Appendix C

Geotechnical Investigation

Geotechnical Investigation
Residential Development
17965 Monterey Road
Morgan Hill, California

Report No. 371754 has been prepared for:

First Community Housing
75 East Santa Clara Street, Suite 1300
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GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT
17965 MONTEREY ROAD
MORGAN HILL, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Residential Development to be located at 17695 Monterey Road in Morgan Hill, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project.

We received and reviewed a site plan titled, "ALTA/NSPS Land Title Survey EAH Inc., a California Non-Profit public benefit corporation, City of Morgan Hill, County of Santa Clara, State of California," prepared by MH Engineering Co. dated April 9, 2019.

1.1 Project Description

The site consists of an approximately 1.5-acre vacant lot. The site is bounded by Monterey Road to the northeast, a mobile home park to the northwest, a parking lot to the southwest, and an existing commercial development to the southeast.

Based on the information provided, we understand that the project will consist developing the site with a multi-family residential structure(s) with several stories above grade and a slab-on-grade floor. We anticipate that minor grading will be required. Additional improvements will include exterior flatwork, pavements, and landscaping.

Structural loads have not been provided to us; therefore, we assumed that structural loads will be representative for this type of construction.

1.2 Scope of Services

Our scope of services was presented in our proposal dated November 18, 2019. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling two borings in the area of the proposed improvements and retrieving samples for observation and laboratory testing. We also advanced three Cone Penetration Tests (CPTs).
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate structure foundations, site earthwork, slabs-on-grade, retaining walls, and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Staff Engineer performed a reconnaissance of the site on December 9, 2019. At the time of the reconnaissance, the site was relatively flat with minor grade variation for drainage purposes. Additionally, the exploration locations were marked, and notification was provided to Underground Service Alert (USA) prior to beginning fieldwork to identify public and/or private underground utilities. We also contracted a private utility locator to reduce the risk of damaging unidentified underground utilities.

2.2 Exploration Program

Subsurface exploration was performed on January 2, 2020 using conventional, truck-mounted hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. Two hollow-stem auger exploratory borings were drilled to a depth of 45 feet. Subsurface exploration was also performed on December 12, 2019 using CPT equipment to investigate subsurface soils. Three CPTs were advanced to a depth of up to 45 feet.

Our borings and CPTs were performed and backfilled in accordance with Santa Clara Valley Water District guidelines. The approximate locations of our borings and CPTs are shown on the Site Plan, Figure 2. The logs of the borings and CPTs and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

In general, soils encountered in our CPTs were interpreted to include interbedded layers of clay, silty clay, clayey silt, sandy silt, and silty sand to a depth of approximately 45 feet.

Our borings generally encountered medium stiff to hard lean clay, and hard lean clay to a depth of approximately 12 feet. Below the depth of 12 feet, our borings generally encountered interbedded layers of dense to very dense clayey sand, very dense poorly graded gravel with clay and sand, very dense clayey gravel with sand, and hard lean clay with sand to a depth of approximately 27 feet. Below the depth of 27 feet, our borings encountered interbedded layers of very stiff to hard lean clay, and hard sandy lean clay with gravel to a depth of 45 feet, the maximum depth explored.

Two Plasticity Index (PI) tests were performed on clay soil samples from borings EB-1 and EB-2 collected at depths of approximately 2 and 3½ feet, which resulted in PI's of 7 and 17, respectively indicating low plasticity to moderate expansion potential of the near-surface soils.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in all our borings at depths of approximately 23 and 27½ feet. Based on pore pressure dissipation measurements, our CPT's inferred ground water at depths of approximately 13½ and 16 feet. Based on the depth to historically high ground water map prepared by the California Geological Survey for the Morgan Hill Quadrangle (CGS, 2004), the depth to historically high ground water levels in the site vicinity is on the order of 20 feet below the existing ground surface (bgs). We judge a ground water depth of 13 feet to be appropriate for design and construction. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone), or a Santa Clara County Fault Rupture Hazard zone (SCC, 2012). No known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

The seismic history of the region and studies of recurrence intervals of major faults indicate the site will experience strong ground shaking from a significant earthquake during the design life of the planned development. We performed a ground motion hazard analysis, in accordance with the American Society of Civil Engineers (ASCE) 7-16. Please refer to Appendix C for the ground motion hazard analysis.

In general, it is our opinion that the site subsurface profile for the project is consistent with Site Class D classification. In accordance with the ASCE 7-16 for sites classified as D, the average soil shear wave velocity in the upper 100 feet is 600 to 1,200 feet per second, which is consistent with the results of our estimated average shear wave velocity measurements based on our explorations.

Based on the results of the ground motion hazard analysis and on Equation 11.8-1 of ASCE 7-16, a maximum considered earthquake geometric mean peak ground acceleration of 0.86g can be expected to occur at the site during a design level earthquake.

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is not located within an area zoned by the State of California for seismically induced liquefaction hazard (CGS, 2002). The site is also not located within an area zoned by the Santa Clara County Geologic Hazard Zones maps as a Liquefaction Hazard Zone (2012). During cyclic ground shaking, such as earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

Based on our explorations, no loose to moderately dense non-cohesive soils were encountered below the design ground water depth of 13 feet. Therefore, we judge the risk of liquefaction at the project site to be low.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. Our explorations did not encounter any cohesionless soil layers above the design groundwater level. Therefore, we judge the probability of significant differential settlement of non-saturated cohesionless soil layers at the site to be low.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable since it is difficult to evaluate where the first tension crack will occur.

Coyote Creek is located approximately 2 miles north of the site. Because of the low probability for liquefaction, the probability of lateral spreading occurring at the site during a seismic event is low.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted two samples collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 1 below.

Table 1. Results of Corrosivity Testing

Sample	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
EB-1, 2A	3.5	5	105	6.5	3,206	Moderately	Negligible
EB-2, 1B	2.0	58	89	8.4	5,067	Mildly	Negligible

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 2 below.

Table 2. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 3.

Table 3. Relationship Between Sulfate Concentration and Sulfate Exposure

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1]= seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 1, the soil resistivity results were 3,206 and 5,067 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 2, the corrosion potential to buried metallic improvements may be characterized as mildly to moderately corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed improvements may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

5.1 Primary Geotechnical Concerns

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Moderately expansive near surface soils
- Corrosion potential of the near-surface soils

We have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the improvements.

5.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed improvements be designed in accordance with the seismic design criteria provided in the site-specific ground motion hazard analysis included in Appendix C.

5.1.2 Moderately Expansive Soils

To reduce the potential for damage to the planned structures due to the presence of moderately expansive surficial soils, we recommend slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill and that any shallow foundations extend below the zone of seasonal moisture fluctuation. Detailed recommendations are presented in the following sections of this report.

5.1.3 Corrosion Potential of Near-Surface Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the soils may be characterized as mildly to moderately corrosive. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipefittings.

5.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings and CPTs may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The proposed project areas should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

6.2 Removal of Undocumented Fill

If undocumented fill is encountered, it should be removed down to the native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. Side slopes of fill removal excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed improvement areas should be removed in their entirety. Utilities within the proposed improvement area would only be considered for in-place abandonment provided they do not conflict with new improvements, if the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building area should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill or pavements should be scarified to a depth of 12 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill material should be inorganic, have a PI of 20 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Non-expansive fill (NEF) should have a PI of 15 or less. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.6 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content 2 to 3 percent over the laboratory optimum. The native soils should be compacted between 87 and 92 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content at least 3 percent over the laboratory optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native clays, which should be compacted as noted above. Aggregate base should be compacted at a moisture content near the laboratory optimum moisture content. Import soils with a PI between 15 to 20 should be compacted at a moisture content at least 3 percent over optimum.

6.7 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. The contractor should be aware that soils at the bottom of the excavation may contain high moisture content. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,

- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,

- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

6.8 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements of the governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided, they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. If native moderately expansive soil is used for trench backfill, it should be compacted to between 87 to 92 percent at a moisture at least 3 percent over optimum. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with expansive subgrade soils.

6.9 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type C based on soil classification by OSHA. Therefore a maximum slope 1.5:1 (horizontal:vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

6.10 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.11 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or drip irrigation systems,
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeters.

We recommend that the landscape architect consider these items when developing the landscaping plans.

6.12 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

7.0 FOUNDATIONS

We recommend that the proposed structures be supported on shallow foundations, provided the estimated settlements discussed below are acceptable. Recommendations for shallow foundations are presented in the sections below.

7.1 Footings

The proposed structures may be supported on conventional spread footing foundations bearing on natural, undisturbed soil or compacted engineered fill. All footings should have a minimum width of at least 18 inches and footing bottoms should extend at least 18 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Footings constructed on native soil or engineered fill in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. We should observe all footing excavations before reinforcing steel is placed.

7.1.1 Footing Foundation Settlement

Structural loads were not available for our review at the time of our investigation. Therefore, we assumed interior column loads on the order of 300 kips and using the maximum allowable bearing pressures recommended above, we estimate that total static settlement for footings will be up to approximately 1-inch, with differential settlements of ½-inch over a horizontal distance of 50 feet. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

7.1.2 Lateral Loads on Footings

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade. A maximum allowable frictional resistance of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

7.1.3 Slabs-on-Grade

We recommend concrete slabs used in conjunction with shallow footings be supported on at least 6 inches of non-expansive fill (NEF). NEF may include aggregate base, crushed rock, quarry fines or import soil having a PI of 15 or less. We also recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Areas to receive special attention include slab joints and areas where building columns pass through the floor slab.

If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Protection Considerations" section of this report should be considered.

Post-construction cracking of concrete slabs-on-grade is inherent in any project. In our opinion, consideration should be given toward a maximum control joint spacing of 10 to 15 feet in both directions for the interior slab-on-grade construction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

7.2 Moisture Protection Considerations

Since the long-term performance of concrete slabs-on-grade foundations depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with the slab-on-grade construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the slab. The vapor barrier should extend to the edge of the slab. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break (no sand). The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted that the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

8.0 RETAINING WALLS

8.1 Lateral Earth Pressures

Any proposed retaining walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

8.2 Seismic Lateral Earth Pressures

Walls greater than 6 feet in height need to be designed for seismic lateral loading. For our analysis, we have assumed that the walls will have flat, non-sloping backfill. We used the Mononobe-Okabe approach to approximate the increased earth pressures induced by earthquakes. As discussed in Section 3.2 of our report, a peak ground acceleration of 0.86g is expected at the site. We performed calculations using this ground acceleration and estimated an additional seismic increment of 25.2 pcf to be applied to in addition to the static lateral earth pressures given in Section 8.1 for flexible walls. For restrained walls, under seismic conditions the total pressure to be used in analysis (seismic plus static) should be the greater of at-rest pressure or the sum of the active pressure and the seismic increment acting in a triangular distribution. For unrestrained walls under seismic loading, the total pressure should be sum of the active pressure and the seismic increment acting in a triangular distribution.

8.3 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½- to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

8.4 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced

8.5 Foundation

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Footings" section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the "Lateral Loads."

9.0 PAVEMENTS

9.1 Asphalt Concrete

Based on the near-surface soils encountered during our explorations, which generally consisted of lean clay, we judged an R-value of 10 to be applicable for design based on a subgrade consisting of untreated on-site soils. Using estimated traffic indices for various pavement-loading requirements and untreated on-site soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 4.

Table 4. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 10

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking Channel	5.0	3.0	9.0	12.0
	5.5	3.0	11.0	14.0
Truck Access & Parking Areas	6.0	3.5	11.5	15.0
	6.5	4.0	13.0	17.0

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are moderately expansive, some increased maintenance and reduction in pavement life should be expected. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. Because of the presence of moderately expansive clay at the site, some increased amount of maintenance should be expected.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

In addition, it has been our experience that asphalt concrete pavements constructed over expansive soils and adjacent to non-irrigated open space areas may experience cracking parallel to the edge of the pavement. This is typically caused by seasonal shrinkage and swelling adjacent to non-irrigated edges of the pavement. The cracks typically occur within the first few years of construction and are typically located within a few to several feet of the edge of the pavement. The cracks, if they occur, can be filled with a bituminous sealant. Otherwise, a moisture barrier would need to be installed to a depth of at least 24 inches to reduce the potential for shrinkage of the pavement subgrade soils.

9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 5. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 5. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 10 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch (psi), and a modulus of rupture of at least 550 psi .In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

9.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

9.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 4 inches of NEF or Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report.

We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction.

10.0 LIMITATIONS

This report has been prepared for the sole use of First Community Housing, specifically for design of the proposed Residential Development in Morgan Hill, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between the borings and CPTs do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report assume that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

11.0 REFERENCES

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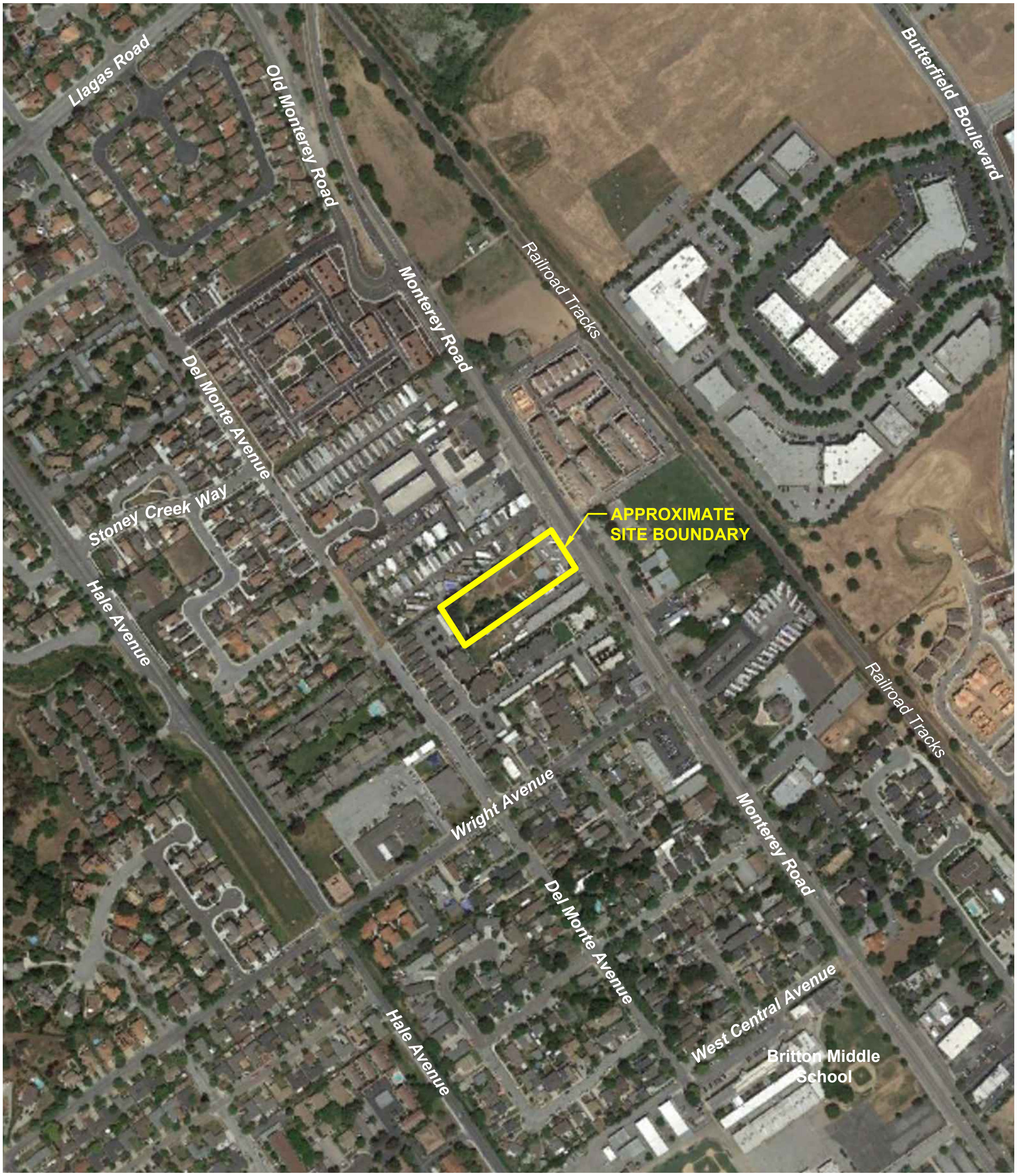
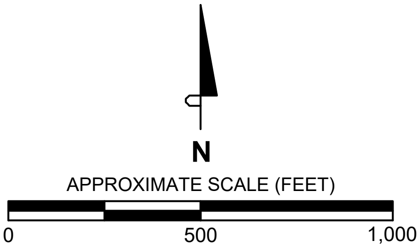


SOURCE AERIAL PHOTO: Google Earth, May 2018.



VICINITY MAP

Residential Development
17965 Monterey Road
Morgan Hill, California

TRC | 371754 | **FIGURE 1**

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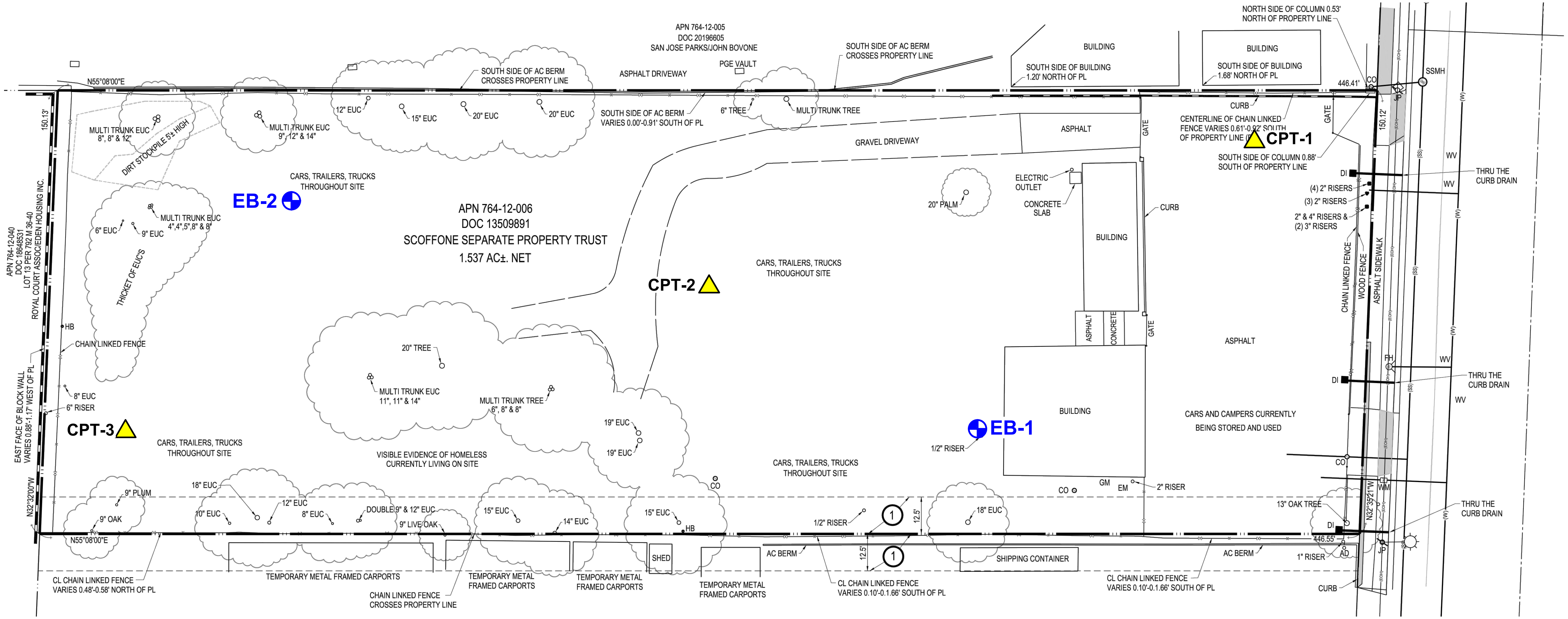
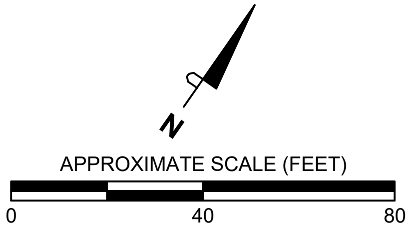


LEGEND

Approximate locations of:

△	Cone penetration test (CPT)
⊕	Exploratory boring



SOURCE: ALTA/NSPS Land Title Survey by MH Engineering Co., April 2019.

SITE PLAN

Residential Development
17965 Monterey Road
Morgan Hill, California

TRC | 371754 | **FIGURE 2**

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APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment, and cone penetration test (CPT) equipment. Two exploratory borings were drilled on January 2, 2020 to a maximum depth of 45 feet. Three CPTs were advanced on December 12, 2019 to a maximum depth of 45 feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the borings were not determined. The locations should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached borings and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel—sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel—sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel—sand—silt mixtures, plastic fines
			GC		Clayey gravels, gravel—sand—clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM		Silty sands, sand—silt—mixtures, non—plastic fines
			SC		Clayey sands, sand—clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS 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 of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT		Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE: 200 — 40 — 10 — 4

CLEAR SQUARE SIEVE OPENINGS: 3/4" — 3" — 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 — 0.4 — 2 — 5 — 19 — 76mm

GRAIN SIZES

⊠	TERZAGHI SPLIT SPOON STANDARD PENETRATION
⧗	MODIFIED CALIFORNIA
▯	ROCK CORE
■	PITCHER TUBE
○	NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0–1/4	0–2
SOFT	1/4–1/2	2–4
MEDIUM STIFF	1/2–1	4–8
STIFF	1–2	8–16
VERY STIFF	2–4	16–32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2–inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D–2487)

TRC

FIGURE A–1

EXPLORATORY BORING: EB-1

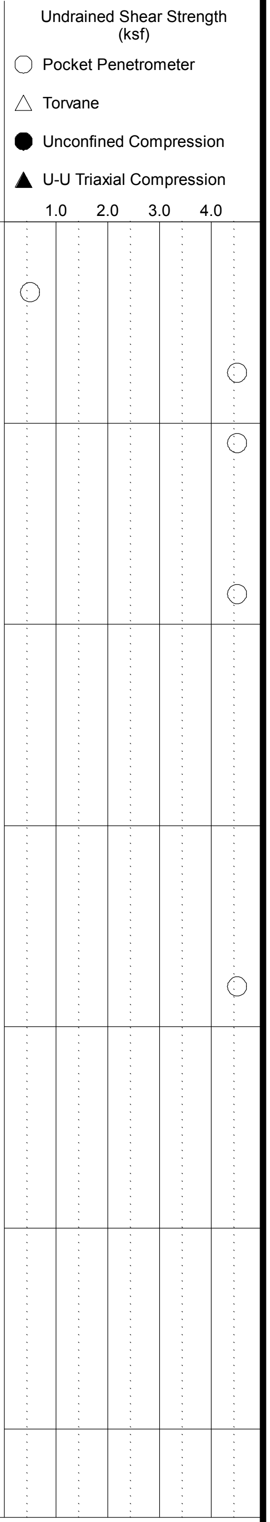
Sheet 1 of 2

DRILL RIG: TRUCK MOBILE B-53
BORING TYPE: 8-INCH HOLLOW-STEM AUGER
LOGGED BY: BM
START DATE: 1-2-20 FINISH DATE: 1-2-20

PROJECT NO: 371754
PROJECT: RESIDENTIAL DEVELOPMENT
LOCATION: MORGAN HILL, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0		**LEAN CLAY (CL)** medium stiff, moist, brown, low plasticity, trace fine to coarse sand Liquid Limit = 22, Plasticity Index = 7	CL	8		21	101	
			hard		30 50/6		16	101	
	5				33 50/6		15	115	
	10		trace fine gravel (subangular to subrounded)		52		17	112	
	15		**CLAYEY SAND WITH GRAVEL (SC)** dense, moist, brown, medium plasticity, fine to coarse sand, fine gravel (subangular to subrounded)	SC	58		9	109	
	20		**LEAN CLAY WITH SAND (CL)** hard, moist, brown, medium plasticity, fine to medium sand	CL	28 50/6		15	115	
	25		**POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC)** very dense, moist, brown to light brown, medium plasticity, fine to coarse sand, fine to coarse gravel (subangular to subrounded)	GP-GC	69				
⊽	30		**LEAN CLAY (CL)** hard, moist, brown, medium plasticity	CL	39				



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GROUND WATER OBSERVATIONS:
⊽ : FREE GROUND WATER MEASURED DURING DRILLING AT 27.5 FEET

LA_CORP.GDT 1/25/20 MV, CA*

EXPLORATORY BORING: EB-1 Cont'd

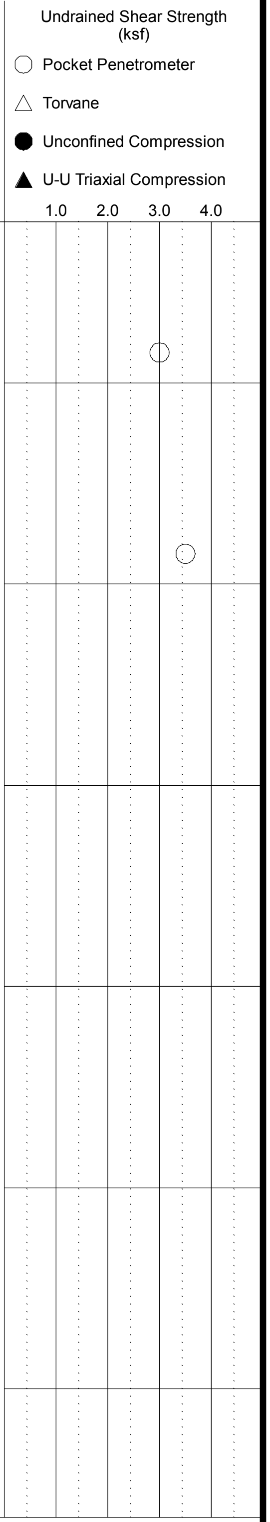
Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-53
BORING TYPE: 8-INCH HOLLOW-STEM AUGER
LOGGED BY: BM
START DATE: 1-2-20 FINISH DATE: 1-2-20

PROJECT NO: 371754
PROJECT: RESIDENTIAL DEVELOPMENT
LOCATION: MORGAN HILL, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
		▨	**LEAN CLAY (CL)** very stiff, moist, brown, medium plasticity, trace fine to coarse sand						
		▨			39	⧗	21	109	
	35	▨		CL					
		▨			42	⧗	26	99	
	40	▨							
		▨	**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, medium plasticity, medium to coarse sand	CL					
		▨			28	⧖			
	45		Bottom of boring at 45 feet						
	50								
	55								
	60								



GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 27.5 FEET

LA_CORP.GDT 1/25/20 MV, CA*

EXPLORATORY BORING: EB-2

Sheet 1 of 2

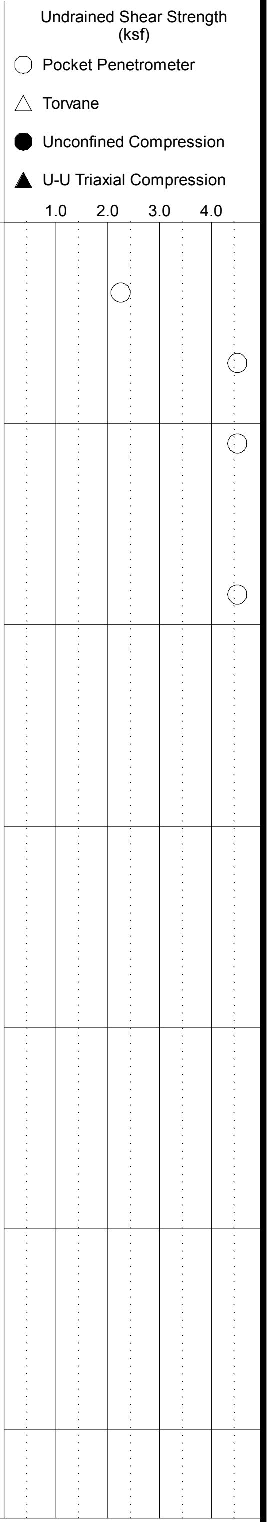
DRILL RIG: TRUCK MOBILE B-53
BORING TYPE: 8-INCH HOLLOW-STEM AUGER
LOGGED BY: BM
START DATE: 1-2-20 FINISH DATE: 1-2-20

PROJECT NO: 371754
PROJECT: RESIDENTIAL DEVELOPMENT
LOCATION: MORGAN HILL, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

SURFACE ELEVATION:

MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
LEAN CLAY (CL) very stiff, moist, brown, medium plasticity, trace fine to medium sand Liquid Limit = 32, Plasticity Index = 17	CL	35	17	108	
LEAN CLAY WITH SAND (CL) hard, moist, brown, medium plasticity, fine to medium sand	CL	28 50/5	13	111	
		38 50/6	10	114	
trace fine gravel (subangular to subrounded)		64	11	101	
CLAYEY SAND (SC) very dense, dry, brown, medium plasticity, trace fine sand	SC	28 50/5	13	114	
CLAYEY GRAVEL WITH SAND (GC) very dense, dry, brown, medium plasticity, fine to coarse sand, fine to coarse gravel (subangular to subrounded)	GC				
LEAN CLAY WITH SAND (CL) hard, dry, brown, medium plasticity, fine to coarse sand, trace fine gravel (subangular to subrounded)	CL	54			
CLAYEY GRAVEL WITH SAND (GC) very dense, moist, brown to dark brown, medium plasticity, fine to coarse sand, fine to coarse gravel (subangular to subrounded)	GC	73	13	113	
SANDY LEAN CLAY WITH GRAVEL (CL) hard, moist, brown, medium plasticity, fine to coarse sand, fine gravel (subangular to subrounded)	CL	34			



Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 23.0 FEET

LA_CORP.GDT 1/25/20 MV, CA*

EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE B-53
BORING TYPE: 8-INCH HOLLOW-STEM AUGER
LOGGED BY: BM
START DATE: 1-2-20 FINISH DATE: 1-2-20

PROJECT NO: 371754
PROJECT: RESIDENTIAL DEVELOPMENT
LOCATION: MORGAN HILL, CA
COMPLETION DEPTH: 45.0 FT.

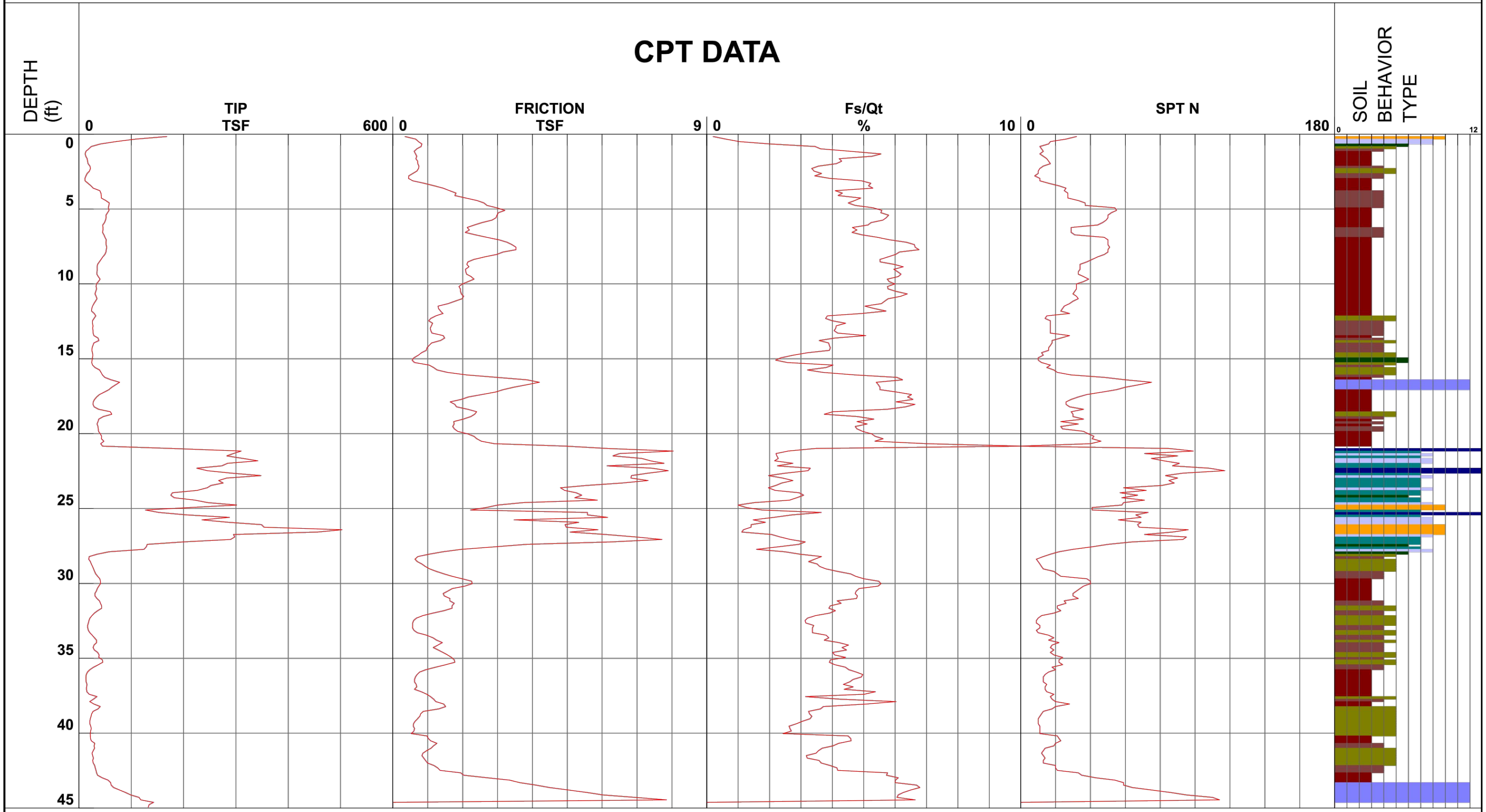
This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
		▨		CL						
		▨	**LEAN CLAY WITH SAND (CL)** very stiff, moist, brown, medium plasticity, fine to coarse sand	CL						
		▨			30	⧗	29	99		○
	35	▨								
		▨			51	⧗	27	98		○
	40	▨								
		▨	**LEAN CLAY (CL)** hard, moist, brown, medium plasticity, trace fine sand	CL						
		▨			48	⧖				
	45		Bottom of boring at 45 feet							
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 23.0 FEET

Net Area Ratio .8



CPT DATA

■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST
CPT-1

Residential Development
17965 Monterey Road
Morgan Hill, California

TRC | 371754 | CPT-1

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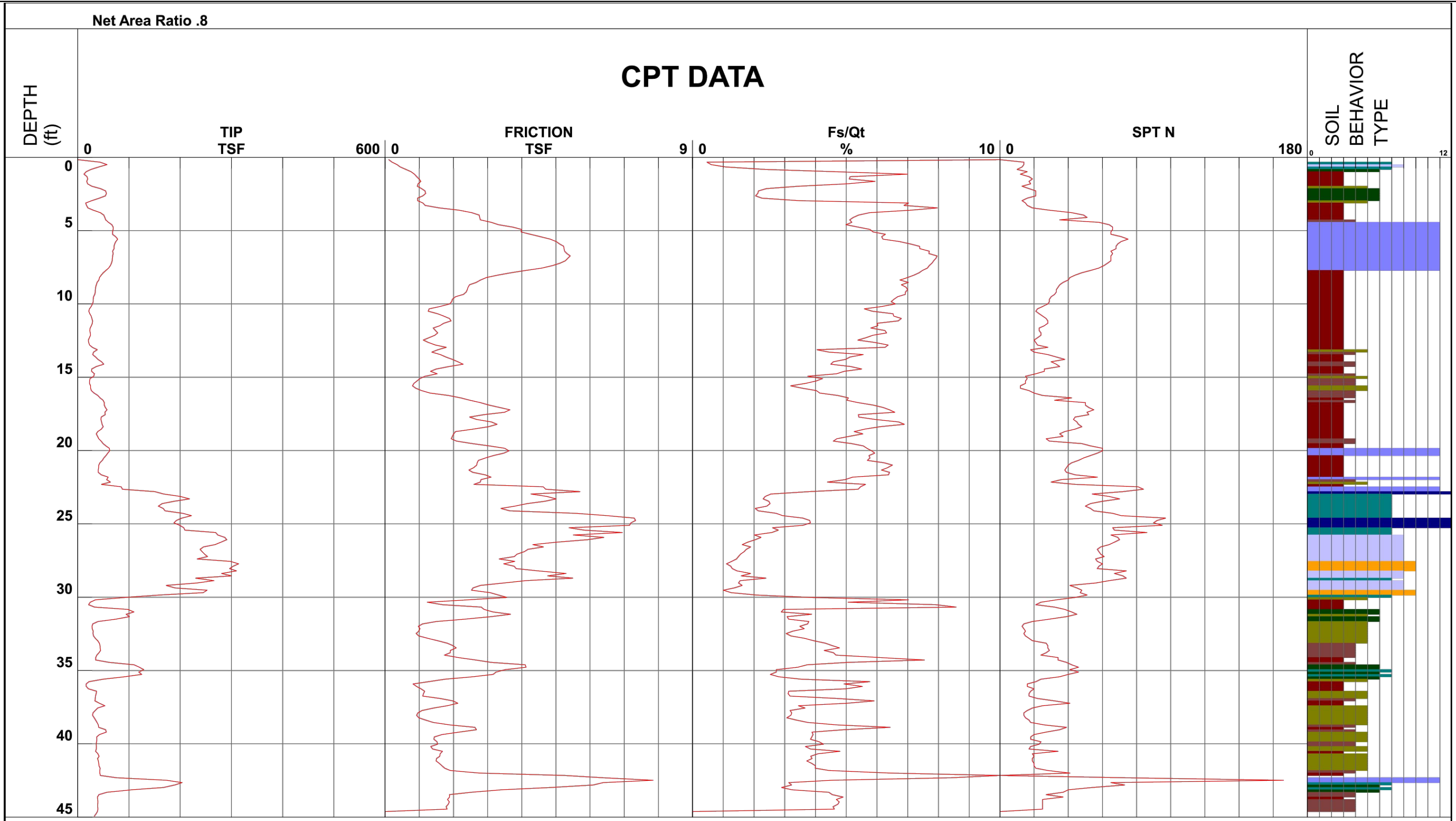

■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST
CPT-2

Residential Development
17965 Monterey Road
Morgan Hill, California

TRC | 371754 | **CPT-2**

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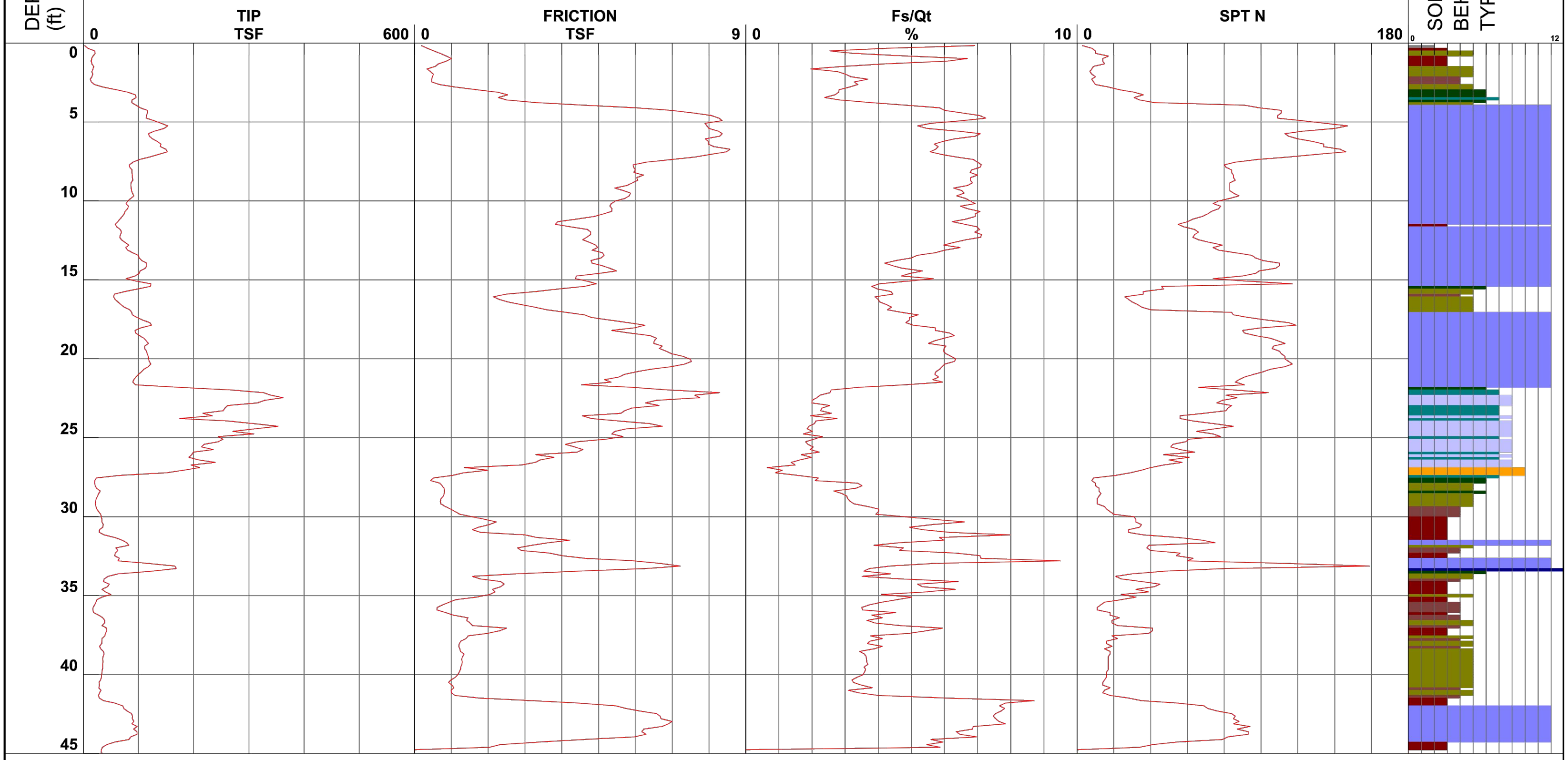
Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

CONE PENETRATION TEST
CPT-3

Residential Development
17965 Monterey Road
Morgan Hill, California

TRC | 371754 | **CPT-3**

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APPENDIX B

LABORATORY PROGRAM

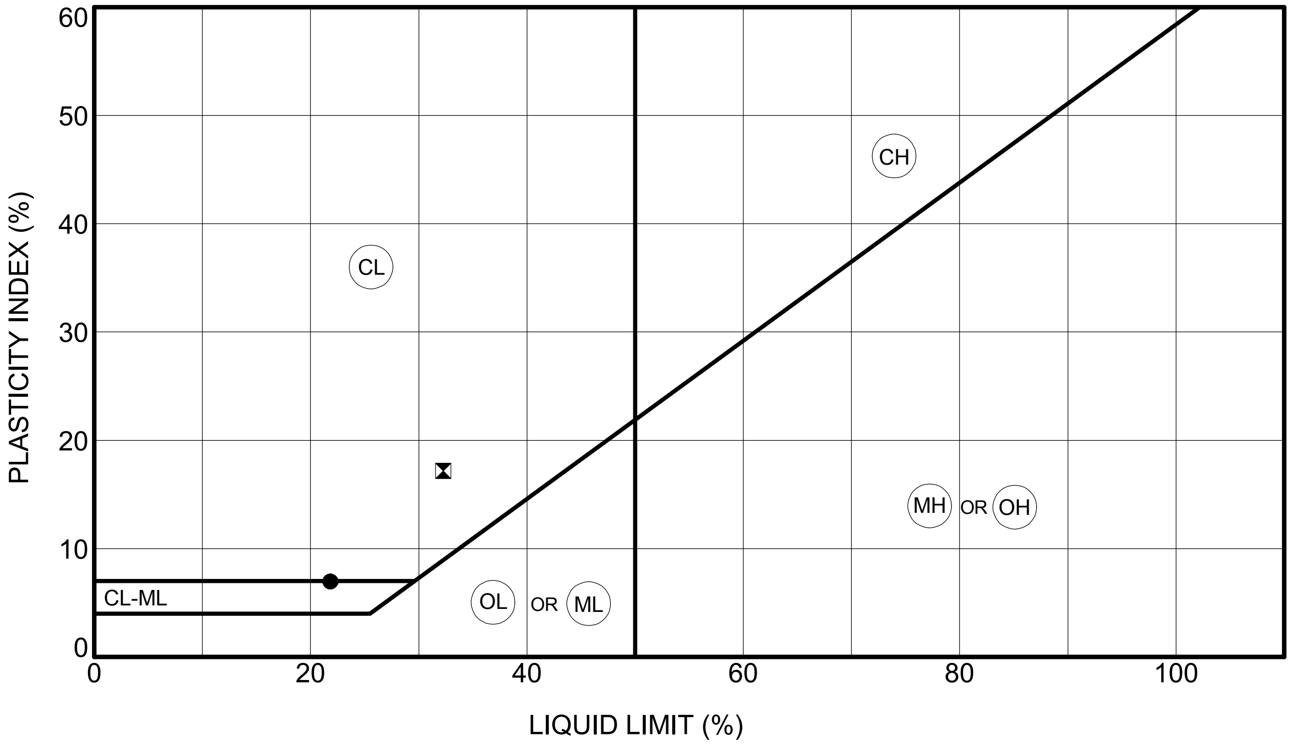
The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring log at the appropriate sample depths.

Plasticity Index: Plasticity Index (PI) test determinations (ASTM D4318) were performed on samples of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on the Plasticity Chart of this appendix and on the logs of the boring at the appropriate sample depths.

* * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-1**	**2.0**		**22**	**15**	**7**		
⧗	**EB-2**	**3.5**		**32**	**15**	**17**		

PLASTICITY CHART AND DATA

Project: RESIDENTIAL DEVELOPMENT

Location: MORGAN HILL, CA

Project No.: 371754

FIGURE B-1

LA_CORP.GDT 1/25/20 MV, CA*

COOPER
TESTING LABORATORY

Corrosivity Tests Summary

CTL # 028-2868 **Date:** 1/13/2020 **Tested By:** PJ **Checked:** PJ

Client: TRC **Project:** 17965 Mont. Rd Dev **Proj. No:** 371754

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	2A	3.5	-	-	3,206	5	105	0.0105	6.5	-	-	-	20.5	Dark Yellowish Brown Sandy CLAY
EB-2	1B	2.0	-	-	5,067	58	89	0.0089	8.4	-	-	-	17.4	Brown Clayey SAND

APPENDIX C

SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

TRC

1920 Old Middlefield Way
Mountain View, CA 94043

T 650.967.2365
TRCcompanies.com

January 27, 2020
371754

Mr. Michael Schaefer, AIA, LEED AP
Construction Manager
FIRST COMMUNITY HOUSING
75 East Santa Clara Street, Suite 1300
San Jose, California 95113

RE: **SITE SPECIFIC GROUND MOTION**
RESIDENTIAL DEVELOPMENT
17965 MONTEREY ROAD
MORGAN HILL, CALIFORNIA

Dear Mr. Schaefer:

We are providing the results of a site-specific ground motion analysis as described in the American Society of Civil Engineers (ASCE) Publication 7-16.

Method of Analysis

We performed a ground motion hazard analysis in general accordance with Chapter 21 of the ASCE document 7-16 titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," and the updated procedures as described in the ASCE document titled "Standard 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 1," with an effective date of December 12, 2018. The ASCE 7-16 requires the use of the 84th percentile of the deterministic Maximum Credible Earthquake (MCE) response spectra and requires that the maximum directional spectral acceleration shaking be used.

The intent of our analysis was to evaluate the site subsurface characteristics on site seismic ground response by using the attenuation relationships and to consider the effects of the local and regional geologic and geotechnical conditions related to site seismicity. Details regarding the attenuation relationship parameters used in the analysis are discussed below.

NGA Calculation Details

NGA response spectra calculations were performed using the computer software EZ-FRISK version 8.06 published by Risk Engineering, Inc. The recommended site response presented in this letter was developed by performing ground motion hazard analysis using the mean response of the four NGA relationships published by Abrahamson-et al (2014), Boore-et al (2014), Campbell-Bozorgnia (2014), and Chiou-Youngs (2014). The analysis included the application of the near-source directivity using the method based on Huang, Whittaker, and Luco (2008) to estimate the maximum rotated component from attenuation equations that evaluate the geometric mean of the horizontal components of ground motions, such as the NGA equations above. Details regarding our seismicity model, site soil profile, NGA calculation parameters, and rotation modification factors are discussed below.

Seismicity Model

2008 United States Geological Survey (USGS) fault models were used to evaluate both the probabilistic and deterministic seismic hazards.

Soil Profile

The site shear-wave velocity profile was based on measurements taken by field standard penetration resistance as described in the ASCE 7-16 Section 20.4. The site subsurface profile is judged to be consistent with Site Class D classification. Based on our subsurface exploration and local experience, we estimate that the depth to bedrock with a shear wave velocity of 1,000 meters per second is approximately 30 meters below the ground surface.

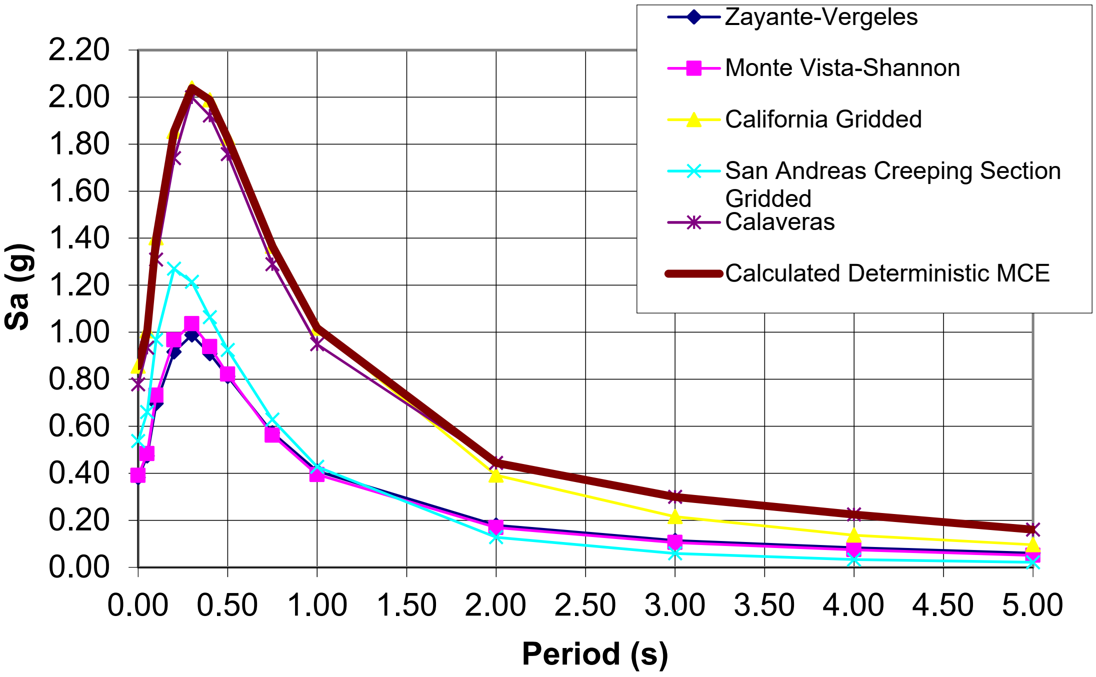
Calculation Parameters

For the NGA analysis, input parameters V_S30, Depth to V_s of 1,000 meters per second, and Z25, were taken as about 331 meters per second, 30 meters, and 0.63 kilometers, respectively. For both probabilistic and deterministic analyses, the mean result of the four attenuation relationships was used to determine the recommended MCE response spectrum.

Calculated Deterministic MCE

A summary of the response spectra from several characteristic earthquakes that are significant from a deterministic viewpoint is presented in Figure 1, below. The spectral accelerations shown on Figure 1 are the mean of the 84th percentile deterministic spectral response accelerations calculated using the four NGA equations (described above) that include the application of the near-source directivity using the method based on Huang, Whittaker, and Luco (2008). Additionally, the largest spectral response acceleration of the resulting deterministic ground motion response spectrum is greater than $1.5F_A$, therefore, scaling of the response spectrum was not applied. The California Gridded and Northern San Andreas faults, modeled as 7.0 and 8.05Mw earthquakes located 5.0 and 16.05 kilometers, respectively from the site, possess the largest hazards and control the deterministic MCE.

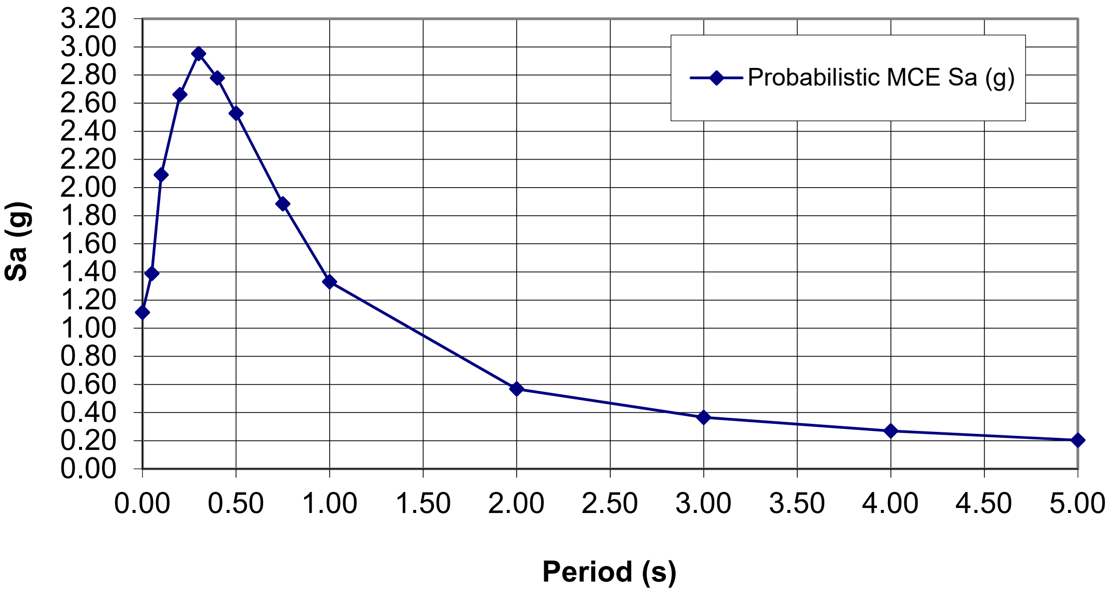
Figure 1. Deterministic MCE for Selected Faults 5% Damping



Probabilistic MCE_R

A summary of the 2 percent chance in 50 year horizontal acceleration response spectra is shown on Figure 2, below. The spectral accelerations shown on Figure 2 were calculated using the four NGA equations (described above) that included the application of the near-source directivity using the method based on Huang, Whittaker, and Luco (2008). Additionally, in accordance with Section 21.2.1.1 of ASCE 7-16, the spectral response accelerations were multiplied by the risk coefficient, $C_{R.}$ At spectral responce periods less than or equal to 0.2s, the value of C_R was taken as equal to $C_{RS.}$ At spectral responce periods greater than or equal to 1.0s, the value of C_R was taken as equal to $C_{R1.}$ At responce spectral periods between 0.2 seconds and less than 1 second, the value of C_R was based on linear interpolation of C_{RS} and C_{R1}.

Figure 2. Probabilistic MCE_R 5% Damping



Site Specific MCE_R

The site-specific MCE_R spectral response acceleration at any period, $S_{aM,}$ was taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1. and the deterministic ground motions of Section 21.2.2. Additionally, the site-specific MCE_R spectral response accelerations were adjusted so that spectral response acceleration at any period is not less than 150% of the site-specific design response spectrum determined in accordance with 21.3.

Recommended Site Specific Design

As discussed in Chapter 21.2.3 of ASCE 7-16, both a deterministic and probabilistic seismic hazard analysis were performed, with the lower of either the deterministic spectral acceleration or the 2 percent in 50-year exceedance spectral acceleration from the probabilistic analysis used to determine the MCE spectral acceleration for the site for 5 percent structural damping. The design site specific spectral response is the greater of 2/3 of the spectral response acceleration (S_{aM}) or 80 percent of the general mapped spectrum, as shown in Figure 3 below. The S_{aM} response acceleration is above the 80 percent general mapped spectrum, except for periods between 0.90s and 0.95s, therefore, the site specific design spectrum (S_a) is governed by a combination of the spectral response accelerations and the general mapped spectrum.

Figure 3. Site Specific Design

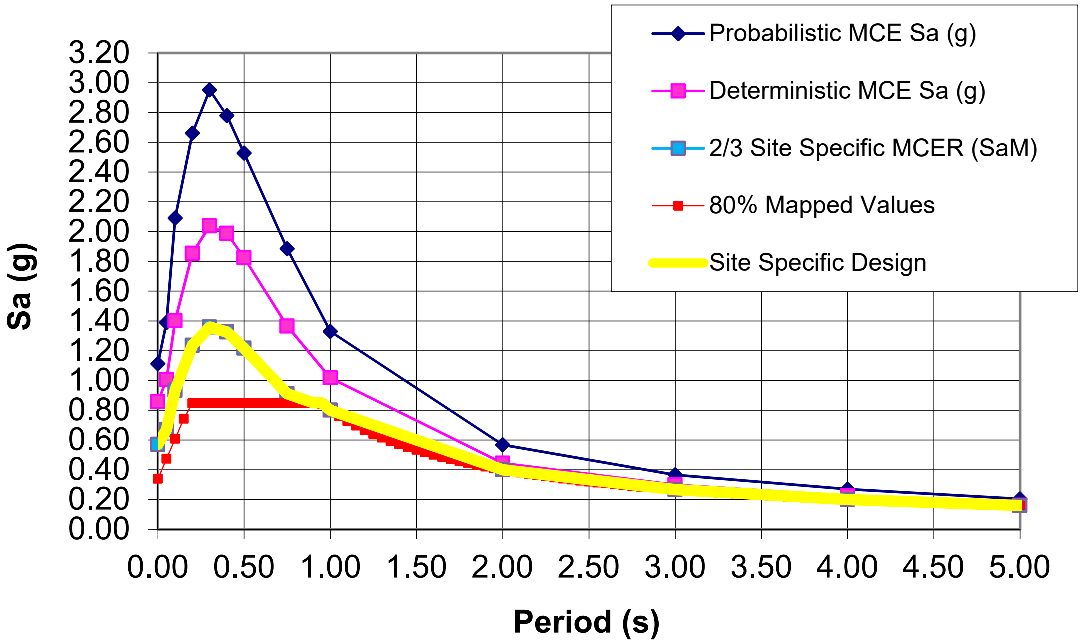


Table 1 below summarizes the details of our calculations including the calculated deterministic response spectrum, the calculated probabilistic response spectrum having a 2 percent chance of exceedance in 50 years, the site-specific MCE$_R$ spectral response accelerations, and the recommended site-specific design response spectrum.

Table 1. Summary of MCE Response Spectrum Calculations for 5 Percent Structural Damping

Period (sec)	Calculated Deterministic MCE	Probabilistic MCE$_R$	Site Specific MCE$_R$	Site Specific Design
0.00	0.86	1.11	0.86	0.57
0.05	1.01	1.39	1.01	0.67
0.10	1.40	2.09	1.40	0.93
0.20	1.85	2.66	1.85	1.24
0.30	2.04	2.95	2.04	1.36
0.40	1.99	2.78	1.99	1.33
0.50	1.83	2.53	1.83	1.22
0.75	1.37	1.88	1.37	0.91
1.00	1.02	1.33	1.20	0.80
2.00	0.44	0.57	0.60	0.40
3.00	0.30	0.37	0.40	0.27
4.00	0.22	0.27	0.30	0.20
5.00	0.16	0.20	0.24	0.16

Table 2, below, presents recommended S_{MS}, S_{M1}, S_{DS} and S_{D1} seismic coefficients for seismic design of the project.

Table 2. Recommended Seismic Coefficients

Seismic Design Coefficient	Value
S_{MS}	1.83
S_{M1}	1.20
S_{DS}	1.22
S_{D1}	0.80

Based on Section 21.5.3 of ASCE 7-16, a site specific maximum considered earthquake geometric mean peak ground acceleration (PGA_M) of 0.86g is the peak horizontal acceleration that can be anticipated to occur under a design level earthquake. Deaggregation of probabilistic seismic hazard indicates that a modal magnitude (M_w) for a 0.1 bin size is 6.25 and modal distance for a 2.5km bin size is 8.75 kilometers (EZ-FRISK 2015).

Seismic Design Category

We recommend the structural engineer determine the seismic design category using Table 2 above and an S_1 value of 0.60 in accordance with the 2016 ASCE 7-16.

For reference only, Table 3 below presents the 2016 ASCE 7-16 Site Class and Site Seismic Coefficients.

Table 3. ASCE 7-16 Site Class and Site Seismic Coefficients (for reference only)

Latitude: **37.1359 N** **Longitude:** **-122.6604 W**	CBC Table/ Figure	Factor/ Coefficient	Value
Soil Profile Type	Table 1613.3.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.3(1)	S_s	1.59
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.3(2)	S_1	0.60
Site Coefficient	Table 1613.3.3(1)	F_a	1.00
Site Coefficient	Table 1613.3.3(2)	F_v	Null – See Section 11.4.8
Adjusted MCE Spectral Response Parameter	Equation 16A-37	S_{MS}	1.59
Adjusted MCE Spectral Response Parameter	Equation 16A-38	S_{M1}	Null – See Section 11.4.8
Design Spectral Response Acceleration Parameter	Equation 16A-39	S_{DS}	1.06
Design Spectral Response Acceleration Parameter	Equation 16A-40	S_{D1}	Null – See Section 11.4.8

Closure

This letter has been prepared for the sole use of First Community Housing, specifically for design of the Residential Development in Morgan Hill, California. The recommendations presented in this letter have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this letter was written. No warranty, expressed or implied, is made or should be inferred.

If you have any questions concerning this letter, please call and we will be glad to discuss them with you.

Sincerely,

Scott M. Leck, P.E., G.E.
Principal Geotechnical Engineer

Attachment: References

References

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