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GEOCON PROJECT NO. E9291-04-01

MAY 2022

Project No. E9291-04-01
May 18, 2022

COAR Design Group
200 EAST Street
Santa Rosa, California 95404

Attention: Mr. Jeff Katz

Subject: BUTTERFIELD FIRE STATION
17285 BUTTERFIELD BOULEVARD
MORGAN HILL, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Katz:

In accordance with your authorization, we have performed a design-level geotechnical investigation for the subject fire station in Morgan Hill, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

A. E. Ashour

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a planned fire station at 17285 Butterflied Boulevard in Morgan Hill, California (See Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and geotechnical recommendations for project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration included six soil borings performed on December 21, 2021. Our soil borings were advanced to depths ranging from approximately 5 to 30 feet below existing grade. The locations of our explorations are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. In addition, two soil samples were submitted to our laboratory for screening-level corrosion testing. Appendix B presents the laboratory test results in tabular and graphical format.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The project site is an approximately 1 ¾-acre parcel (Santa Clara County APN 726-15-072) fronting the southwest side of Butterfield Boulevard, approximately ¼ mile north of East Dunne Avenue. The parcel is occupied by a community park with concrete walkways, benches and landscaping, and a former community garden. To the northwest, the site is bordered by a surface parking lot for the nearby transit center. A Santa Clara County courthouse is present to the southeast. The back (southwest side) of the parcel abuts the Caltrain rail line. Topographically, the site is relatively flat with ground surface elevations of approximately 350 feet MSL based on web-based mapping.

Based on the information provided by COAR Design Group, the project will include the design and construction of a new one- to two-story fire station. The new station will be designed to accommodate up to three fire apparatus. The layout of the station includes three drive-through apparatus bays, a separate workshop area, living quarters and administrative/conference room areas. No subterranean levels are anticipated but a localized deeper excavation may be needed if the two-story portion of the building includes an elevator. Ancillary site improvements such as new pavements, exterior flatwork and underground utilities are also planned.

Civil plans were not available at the time of this report, but we anticipate cuts and fills in order of 2 feet or less will be required to establish design subgrade elevation for the fire station building pad. The proposed site configuration is depicted on Figure 2.

3. GEOLOGIC SETTING

Morgan Hill is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward and Calaveras faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Geologic mapping by the California Geological Survey indicates the site is underlain by Pleistocene-age alluvial fan deposits. Artificial fills from past episodes of site development are also present.

4. GEOLOGIC HAZARDS

4.1 Faulting and Seismicity

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The site and greater Bay Area are seismically dominated by the presence of the active San Andreas Fault System. In the theory of plate tectonics, the San Andreas Fault System is a transform fault that forms the boundary between the northward moving Pacific Plate (west of the fault) and the southward moving North American Plate (east of the fault). Locally, the movement is distributed across a complex system of strike-slip, right lateral parallel and subparallel faults – including the San Andreas, Hayward and Calaveras faults.

The table below presents approximate distances to active faults within approximately 20 miles of the site based on web-based mapping by CGS, as previously published by Caltrans. Site coordinates are N 37.1297 and W 121.6493°.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
Silver Creek	3	6.9
Calaveras (Central)	3 ½	6.9
Sargent	7 ½	7.0
Hayward (Southern Extension)	8 ¼	6.7
San Andreas	10	8.0
Zayante-Vergelas (Upper)	13 ¾	7.0
Monte Vista- Shannon	15	6.4
Zayante-Vergelas (Lower)	16 ½	7.0

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The California Geological Survey defines an active fault as a fault that shows evidence for activity within the last 11,000 years. A potentially active fault is generally defined as a fault that has shown evidence of displacement between 11,000 and 1.6 million years ago. Faults that have not demonstrated evidence of movement with the past 1.6 million years are generally considered inactive.

4.3 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and modal (most probable) magnitude associated with a 2,475-year return period. This return period corresponds to an event with 2% chance of exceedance in a 50-year period. The USGS estimated PGA is 0.99g and the modal magnitude is 7.3 for Seismic Site Class D (V_s30 = 259 m/sec) based on a 2014 model within the application.

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.4 Liquefaction and Dynamic Compaction

The site is not located within a CGS Seismic Hazard Zone for liquefaction, nor within a Santa Clara County Geologic Hazard Zone for the same. In addition, web-based hazard mapping by the Association of Bay Area Governments indicates a low susceptibility to liquefaction at the site. Our soil borings did not encounter soils susceptible to liquefaction or dynamic compaction (dry sand settlement) at the intervals sampled.

4.5 Landslides

There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a significant hazard to this project.

4.6 Seiches

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Undocumented Fill

We encountered undocumented fill materials to depths up to approximately 2 ½ feet below existing grade in Borings B4 through B6. The source of the fill materials is unknown. As observed in our soil borings, the fill materials consisted of stiff silty clay with various amount of sand and gravel and loose silty/clayey sand with various amount of gravel. Based on our laboratory test results, the undocumented fills are low plasticity clays that possess low expansion potential.

5.2 Alluvium

Alluvial deposits at the site generally extend from below the undocumented fill to the maximum depths explored–approximately 30 feet below existing grade. The alluvium generally comprises stiff to hard silty clay with variable amounts of sand and gravel and dense clayey sand with occasional sandstone fragments. A medium dense clayey sand layer with silt and gravel was encountered at depths between approximately 2 ½ to 5 feet in Boring B1 and to a depth of approximately 3 ½ feet in Boring B2. In Boring B4, we encountered a layer of medium stiff sandy clay between approximately 2 ½ and 7 feet below existing grade.

Soil conditions described in the previous paragraphs are generalized. Therefore, we advise the reader to consult the exploratory boring logs included in Appendix A.

5.3 Groundwater

Groundwater was not encountered in our soil borings to the maximum depth explored – approximately 30 feet below existing grade. Historic high groundwater levels are approximately 20 to 25 feet below natural grade per CGS mapping. Information maintained online by Santa Clara Valley Water District indicates historic high groundwater levels are about 20 feet below grade in the site vicinity. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study. Additionally, areas of seepage may develop where not previously observed, particularly during or after periods of heavy precipitation.

5.4 Soil Corrosion Screening

Two soil samples obtained during our field exploration were subjected to laboratory testing for minimum resistivity, pH, and chloride and water-soluble sulfate. The laboratory test results and published screening levels are presented in Appendix B. Soil corrosivity should be considered in the design of buried metal pipes, underground structures, etc.

Water-soluble sulfate test results on the selected sample of site soils indicate an S0 exposure classification for sulfate attack on normal portland cement concrete (PCC) as defined in Chapter 318, Table 19.3.1.1 of the ACI *Building Code Requirements for Structural Concrete*. ACI does not set forth requirements for S0 sulfate exposure classification. In addition, the soil samples that we tested would not be classified as corrosive to buried metal improvements based on Caltrans criteria.

Geocon does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the presence undocumented fill materials and the potential for strong seismic shaking. Remedial grading will be required to remove and recompact undocumented fill materials in areas to receive settlement sensitive improvements.

6.1.2 Based on the observed soil conditions and the assumed structural loading, we anticipate that a conventional shallow foundation system (strip and spread footings) used in conjunction with the recommended remedial grading presented herein can be used to support the proposed fire station building.

6.1.3 The proposed project redevelops a site with past episodes of site development. As such, unknown underground improvements and/or areas of undocumented fill materials may be present. Supplemental recommendations may be provided during site development.

6.1.4 For shallow foundation systems designed and constructed as recommended herein, estimated post-construction settlement due to dead + live loads should be ¾ inch or less with differential settlements of approximately ½ inch or less across a horizontal distance of 50 feet.

6.1.5 Project grading plans were not available at the time of this report. We should review grading plans once available to determine applicability of the recommendations provided herein, particularly those related to site grading and building pad preparation. Updated or supplemental recommendations may be necessary.

6.1.6 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.7 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.2 Seismic Design Criteria

6.2.1 We understand that seismic structural design will be performed in accordance with the provisions of the 2019 CBC which is based on the American Society of Civil Engineers (ASCE) publication *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16). We derived the following seismic design parameters using the web-based Structural Engineers Association of California application *U.S. Seismic Design Maps*. Results are summarized in Table 6.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R) and Seismic Risk Category IV.

TABLE 6.2.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.606g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.606g	Section 1613.2.3 (Eq. 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.02*	Section 1613.2.3 (Eq. 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.07g	Section 1613.2.4 (Eq. 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.68g*	Section 1613.2.4 (Eq. 16-39)

Note:
*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with S_s greater than or equal to 1.0g and for Site Class "D" and "E" sites with S_1 greater than 0.2g. Section 11.4.8 also provides exceptions where ground motion hazard analysis may be waived. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed in project design.

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-16 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.2
2019 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.67g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.737g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 The onsite soils can be excavated with moderate effort using conventional excavation equipment. We do not anticipate excavations in the native alluvium at the site will generate oversize material (greater than 3 inches in nominal dimension). However, unknown or unanticipated constituents may exist, especially within areas of undocumented fill.

6.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.3 The recommendations of this report assume the foundation system for the fire station will derive support in engineered fill or competent alluvium.

6.4 Materials for Fill

6.4.1 Excavated soils generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 3 inches in maximum dimension.

6.4.2 If needed, import material should possess a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 3 inches in greatest dimension.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All clearing operations and earthwork (including over-excavation, scarification, and recompaction) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of building, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.5.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.5.4 The site should be stripped of all surface vegetation and pavement from the area to be removed. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from demolition and site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 After stripping and clearing the site, the exposed subgrade in the building pad, exterior flatwork and pavement areas should be over-excavated to a depth of 12 inches below existing grade or proposed subgrade (whichever is lower). The resultant bottom should then be scarified to a depth of approximately 1 foot, moisture conditioned above optimum moisture and recompacted to at least 90% relative compaction.

6.5.6 Over-excavation, exposed bottom surfaces and bottom processing should be observed by our representatives on a full-time basis. Undocumented fills in the building pad should be completely removed to expose a competent native bottom and replaced with properly compacted fill. Supplemental recommendations may be provided based site conditions during grading. Based on the subsurface conditions encountered in our soil borings, over-excavations on the order of 3 feet below existing grade may be required to remove existing undocumented fill materials in the building pad. Over-excavation depths in pavement and exterior flatwork areas may be relaxed at the sole discretion of Geocon based on the conditions encountered during grading.

6.5.7 In general, over-excavated materials may be used for new engineered fill provided they do not contain deleterious matter, organic material, or cementations larger than 3 inches in maximum dimension.

6.5.8 All structural fill and backfill should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 to 12 inches). Fill soils should be placed and compacted to at least 90% relative compaction at slightly above optimum moisture.

6.6 Temporary Excavations

6.6.1 We anticipate that the majority of the site alluvial soils will be classified as Cal-OSHA "Type B" soil when encountered in excavations during site development and construction. If active seepage, loose gravelly or sandy soil, or undocumented fills are encountered, the Cal-OSHA classification should be downgraded to "Type C". Excavation sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation to evaluate trench conditions and make appropriate recommendations where necessary.

6.6.2 It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.7 Shallow Foundations

6.7.1 The fire station may utilize conventional shallow foundations consisting of continuous strip footings and isolated spread footings bearing in competent alluvium or in properly compacted fill following the remedial grading discussed in Section 6.5.5. Continuous strip footings may be used for ancillary site structures such as short retaining walls, screen walls, or trash enclosures.

6.7.2 Strip and spread footings should have a minimum embedment depth of 18 inches below the lowest adjacent pad grade, which is not to be confused with finished floor elevation. The strip footings should be at least 15 inches wide. Spread footings should be at least 4 feet square and founded at least 18 inches below the lowest adjacent pad grade. Perimeter footings for the fire station should consist of continuous strip footings. Any column footings at the perimeter should be integral with the strip footing.

6.7.3 Footings proportioned as recommended may be designed for a net allowable soil bearing pressure of 3,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads and may be increased by up to one-third for transient loads due to wind or seismic forces.

6.7.4 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided the frictional resistance is reduced by 50%. Where not protected by pavement or flatwork, the upper one foot of soil should be ignored when calculating passive resistance.

6.7.5 Minimum reinforcement for continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom. Reinforcement for isolated column footings should be determined by the structural engineer.

6.7.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.7.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.7.8 Footing bottoms and walls should be sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.8 Underground Utilities

6.8.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction above optimum moisture.

6.8.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; open-graded materials such as ¾ inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Pipe bedding and backfill should also conform to the requirements of the governing utility agency.

6.9 Concrete Slabs-on-Grade

6.9.1 Concrete slabs-on-grade subject to vehicle loading are considered pavements and should be designed in accordance with the recommendations in Section 6.11 of this report.

6.9.2 Concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 5 inches thick. Minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

6.9.3 Interior slabs in areas with floor coverings or slabs in areas where moisture would be objectionable should be underlain by 3 inches of ½-inch or ¾-inch crushed rock with no more than 5% passing the No. 200 sieve to serve as a capillary break.

6.9.4 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions, positioned near the slab midpoint. The subgrade should be scarified 12 inches, moisture conditioned to slightly above optimum moisture content and recompacted to at least 90% relative compaction.

6.9.5 In lieu of specific recommendations from the structural or civil engineer, we recommend that crack control joints be spaced at intervals not greater than 8 feet for 4-inch-thick slabs (10 feet for 5-inch-thick slabs). Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.9.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.10 Moisture Protection Considerations

6.10.1 A vapor barrier is not required beneath interior slabs-on-grade for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.10.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.10.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.10.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.11 Pavement Recommendations

6.11.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned slightly above optimum moisture and compacted to at least 95% relative compaction. Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.11.2 Sidewalk, curb and gutter, and driveway encroachments should be designed and constructed in accordance with City of Morgan Hill requirements, as applicable.

6.11.3 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs. We can provide additional sections based on other TIs if necessary.

TABLE 6.11
FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3 ½	12 ½
Heavy-Duty	7.0	4	15 ½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. Subgrade soil has an R-Value of 5.
2. Class 2 AB has a minimum R-Value of 78 and meets the requirements of Section 26 of the latest Caltrans Standard Specifications.
3. Class 2 AB and subgrade is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AC, the AB should be proof rolled with a loaded water truck to verify stability.
4. Asphalt concrete should conform to local agency standards.

6.11.4 The AC sections in Table 6.11 are final, minimum thicknesses. If staged pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1.5 inches thick.

6.11.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. Where heavy truck or apparatus traffic is anticipated, the minimum concrete thickness should be increased to 8 inches. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi.

6.11.6 We recommend that at least 6 inches and 12 inches of Class 2 Aggregate Base be used below rigid concrete pavements for light and heavy traffic, respectively. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.11.7 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.11.8 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.11.9 Asphalt pavement section recommendations for driveways and parking areas are based on Caltrans design procedures. It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The HDM indicates that the resulting pavement sections for parking lots are minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems. It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.12 Retaining Wall Design

6.12.1 Lateral earth pressures may be used in the design of retaining walls. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.12 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.12
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	45 pcf
At-Rest	60 pcf

6.12.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.01H (where H is the height of the wall). The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area.

6.12.3 Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.12.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.12.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.13 Surface Drainage

6.13.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.13.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.13.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.13.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 5 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers and automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

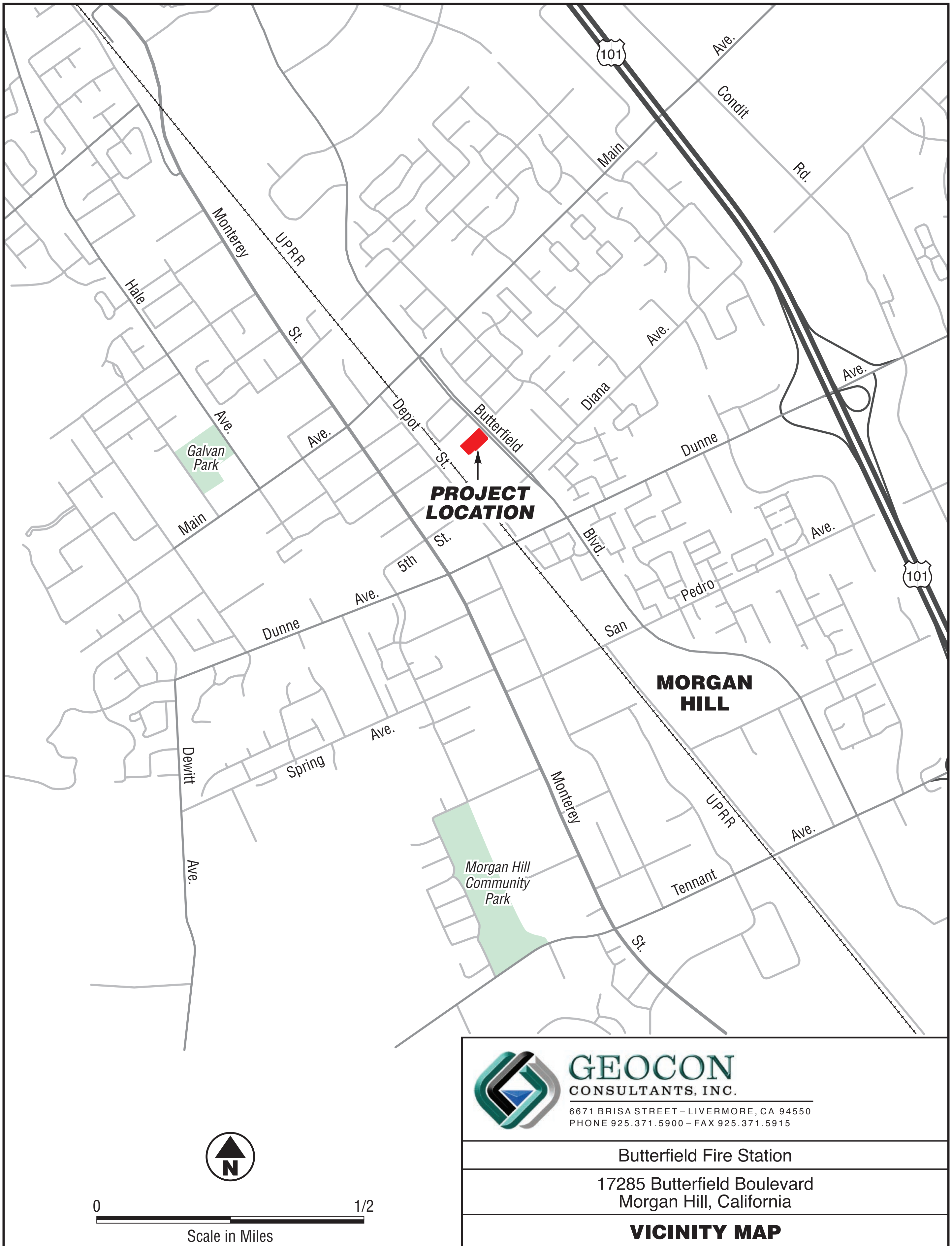
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.



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Butterfield Fire Station

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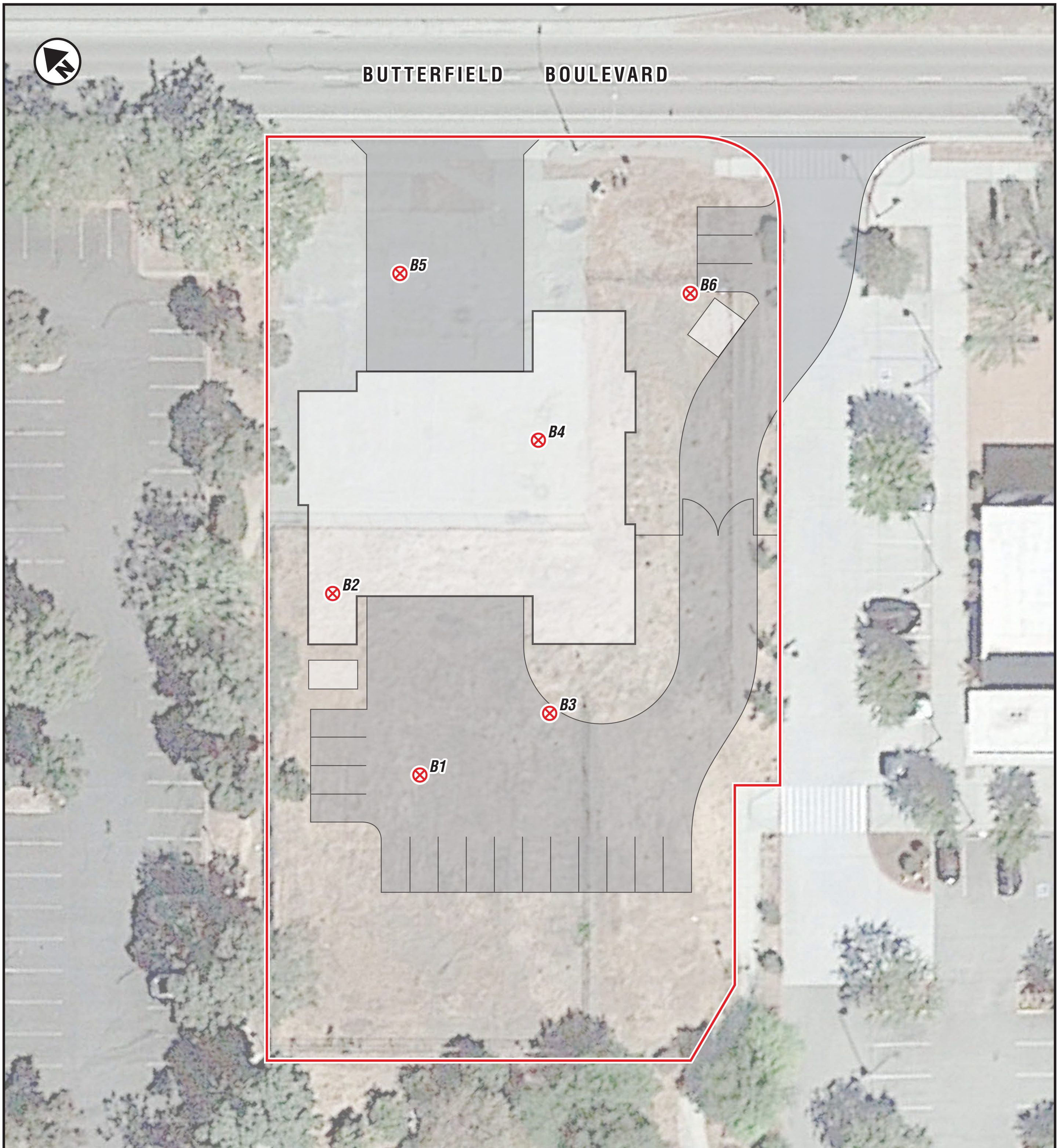
Morgan Hill, California

VICINITY MAP

E9291-04-01	May 2022	Figure 1



LEGEND:

B6 ⊗ Approximate Boring Location

0 40

Scale in Feet

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SITE PLAN

E9291-04-01	May 2022	Figure 2

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings are shown on the Site Plan, Figure 2. Soil boring logs for our exploration are presented as figures following the text in this appendix. The borings were located by pacing from existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our field exploration included six exploratory soil borings to depths ranging from approximately 5 to 30 feet. Our borings were performed on December 21, 2021 by Cenozoic Exploration of Los Gatos, California using a truck-mounted Simco 2400-SKJ drill rig equipped with 6-inch solid-flight augers under Geocon supervision. Sampling in the borings was accomplished using a 140-pound Cathead hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD). The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT “N” values; corrections have not been applied.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

Upon completion of drilling, the borings are backfilled with cuttings and lean cement grout. Borings in pavement areas capped with asphalt cold patch.

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	**GRAVELS** MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	**SILTS AND CLAYS** LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	S<25	DRY
SLIGHT INDICATION OF MOISTURE	25≤S<50	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	50≤S<75	MOIST
MINOR VISIBLE FREE WATER	75≤S<100	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

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KEY TO LOGS

BORING B1

ELEV. (MSL.)	
DATE COMPLETED	12/21/2021
ENG./GEO.	JBM
DRILLER	Cenezoic
EQUIPMENT	Simco 2400-SK1 w/ 6" SFA
HAMMER TYPE	Cathead

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-0-5			CL-ML	ALLUVIUM Very stiff, damp to moist, brown, (f-c) Gravelly Silty CLAY with little (f-c) sand -wood chips at surface -pp>4½			
1								
2	B1-1.5-2 B1-2					34	134.4	8.0
3				SC	Medium dense, moist, brown, Clayey SAND with little gravel			
4	B1-4-4.5 B1-4.5					15	118.4	10.1 13.9
5					END OF BORING AT APPROXIMATELY 5 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS AND GROUT			

Figure A2, Log of Boring B1, Page 1 of 1

GEOCON BORING LOG W/FIG# STARTING W/ A2 E9291-04-01 BORING LOGS.GPJ 05/18/22

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B2

ELEV. (MSL.) ____________ DATE COMPLETED **12/21/2021**

ENG./GEO. **JBM** DRILLER **Cenezoic**

EQUIPMENT **Simco 2400-SK1 w/ 6" SFA** HAMMER TYPE **Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC-SM	ALLUVIUM Medium dense, damp to moist, brown, Silty Clayey (f-c) SAND with few (f) gravel -woodchips at surface			
1–3	B2-1.5-2 B2-2					38		8.6
4	B2-4-4.5 B2-4.5			CL	Very stiff, moist, brown, (f-c) Sandy CLAY with little (f) gravel -less gravel -pp>4½	37	110.2	11.0
8				SC	Dense, moist, brown, Clayey (f-c) SAND with few sandstone fragments			
9	B2-9-9.5 B2-9.5					53		12.1
14	B2-14-14.5 B2-14.5			CL	Hard, moist, brown, Sandy CLAY with trace sandstone fragments -pp>4½	48	123.2	12.2
19	B2-19-19.5 B2-19.5				-with few (f-c) rounded gravels -pp>4½ -(c) gravel in shoe	52		
24	B2-24-24.5 B2-24.5				-more sand	78/12"		

Figure A3, Log of Boring B2, Page 1 of 2

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

BORING B2

ELEV. (MSL.) ____________ DATE COMPLETED **12/21/2021**

ENG./GEO. **JBM** DRILLER **Cenezoic**

EQUIPMENT **Simco 2400-SK1 w/ 6" SFA** HAMMER TYPE **Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25					-pp>4½			
26								
27								
28								
29	B2-29-29.5 B2-29.5				-sand (c), more gravels	65		
30					END OF BORING AT APPROXIMATELY 30 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS AND GROUT			

Figure A3, Log of Boring B2, Page 2 of 2

GEOCON

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

PROJECT NO. **E9291-04-01** PROJECT NAME: **Butterfield Fire Station**

BORING B3

ELEV. (MSL.)		DATE COMPLETED	**12/21/2021**
ENG./GEO.	**JBM**	DRILLER	**Cenezoic**
EQUIPMENT	**Simco 2400-SK1 w/ 6" SFA**	HAMMER TYPE	**Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3-0-5			CL-ML	FILL Very stiff, damp, brown, Silty (f-c) Sandy CLAY with little (f) gravel -woodchips at surface			
1								
2	B3-1.5-2 B3-2				-pp>4½	27	128.9	8.9
3				CL	ALLUVIUM Stiff, damp, brown, Sandy CLAY			
4						21		
5								
6								
7								
8								
9	B3-9-9.5 B3-9.5				-very stiff, gravelly	43	129.1	8.8
10								
11								
12								
13								
14	B3-14-14.5 B3-14.5				-hard	70		
15								
16								
17								
18								
19	B3-19-19.5 B3-19.5					50	130.3	9.7
20					END OF BORING AT APPROXIMATELY 20 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS AND GROUT			

Figure A4, Log of Boring B3, Page 1 of 1

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

BORING B4

ELEV. (MSL.) ____________ DATE COMPLETED **12/21/2021**

ENG./GEO. **JBM** DRILLER **Cenezoic**

EQUIPMENT **Simco 2400-SK1 w/ 6" SFA** HAMMER TYPE **Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 2 inches asphalt			
1				CL-ML	Approximately 6 inches aggregate base			
2	B4-1.5-2 B4-2				FILL Stiff, damp, brown, Silty CLAY with (f) sand and few (f) rounded gravels -pp=3-4½	20	124.8	9.6
3				CL	ALLUVIUM Medium stiff, moist, brown, (f) Sandy CLAY with trace (f) rounded gravels			
4–5	B4-4-4.5 B4-4.5					7	117.7	16.0 14.3
6					-pp=1½			
7					-hard			
9–10	B4-9-9.5 B4-9.5				-some sandstone fragments -damp	65	121.5	8.7
14	B4-14-14.5 B4-14.5			SC	Dense, moist, brown, Clayey (f-c) SAND with few sandstone fragments	70		8.9
15					END OF BORING AT APPROXIMATELY 15 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS, GROUT AND AC PATCH			

Figure A5, Log of Boring B4, Page 1 of 1

GEOCON BORING LOG W/FIG# STARTING W/ A2 E9291-04-01 BORING LOGS.GPJ 05/18/22

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B5

ELEV. (MSL.) ______ DATE COMPLETED **12/21/2021**

ENG./GEO. **JBM** DRILLER **Cenezoic**

EQUIPMENT **Simco 2400-SK1 w/ 6" SFA** HAMMER TYPE **Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Approximately 2 inches asphalt			
1	B5-1-5			SC-SM	Approximately 6 inches aggregate base			
2	B5-1.5-2 B5-2				FILL Loose, moist, brown, Silty Clayey (f) SAND with few (f) gravels -rootlets	10		12.8
3				CL	ALLUVIUM Very stiff, moist, brown, CLAY with sand and gravel			
4	B5-4-4.5 B5-4.5				-pp>4½	29	121.1	8.8
5								

END OF BORING AT APPROXIMATELY 5 FEET
NO FREE WATER ENCOUNTERED
BACKFILLED WITH COMPACTED CUTTINGS AND AC PATCH

Figure A6, Log of Boring B5, Page 1 of 1

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL
... DISTURBED OR BAG SAMPLE
... STANDARD PENETRATION TEST
... CHUNK SAMPLE
... DRIVE SAMPLE (UNDISTURBED)
... WATER TABLE OR SEEPAGE

BORING B6

ELEV. (MSL.) ____________ DATE COMPLETED **12/21/2021**

ENG./GEO. **JBM** DRILLER **Cenezoic**

EQUIPMENT **Simco 2400-SK1 w/ 6" SFA** HAMMER TYPE **Cathead**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B6-0-5			CL-ML	FILL Stiff, damp to moist, brown, Silty CLAY with (m-c) sand			
1								
2	B6-1.5-2			CL	ALLUVIUM Stiff, moist, brown, CLAY with little (m-c) sand and trace (f) gravels	20		
	B6-2						124.1	11.2
3								
4	B6-4-4.5					31		17.6
	B6-4.5				-very stiff, trace to no sand and gravel -pp=2½-3 -more sand		112.5	16.0
5								

END OF BORING AT APPROIXMATELY 5 FEET
NO FREE WATER ENCOUNTERED
BACKFILLED WITH COMPACTED CUTTINGS

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

APPENDIX B

APPENDIX B
LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, grain size distribution, plasticity, expansion index, unconfined compressive strength, and screening-level corrosion parameters. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and moisture content test results are included on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B1-1.5-2	21	16	5
B3-0-5	26	17	9
B4-1.5-2	25	18	7
B5-1.5-2	22	17	5

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density* (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B3-0-5	9.3	14.4	113.7	6
B5-1-5	9.9	16.9	108.5	3

*Before saturation.

TABLE B-III
SUMMARY OF LABORATORY GRAIN SIZE ANALYSIS – NO. 200 WASH
ASTM D1140

Boring No.	Sample Depth (feet)	Fraction Passing No. 200 Sieve (%)
B1	4-4.5	33
B2	2	43
B4	4-4.5	62
B5	2	47
B6	4-4.5	74

APPENDIX B
LABORATORY TESTING (cont.)

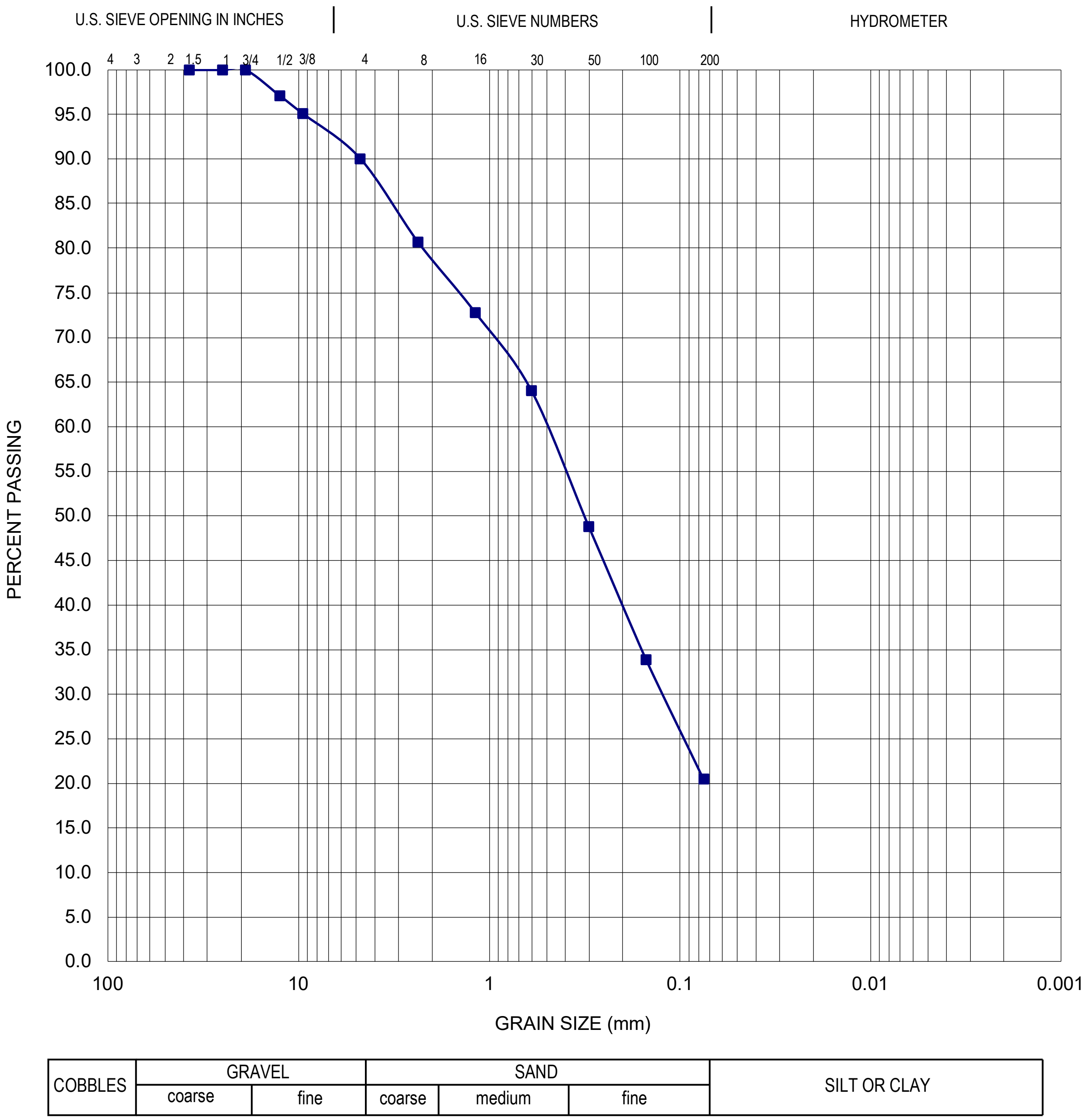
TABLE B-IV
SUMMARY OF SOIL CORROSION PARAMETERS
(CTM 643, CTM 417, CTM 422)

Boring No. (sample depth in feet)	Soil Type (USCS Classification)	Resistivity (ohm-cm)	pH	Chloride (ppm)	Sulfate (ppm)
B2(1.5-2)	Silty Clayey SAND(SC-SM)	3,200	7.1	66	5
B3(1.5-2)	Silty Sandy CLAY(CL-ML)	3,000	7.2	25	1

*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- The pH is equal to or less than 5.5.
- Chloride concentration is equal to or greater than 500 parts per million (ppm) or 0.05%.
- Sulfate concentration is equal to or greater than 1,500 ppm (0.15%)

**According to the American Concrete Institute 318 Chapter 19, Type II cement may be used where sulfate levels are below 2,000 ppm (0.2%)



Boring: B2

Depth To Sample: 9 - 9.5'

Sieve Date: 1/10/2021

Tested and Computed by: BO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	97.1	95.1	90.0	80.7	72.8	64.0	48.8	33.9	20.4

Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550
Telephone: (925) 371-5900
Fax: (925) 371-5915

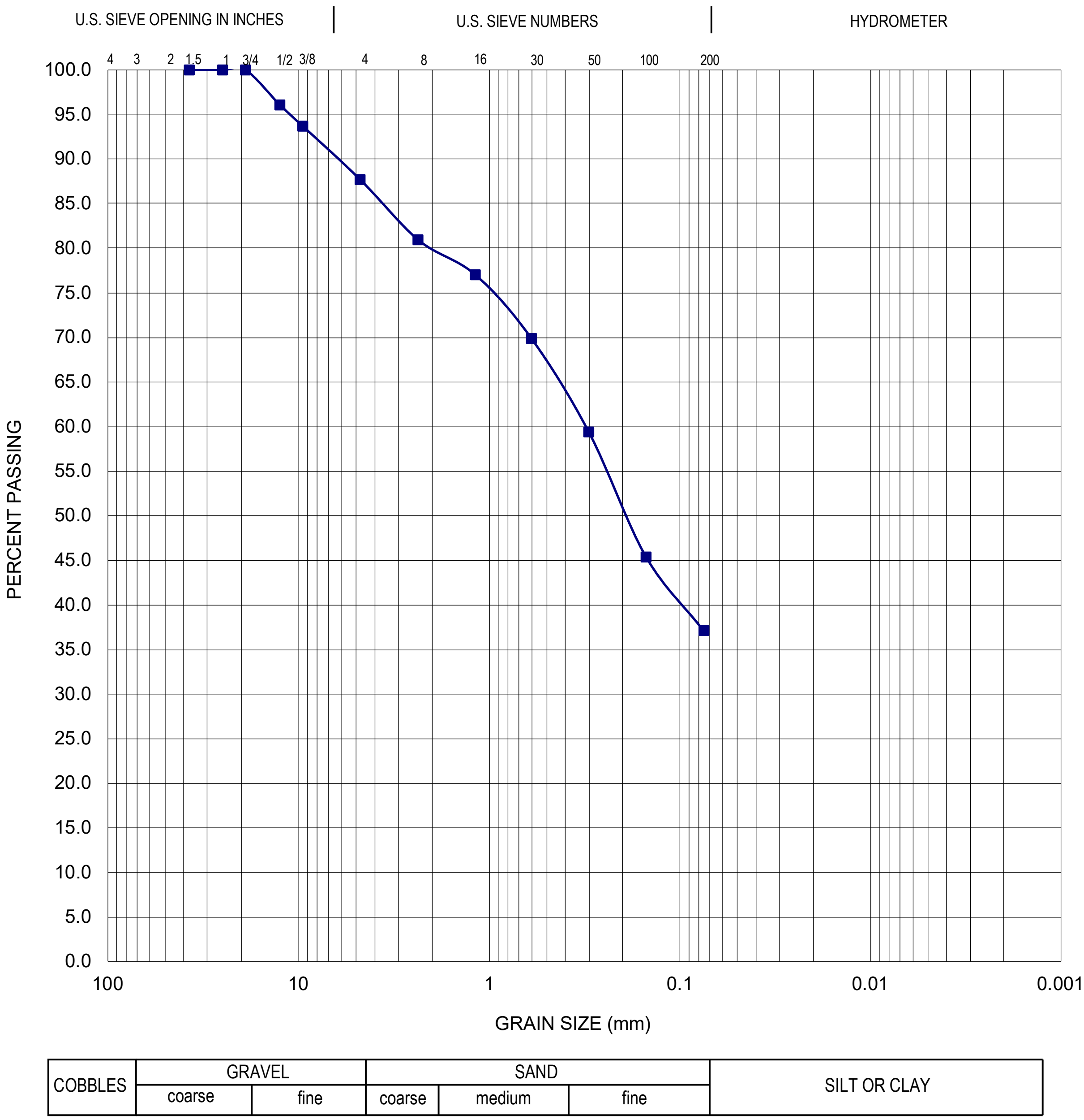
Particle Size Analysis - ASTM D422

Project: Butterfield FS

Location: Morgan Hill, CA

Project No.: E9291-04-01

Figure B1



Boring: B4

Depth To Sample: 14 - 14.5'

Sieve Date: 1/10/2021

Tested and Computed by: BO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100	100	100	96.0	93.7	87.7	80.9	77.0	69.9	59.4	45.4	37.1

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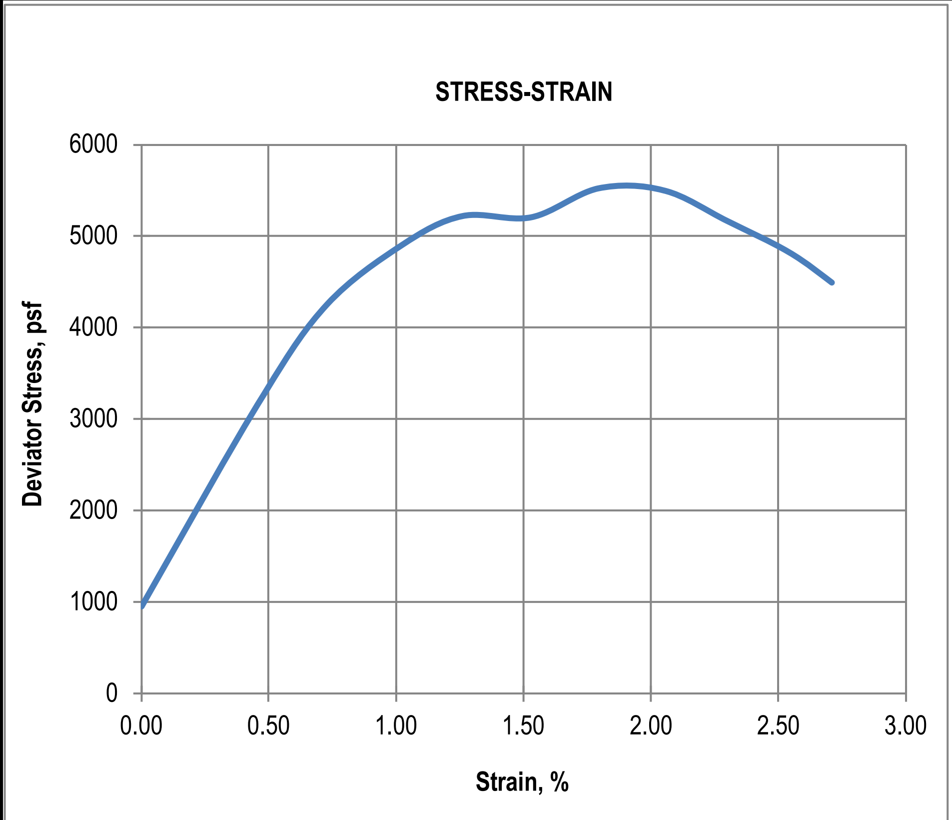
Particle Size Analysis - ASTM D422

Project: Butterfield FS

Location: Morgan Hill, CA

Project No.: E9291-04-01

Figure B2



Failure Photo

Sample Description	
Boring Number	B3
Sample Depth (feet)	2'
Material Description	brown & tan sandy CLAY w/ sandstone chunks
Initial Conditions at Start of Test	
Height (inch) average of 3	5.87
Diameter (inch) average of 3	2.38
Moisture Content (%)	8.9
Dry Density (pcf)	128.9
Estimated Specific Gravity	2.7
Saturation (%)	78.4
Shear Test Conditions	
Strain Rate (%/min)	0.4930
Major Principal Stress at Failure (psf)	5520
Strain at Failure (%)	1.8
Test Results	
Unconfined Compressive Strength (tons/ft^2)	2.8
Unconfined Compressive Strength (lbs/ft^2)	5520
Shear Strength (tons/ft^2)	1.4
Shear Strength (lbs/ft^2)	2760

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Unconfined Compressive Strength (ASTM D2166)

Project: Butterfield FS
Location: Morgan Hill, CA
Proj. No.: E9291-04-01

Figure B3

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