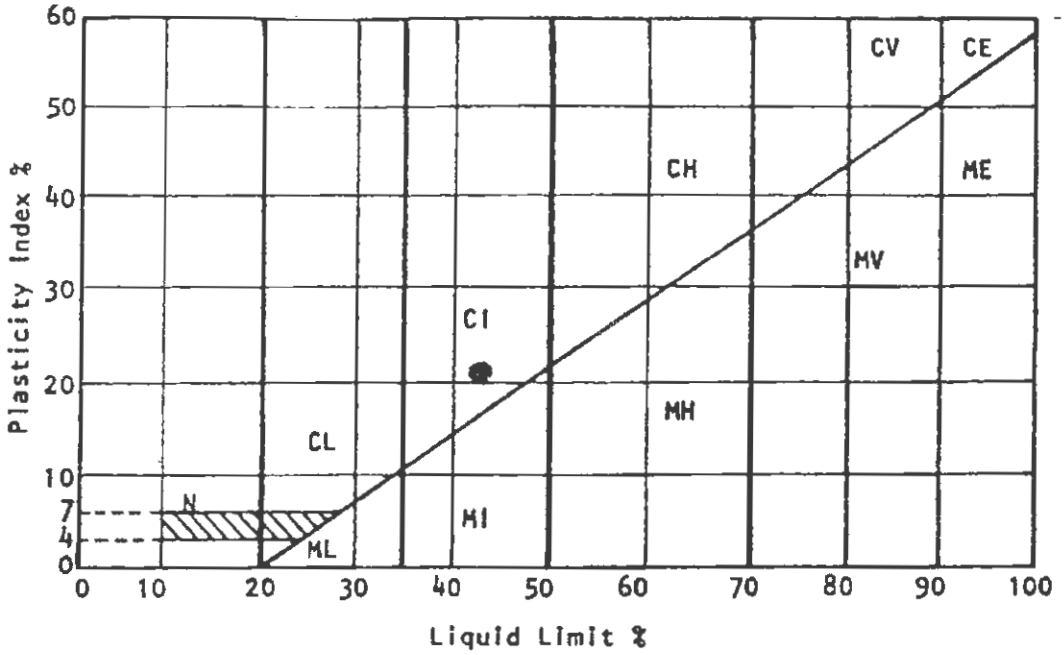
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TABLE I

SUMMARY OF MOISTURE, DENSITY,
UNCONFINED COMPRESSION AND DIRECT SHEAR TESTING

Sample No.	Depth Ft.	In-Place Conditions		Unconfined Compressive Strength k.s.f.	Direct Shear Testing	
		Moisture Content % Dry Wt.	Dry Density p.c.f.		Angle of Internal Friction Degrees	Unit Cohesion p.s.f.
6-1	2-3	18.6	99.0			
6-2	8-9	23.6	103.4			
6-3	15-16	19.3	104.5			
7-1	4-5	22.6	102.2			
7-2	12-13	21.8	104.6			
8-1	3-4	21.4	98.7			
8-2	10-11	22.1	103.6			
8-3	19-20	19.2	106.1			

PLASTICITY CHART



PLASTICITY DATA

Key Symbol	Hole No.	Depth Ft.	Liquid Limit %	Plasti-city Index %	Unified Soil Clas-sification Symbol*
●	BAG "A"	1-1.5'	42.0	20.6	CI

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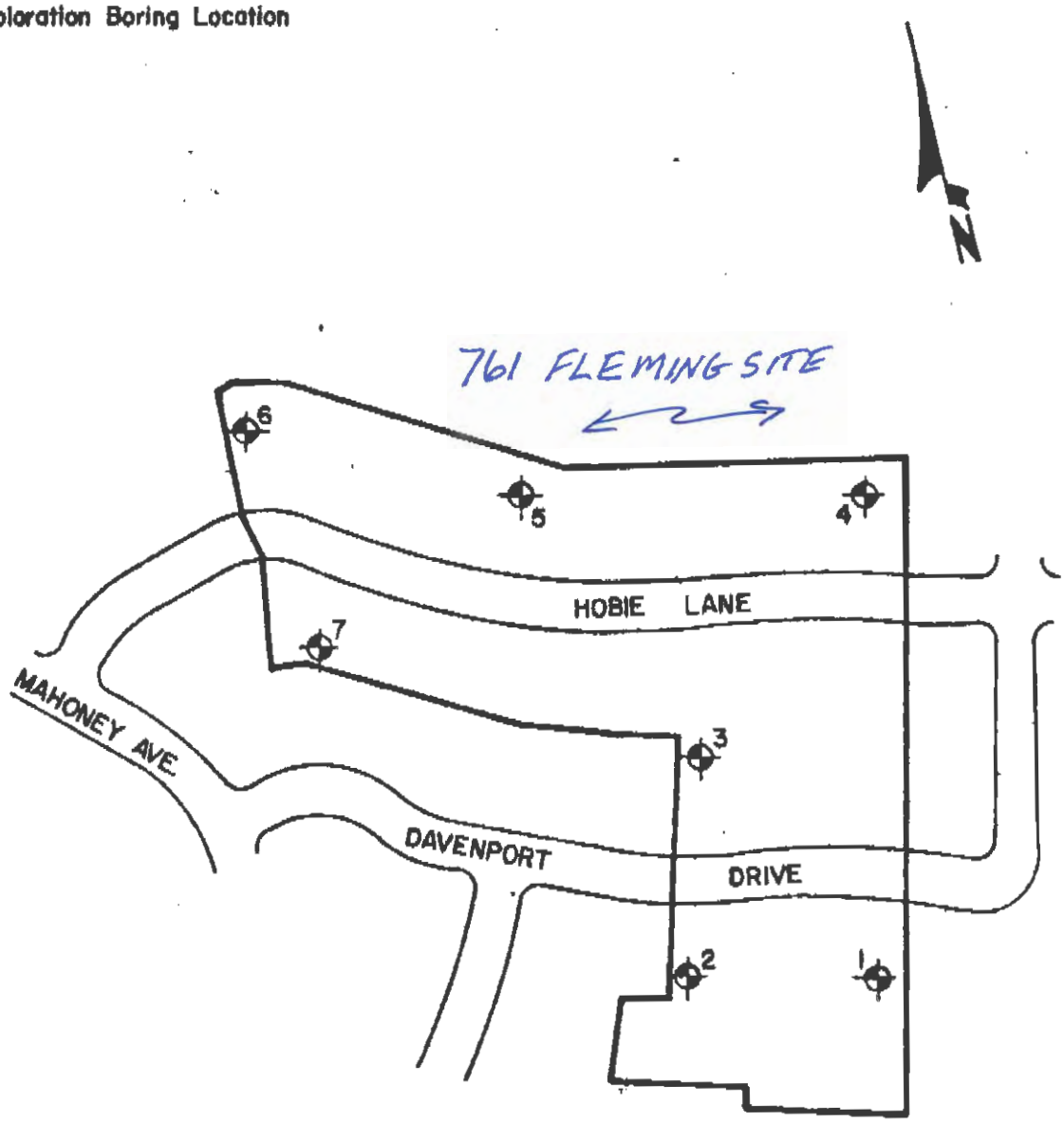
* Soil type classification based on British suggested revisions to Unified Soils Classification System

Figure 10 - Atterberg Limit Tests

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KEY:

Exploration Boring Location



SITE PLAN

PUEBLO TRACT NO. 1
SAN JOSE, CALIFORNIA

-5325

PROJECT 1586 TERRATECH DECEMBER, 1972

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO 1

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 285' DEPTH TO GROUNDWATER INITIAL - FINAL - LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CALIF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
CLAY, Sandy	Red Brn.	Moist	Firm	CIS	1	x		
					2	x		
			Stiff		3	x	8	
SAND, Clayey	Red Brn.	Damp	V.Dense	SWF	4	x		
					5	x	72+	
	Red-Brn. Lt. Brn Mottled	Moist	Dense		6	x		
					7	x		
					8	x	37	4.5+
					9			
					10	x		
SAND, Silty	Lt. Brn.	Moist	V.Dense	SWN	11	x		
					12	x	79+	
					13			
					14			
					15			
					16	x		
					17	x	60+	
					18			
SAND, Coarse with shale fragments					19			
Bottom of Hole - 20.0' NW								

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SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

-5325

PROJECT 1586 TERRATECH

EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO 2

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 270' DEPTH TO GROUNDWATER INITIAL - FINAL - LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CALIF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
SAND, Gravelly, clayey (Shale fragments)	Red Brn.	Moist	Loose	SCI	1	x		
					2	x		
			M.Dense		3	x	8	
SAND, Gravelly to GRAVEL, Sandy	Yel-Brn.	Moist	Dense	SWF to GWF	4	x		
	Gray-Yel Brown				5	x	35	
SAND, Gravelly	Yel-Brn & Red Brown	Moist	Dense	SWF	6	x		
					7	x	45	
					8			
					9			
					10			
SAND, Clayey	Yel Brn. & Red Brn Mtld	Moist	M.Dense	SCI	11	x		
					12	x	18	4.5+
(Gravel seam)					13			
					14			
	Yel.Brn				15			
Bottom of Hole - 15.0' No Water Encountered					16			
					17			
					18			
					19			

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

-5325

PROJECT 1586 TERRATECH

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO. 3

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 245' DEPTH TO GROUNDWATER INITIAL - FINAL - LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CALIF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
CLAY, Fine Sandy	Black	Moist	Firm	CIS	1-3	x x x	4	1.8 1.5
CLAY, Sandy	Brn.with white specks	Moist	Stiff	CLS	4-5	x x x	14	
SAND, Clayey, fine	Brn. & Yel. Brn	Moist	Dense	SPuF	6-12	x x x x x x	32 39	
GRAVEL, Sandy (Clasts up to 1 1/2 inches)	Brown	Moist	Dense	GW	13			
Bottom of Hole - 13.5' No Water Encountered					14-19			

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

PROJECT 1586 TERRATECH

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO. 4

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 235' DEPTH TO GROUNDWATER INITIAL 11'6" ▽ FINAL 10'6" ▼ LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	MODIF CALIF DEPTH	BLOWS PER FOOT	POCKET PENETROMETER	TORVANE	
CLAY, Sandy, silty	Black	Moist	Firm	CIS	1	x			
			Stiff		2	x	7	1.0	
					3				
CLAY, Sandy (Caliche)	Lt.Brn Yel.Brn & White Mottled	Moist	V.Stiff	CLS	4	x			
					5	x	21		
SAND, Clayey	Lt.Brn Yel.Brn & White Mottled	Moist	M.Dense	SPuF	6	x			
					7	x	23		
					8				
					9				
					10				
CLAY, Sandy	Lt.Brn Yel Brn. & White Mottled	▽ ▼	Hard	CLS	11	x			
					12	x	35		
					13				
					14				
					15				
CLAY, Sandy	Lt. Brn. Yel.Brn. & White Mottled		Hard	CLS	16	x		3.5	
					17	x	31	2.2	
					18				
					19				
Bottom of Hole - 20.0'									

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

PROJECT 1586 TERRATECH -5325

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO. 5

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 210' DEPTH TO GROUNDWATER INITIAL 11'6" ▽ FINAL 10'7" ▽ LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CAUF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
CLAY, Silty	Black	Moist	Firm / Stiff	CIS	1-3	x x	8	2.8
CLAY, Sandy	Lt. Brn. / Buff & Org.Mtld	Moist	Hard	CLS	4-9	x x x x x x	30 / 34	4.5
SAND, Clayey	Buff	▽	Dense	SWF	10-15	x x x x	31	2.5
SAND, Gravelly, clayey	Mtld.Yel Brn,Buff & White		Dense	SWF	16-19	x x x	45	
Bottom of Hole - 20.0'								

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO. 6

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 195' DEPTH TO GROUNDWATER INITIAL 13'3" ▽ FINAL 13'7" ▼ LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CALIF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
CLAY, Silty	Black	Moist	Stiff	CIS	1	x		
					2	x	12	
					3			
SAND, Clayey (Caliche)	Buff & White	Moist	Dense	SCL	4	x		
					5	x	31	
	White & Yel.Brn.				6	x		
					7	x		
					8	x	37	
					9			
					10			
SAND with fines	Buff	Moist	V.Dense	SWF to GWF	11	x		4.0
GRAVEL, Sandy	Lt. Brn.				12	x	55+	3.5
		▽ ▼			13			
					14			
					15			
SAND, Clean	Lt. Brn.	Wet	Dense	SPu	16	x		1.8
					17	x	37	2.5
					18			
					19			
Bottom of Hole - 20.0'								

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

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EXPLORATION DRILL HOLE LOG

PROJECT Pueblo Tract #1 (Portion) HOLE NO. 7

DATE 11/28/72 HOLE DIA. 6" DRILL RIG Cont.Flight SAMPLER Mod.Calif.

HOLE ELEV 200' DEPTH TO GROUNDWATER INITIAL - FINAL - LOGGED BY JAB

DESCRIPTION	COLOR	MOISTURE	CONSISTENCY	SOIL TYPE	DEPTH	MODIF CALIF BLOWS PER FOOT	POCKET PENETROMETER	TORVANE
CLAY, Silty	Black	Moist	Firm	CIS	1	x		
						x		
						x	7	1.75
					2			1.5
SAND, Clayey	Buff & Lt. Brn.	Moist	M.Dense	SPuF	3	x		
						x		4.5
					4	x	24	4.5
	Lt. Brn. & White		Dense		5			
						x		
					6	x		
						x	41	
					7			
					8			
					9			
	Lt. Brn & White Mottled				10			
						x		
					11	x		
						x	44	
					12			
					13			
	Lt. Brn.				14			
Bottom of Hole - 15.0' No Water Encountered					15			
					16			
					17			
					18			
					19			

SOIL TYPE CLASSIFICATION BASED ON BRITISH SUGGESTED REVISIONS TO UNIFIED SOILS CLASSIFICATION SYSTEM

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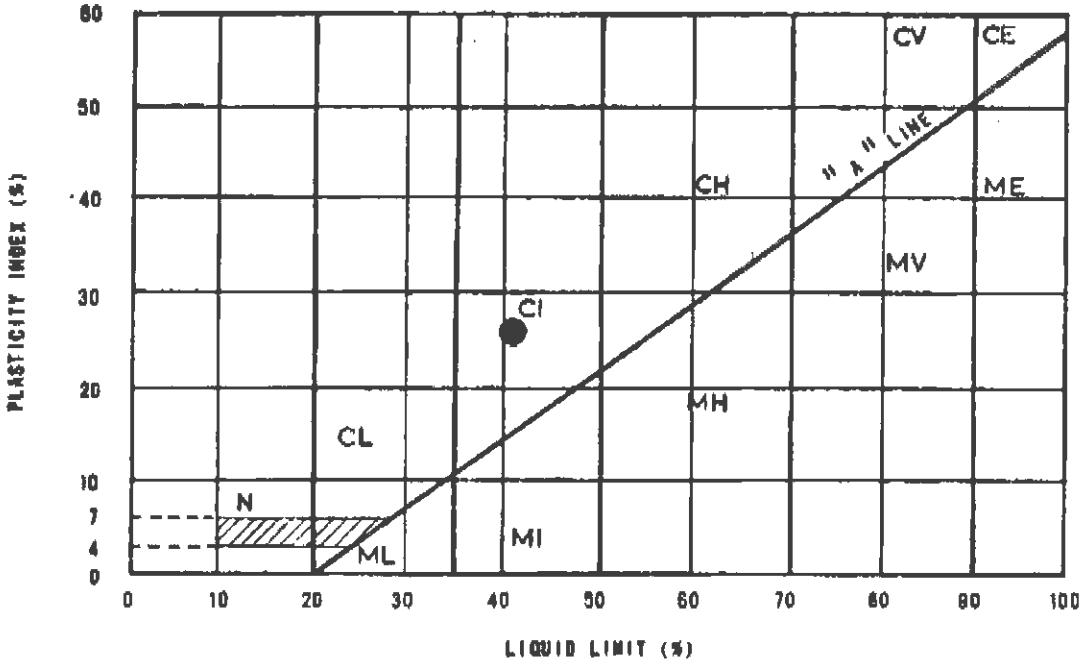
SUMMARY OF LABORATORY TEST DATA

HOLE NO.	DEPTH (Feet)	NATURAL WATER CONTENT (%)	NATURAL DRY DENSITY (pcf)	UNCONFINED SHEAR STRENGTH (psf)
1	1.0	30	92	1190
	5.5	17	106	
	10.5	9	106	
2	1.0	15	102	
	4.0	11	108	
	5.5	14	98	
	11.0	19	103	
3	1.0	14	113	660
	4.0	22	104	
4	1.0	21	104	620
	4.0	19	110	4680
	11.0	19	109	2900
	16.0	25	100	
5	1.0	22	101	1540
	6.0	16	112	3600
6	1.0	28	90	1080
	6.0	16	116	5330
	11.0	15	115	
7	1.0	23	99	1260
	6.0	17	112	

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PLASTICITY CHART



PLASTICITY DATA

KEY SYMBOL	HOLE NO	DEPTH (feet)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	UNIFIED SOIL CLASSIFICATION * SYMBOL
●	Hobie Lane	0 - 0.5	41	26	CIS

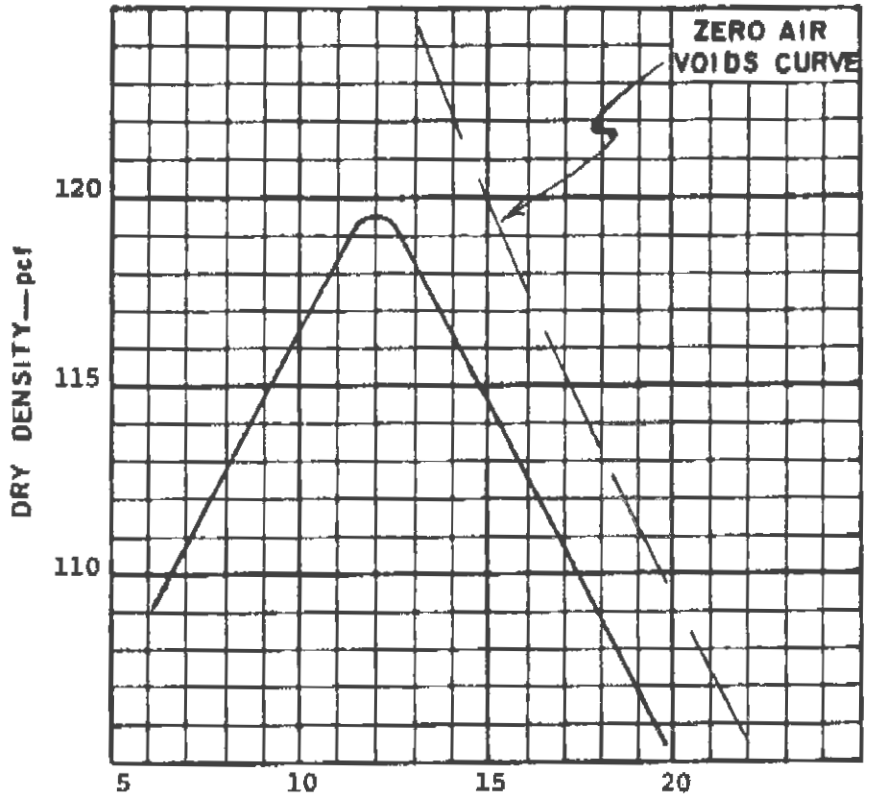
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* Soil Type Classification Based On British Suggested Revisions To Unified Soils Classification System

Project 1586 TERRATECH, INC. Figure 2

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COMPACTION TEST



SAMPLE NO.
Hobie Lane

SAMPLE DEPTH
0 - 0.5

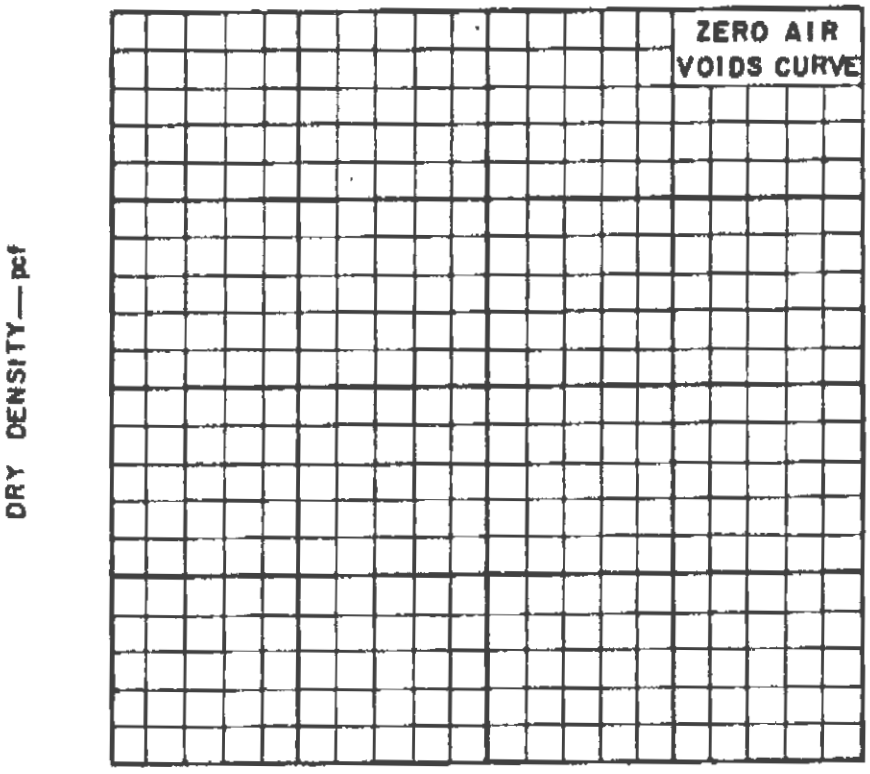
SAMPLE DESCRIPTION
Dark gray, sandy, silty CLAY

SPECIFIC GRAVITY
2.70 (EST)

TEST DESIGNATION
ASTM D1557-70

MAXIMUM DRY DENSITY (pcf)
119.5

OPTIMUM MOISTURE CONTENT, %
12



SAMPLE NO.

SAMPLE DEPTH

SAMPLE DESCRIPTION

SPECIFIC GRAVITY

TEST DESIGNATION

MAXIMUM DRY DENSITY (pcf)

OPTIMUM MOISTURE CONTENT, %

PROJECT NO. 1586

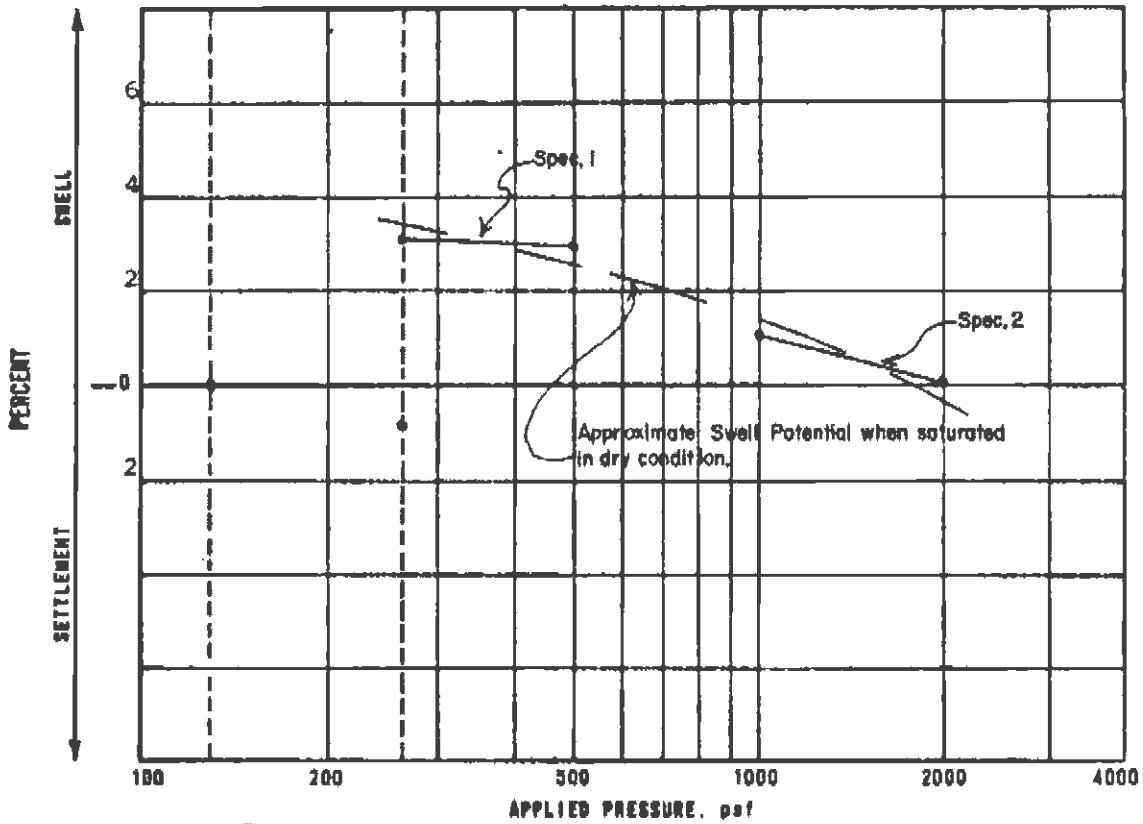
FIGURE 3

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SWELL-COMPRESSION TEST

HOLE NO. Hobie Lane
DEPTH, FT. 0 - 0.5

SPEC. NO.	INITIAL SPECIMEN DATA			FINAL SPECIMEN DATA		
	DRY DENSITY (pcf)	WATER CONTENT (%)	DEGREE OF SATURATION (%)	DRY DENSITY (pcf)	WATER CONTENT (%)	DEGREE OF SATURATION (%)
1	94.6	12.0	47	91.4	26.6	85
2	94.5	12.0	47	94.5	27.3	94



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-5325

PROJECT NO. 1586 TERRATECH FIGURE 4

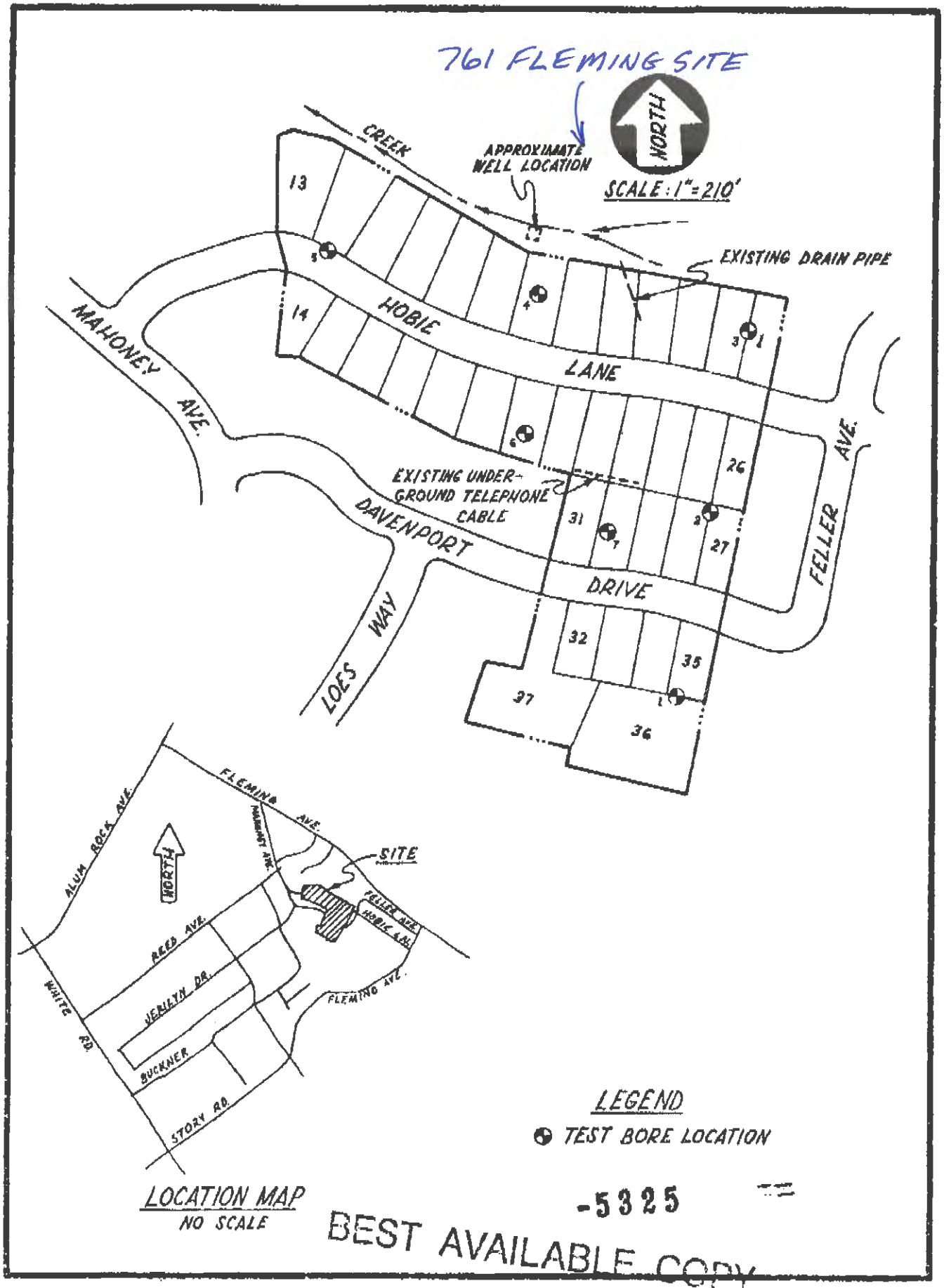


Fig. 1 - Site Plan and Test Bore Location, Tract 5325
San Jose, CA

File No. A2-0175-J1
February 9, 1973

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 1		
0				Ground Surface Elevation=290 Feet		
	1-1			Medium weed growth, Dark Brown Silty CLAY with small Gravel, stiff, moist	105.7	21.5
				Light Brown Sandy GRAVEL very dense, slightly moist		
5	1-2		150	Sandy GRAVEL very dense, slightly moist (gravel up to 6" diameter encountered)	115.5	15.1
10	NG		200			
15						
20				Boring Terminated at 20 Feet		

Fig. 4 - Log of Test Boring No. 1

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File No. A2-0175-J1
February 9, 1973

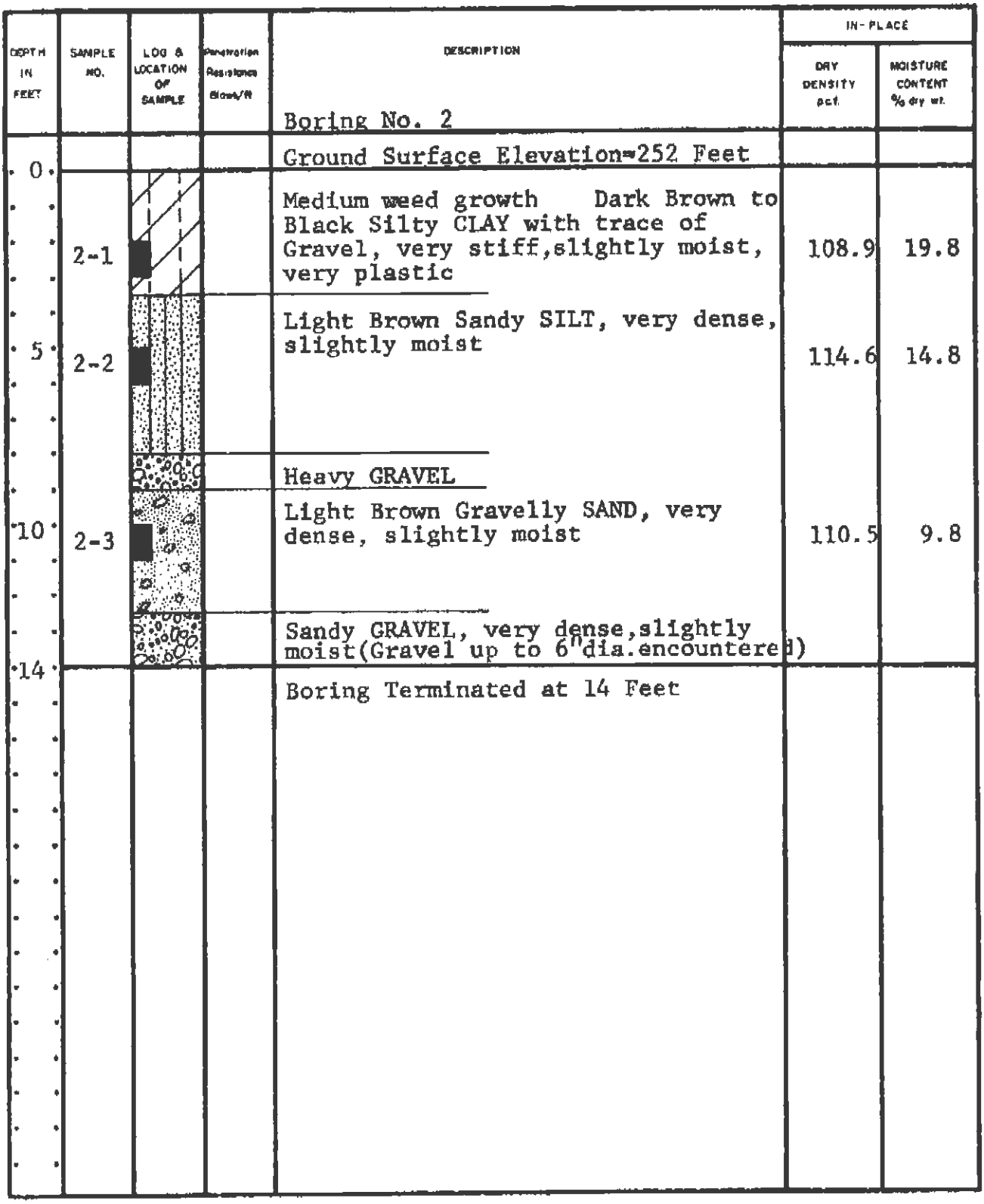


Fig. 5 - Log of Test Boring No. 2

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File No. A2-0175-J1
February 9, 1973

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 3		
				Ground Surface Elevation=230 Feet		
0				Medium weed growth		
	3-1			Black Silty CLAY, stiff, slightly moist, very plastic	104.4	20.2
	3-2				110.0	17.7
5						
				Dark Brown Silty CLAY with a trace of Gravel, very stiff, slightly moist		
	3-3				109.6	18.8
-10				Dark Greyish-Brown Silty CLAY with a trace of Gravel, very stiff, slightly moist		
				~~W/T @ 3PM, 1/4/73~~		
	3-4			Brown mottled Grey Sandy CLAY very stiff, moist	108.6	19.3
-15						
				Brown, Sandy SILT, dense, moist Gravel content increases with depth		
-20				Boring Terminated at 20 Feet		

Fig. 6 - Log of Test Boring No. 3

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File No. A2-0175-J1
February 9, 1973

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 4		
				Ground Surface Elevation=215 Feet		
0	4-1		33	Surface weed growth Black Silty CLAY, very stiff slightly moist, very plastic	98.3	24.0
5	4-2		75	Light Brown to Grey Clayey SILT trace of fines, very dense, slightly moist	111.3	16.5
				Light Brown Sandy SILT, very dense, slightly moist		
10	4-3		100	Brown and Light Brown Sandy CLAY very stiff, slightly moist	115.8	11.6
15				Brown Gravelly CLAY, poorly graded, very stiff, slightly moist		
16				Boring Terminated at 16 Feet		

-5325

Fig. 7 - Log of Test Boring No. 4

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File No. A2-0175-J1
February 9, 1973

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 5		
				Ground Surface Elevation-207 Feet		
0				Medium weed growth		
	5-1			Dark Brown to Black Silty CLAY very stiff, slightly moist	118.2	15.8
5	5-2				108.0	18.1
				Light Brown Sandy SILT very dense, slightly moist		
	5-3				113.4	14.6
10				Brown Sandy GRAVEL, very dense slightly moist		
15				Boring Terminated at 15 Feet		

-5325

Fig. 8 - Log of Test Boring No. 5

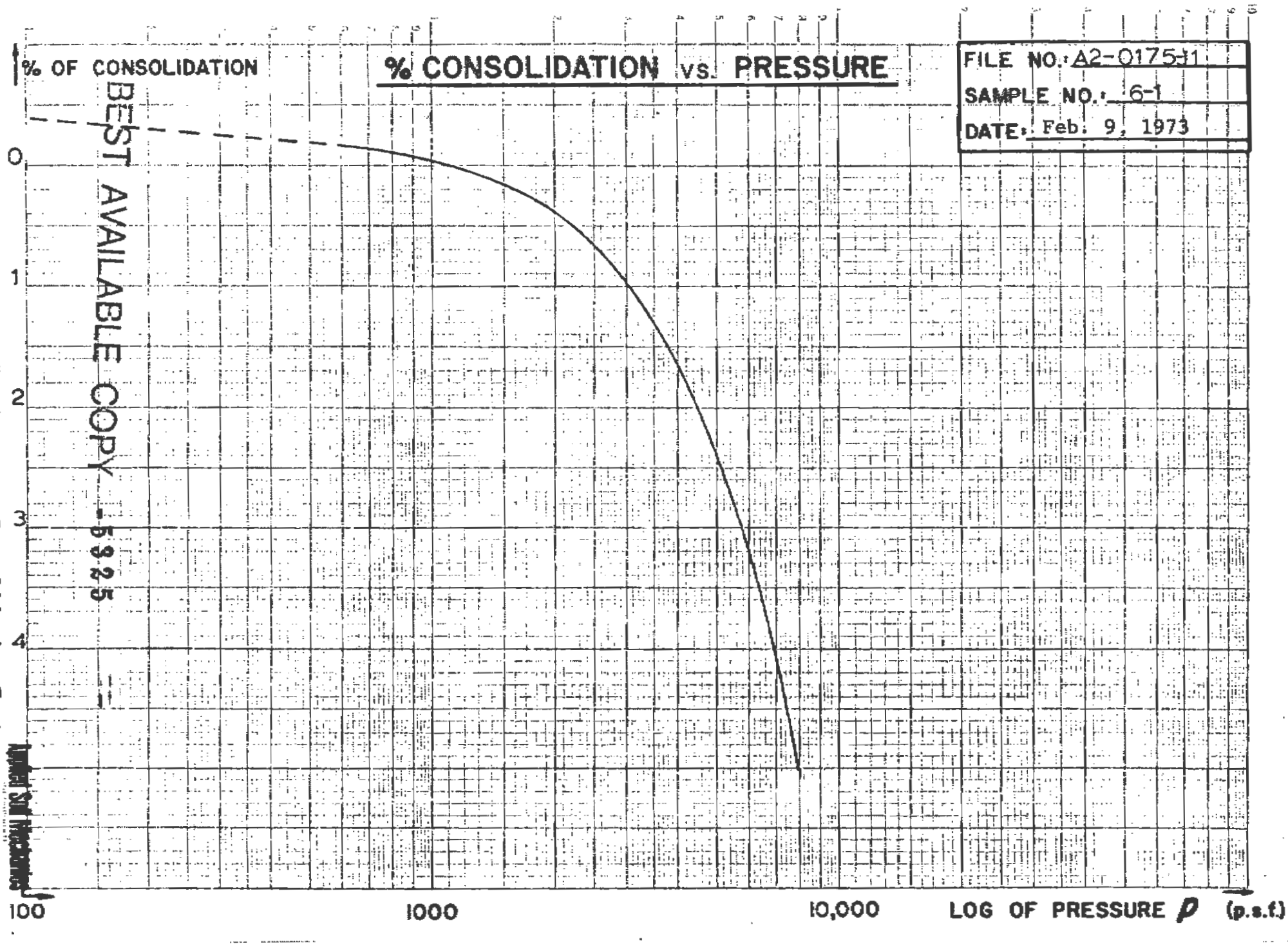
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File No. A2-0175-J1
February 9, 1973

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 7		
				Ground Surface Elevation=250 Feet		
0				Black Silty CLAY, very stiff, slightly moist		
				Brown Sandy SILT, very dense, slightly moist		
5				Brown coarse Sandy SILT, very dense, slightly moist		
10						
				Light Brown Silty SAND, very dense, slightly moist, heavy Gravel		
15				Boring Terminated at 15 Feet		

Fig. 10 - Log of Test Boring No. 7

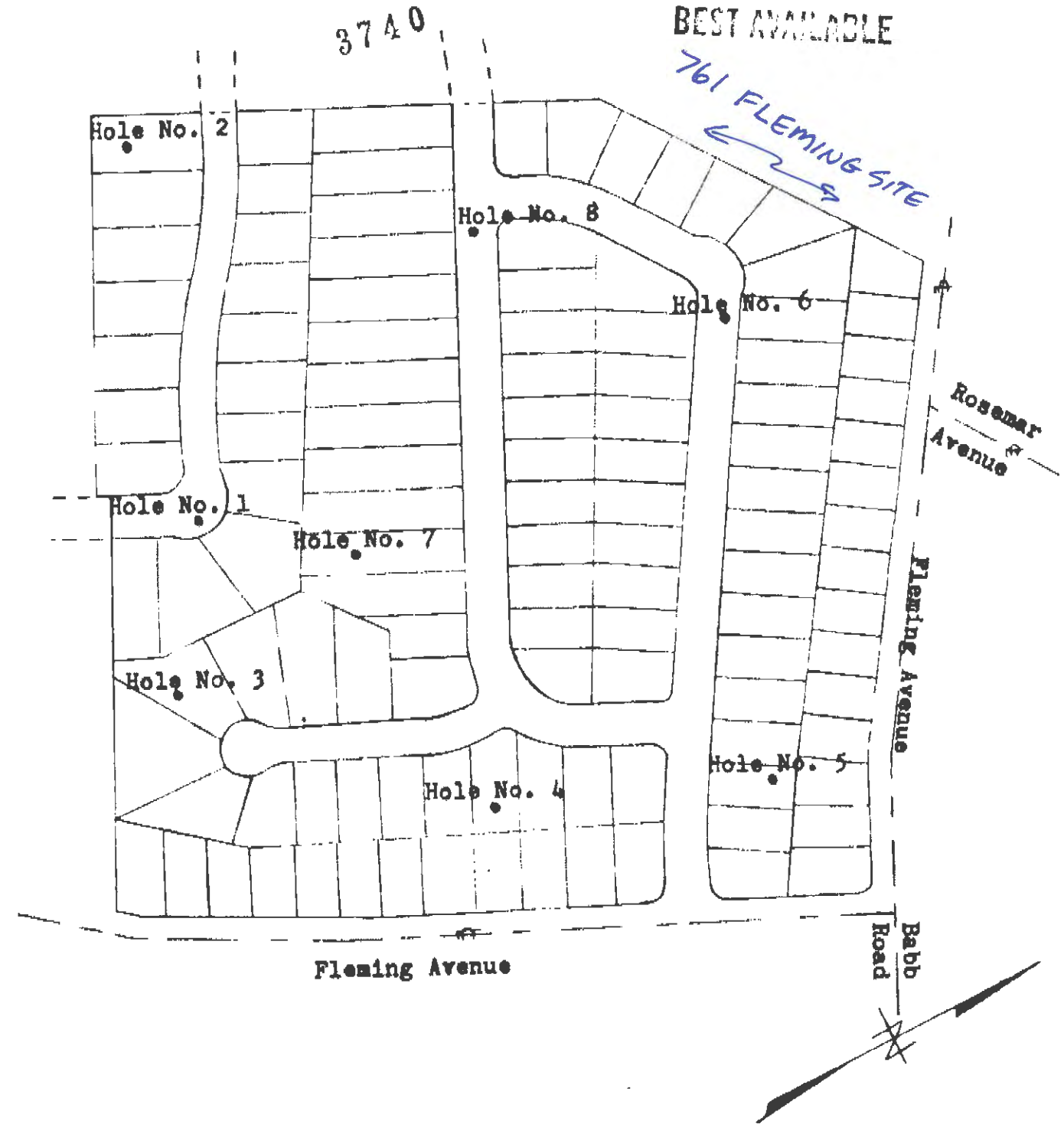
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Fig. 11 - Plot of Laboratory Consolidation Test

File No. T1184-1
8 July 1960
Figure No. 1



File No. T1184-1

8 July 1960

Figure No. 2

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place Dry Density p.c.f.	In - Place Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 1			
1			brown clayey sand - slightly moist, firm			
2						
3			brown silty sand - dry firm			
4						
5	1-1		becoming gravelly	115.2	15.0	
6						
7			becoming moist rockey (weathered sandstone)			
8						
9			brown weathered sandstone			
10			firm, rocky			
11						
12						
13			becoming less firm			
14						
15						
16						
17			Refusal			
18						
19			Bottom of Hole No. 1 Drilled June 6th, 1960 Mobile B40 Continuous flight 7 in dia. hollow stem auger			

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3740

LOG OF TEST BORING

HOLE NO. 1

File No. T1184-1
8 July 1960
Figure No. 3

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place: Dry Density p.c.f.	In - Place: Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 2			
1			brown sandy silt - dry			
2						
3						
4			brown sandy clay - slightly moist, firm			
5			brown clayey sand - firm			
6						
7			becoming rocky			
8			Refusal - Hard Sandstone			
9						
			Bottom of Hole No. 2 drilled 6 June 1960 Mobile B40 Continuous flight 5½ in. dia. auger			

3740

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LOG OF TEST BORING

HOLE NO. 2

File No. T1184
8 July 1960
Figure No. 4

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place: Dry Density p.c.f.	In - Place: Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 3			
1			brown clayey sand - slightly moist, firm			
2			becoming light tan			
3						
4			brown sand w/gravel dry - firm			
5						
6						
7						
8						
9						
10						
11						
12						
13			3740			
14			becoming less firm			
15						
16						
17						
18						
19						
20						
			Bottom of Hole No. 3 Drilled 6 June 1960 Mobile B40 Continuous flight 5½ in. dia. auger			

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LOG OF TEST BORING

HOLE NO. 3

File No. T1184-1
8 July 1960

Figure No. 5

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place Dry Density p.c.f.	In - Place Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 4			
1	4-1		black organic silty sand - dry	95.0	6.5	125
2						
3						
4						
5			light tan silty sand - slightly moist becoming gravelly			
6						
			Bottom of Hole No. 4 Drilled 6 June 1960 Mobile B40 Continuous flight 7 in. dia. hollow stem auger			
			BEST AVAILABLE			
			Ground Surface Hole No. 5			
0						
1	5-1		tan silty sand - dry, loose	92.0	12.2	
2			3740			
3						
4			black clayey sand - moist, firm			
5						
6	5-2			103.0	17.0	
7						
			Bottom of Hole No. 5 Drilled 7 June 1960 Mobile B40 Continuous flight 7 in. dia. hollow stem auger			

LOG OF TEST BORING

HOLE NO. 4 and 5

File No. T1184-1

8 July 1960

Figure No. 6

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place Dry Density p.c.f.	In - Place Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 6			
1	6-1		black organic silty clay - slightly moist	92.0	16.7	
2						
3						
4						
5						
6	6-2			109.0	16.6	
7						
			Bottom of Hole No. 6 Drilled 7 June 1960 Mobile B40 Continuous flight 7 in. dia. hollow stem augers			
			BEST AVAILABLE			
			Ground Surface Hole No. 7			
0						
1	7-1		black organic clayey silt - slightly moist, firm 3740	106.2	14.4	
2						
3						
4			brown clayey sand - slightly moist, firm			
5			becoming moist			
6						
7						
8	7-2		Refusal Rock Sandstone	106.8	17.5	900
			Bottom of Hole No. 7 Drilled 7 June 1960 Mobile B40 Continuous flight 7 in. dia. hollow stem augers			

LOG OF TEST BORING

HOLE NO. 6 and 7

File No. T1184-1

8 July 1960

Figure No. 7

Depth in feet	Sample No.	Log & Location of Sample	Description	In - Place Dry Density p.c.f.	In - Place Moisture Content % dry wt.	Shear Strength p.s.f.
0			Ground Surface Hole No. 8			
1	8-1		black organic silty sand - dry, firm	105.0	14.2	
2						
3						
4						
5			brown silty sand - slightly moist, soft w/large amount clay			
6	8-2			96.7	13.4	
7						
			Bottom of Hole No. 8 Drilled 7 June 1960 Mobile B40 Continuous flight 7 in. dia. hollow stem augers			

BEST AVAILABLE

3740

LOG OF TEST BORING

HOLE NO. 8

APPENDIX D

GBA Guidelines for Geotechnical Report

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, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them 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 herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences 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.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

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

- for a different client;
- for a different project or purpose;
- 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, the reliability of a geotechnical-engineering report can be 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 you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site 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.

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

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement 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 through project completion to obtain 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 judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed 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 continuing 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 available 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-phase observations.

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 information 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. 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. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. 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 provide 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 obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

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, the engineer's services were not designed, conducted, or intended to prevent 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.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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