CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Fifth Street Sewer Main Replacement**
Location	**East 5th Street, Diana Avenue and Depot Street** **Morgan Hill, California**
Client	**City of Morgan Hill**
Client Address	**17575 Peak Avenue** **Morgan Hill, California**
Project Number	**1267-1-1**
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TABLE OF CONTENTS

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of City of Morgan Hill for the Fifth Street Sewer Main Replacement project in Morgan Hill, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of plans titled, “City of Morgan Hill Improvement Plans for Fifth Street Sewer Main Replacement Project,” prepared by RJA Engineers, dated December 2020.

1.1 PROJECT DESCRIPTION

We understand that improvements to the existing sewer mains will occur along East 5th Street, cross Depot Street, and extend under the railroad tracks to connect with the existing sewer main in Diana Avenue in Morgan Hill, California. The planned improvements will consist of replacing three existing manholes and approximately 670 lineal feet (lf) of sanitary sewer main along East 5th Street and constructing approximately 1,100 lf of new sewer main, eight new manholes, and nine drain inlet sediment barriers between East 5th Street and Diana Avenue. Overlays of the existing street pavements are also planned. We understand the invert of the new sanitary sewer pipes will range from about 8 to 15 feet below the existing grades. The new sanitary sewer pipes will range from 12 inches in diameter along East 5th Street to 15 inches in diameter along Depot Street to Diana Avenue.

We understand that both open-cut and pipe bursting methods are being considered for replacement of the existing sewer pipes. For open cuts, trenches of up to 15 feet deep are anticipated for installation of the new pipelines. Additionally, we understand the sewer improvements will cross a Caltrain right of way (ROW) between Depot Street and Diana Avenue. We anticipate jack-and-bore methods will be used to cross under the railroad ROW.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated January 17, 2021, revised January 21, 2021, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations site work and grading, manhole foundations, temporary shoring/retaining walls, temporary dewatering, open-cut and trenchless methods, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of four borings drilled on February 9, 2021 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 19 to 30 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, a Plasticity Index test, washed sieve analyses, and preliminary soil screening corrosion testing. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3: UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earth- quake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Calaveras (South)	3.3	5.3
Sargent	7.4	11.9
Hayward (Southeast Extension)	9.6	15.5
Monte Vista-Shannon	10.6	17.0
San Andreas (1906)	10.9	17.5
Zayante-Vergeles	13.8	22.2

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

Based on historical aerial images provided on the Historic Aerials website (NETROnline, 2021), the site vicinity was generally occupied by agriculture fields, single-family residences and other structures in an image dated back to 1948. Monterey Road, East 5th Street, Depot Street, and the Caltrain railroad tracks are also visible in the 1948 image. Development on the south side of East 5th Street is visible in an image dated back to 2005. Based on the imagery, Diana Avenue and adjacent multi-family residences appear to have been constructed between the years 2012 and 2014. Significant changes to the site were not observed in the images dated after 2014.

3.2 SURFACE DESCRIPTION

The site is located within the roadways and cul-de-sacs of a residential area consisting of single-family homes, multi-family homes, and several commercial structures. The site is bounded by Diana Avenue to the northeast, Monterey Road to the southwest, and Depot Street to the northwest and southeast. Elevations at the site were referenced from the plans prepared by RJA Engineers (2020). The site is relatively level and near the elevation of the adjacent properties. The elevation at the site ranges from approximately Elevation 343½ along East 5th Avenue to 346 feet at Diana Avenue, North American Vertical Datum, 1988 (NAVD 88).

Surface pavements at our Exploratory Borings EB-1, EB-1A, and EB-2 along 5th Street generally consisted of 2 inches of asphalt concrete (AC) over woven pavement fabric over a second 3-inch-thick layer of AC placed directly on the underlying subgrade soils, aggregate base was not observed. Surface conditions at Boring EB-3 consisted of approximately 6 inches of gravel base (i.e. pavement was not encountered or observed). Surface pavement at Diana Avenue, Boring EB-4, consisted of 4½ inches of asphalt concrete over 8 inches of aggregate base. Based on our observations, the existing pavements are in good to poor condition.

3.3 SUBSURFACE CONDITIONS

3.3.1 East 5th Street (EB-1, EB-1A, & EB-2)

Beneath the surface pavements at Borings EB-1 and EB-1A, our exploration encountered very dense, silty gravel with sand, gravel, and cobbles to a depth of 5½ feet below the current grade (corresponding to Elevation 337½ feet); however, EB-1 encountered practical refusal at a depth of 2½ feet below the pavement surface (corresponding to Elevation 340½ feet) as a result of the presence of cobbles. The silty gravel in EB-1A was underlain by hard, sandy lean clay with gravel to a depth of 8½ feet (corresponding to Elevation 334½ feet). The sandy lean clay was underlain by very dense, clayey sand with gravel and cobbles to a depth of 16 feet (corresponding to Elevation 327 feet) and very dense, clayey gravel with sand and cobbles to a depth of 19 feet (corresponding to Elevation 324 feet), the terminal depth of the boring. As discussed above, significant gravels and cobbles were encountered during our field investigation, resulting in an initial shallow refusal when attempting EB-1.

Below the surface pavements at Boring EB-2, our exploration encountered a layer of silty sand with gravel and cobbles to a depth of 2 feet below the current grade underlain by very dense clayey sand with gravel and cobbles to a depth of 3 feet (corresponding to Elevation 342½ and 341½ feet, respectively). The clayey sand was underlain by hard, sandy lean clay with gravel to a depth of 8 feet (corresponding to Elevation 336½ feet) and very dense, clayey sand with gravel and cobbles to a depth of 17 feet (corresponding to Elevation 327½ feet). Below the clayey sand, our exploration encountered dense, well graded sand with silt, gravel, and cobbles to a depth of 20 feet, the terminal depth of the boring (corresponding to Elevation 324½ feet).

3.3.2 Depot Street (EB-3)

Below the surface layer of gravel base, Boring EB-3 encountered dense to very dense, clayey sand with gravel and cobbles to a depth of 27½ feet below current grades (corresponding to Elevation 315½ feet) underlain by very stiff, lean clay with sand to a depth of 30 feet, the maximum depth explored (corresponding to Elevation 313 feet).

3.3.3 Diana Avenue (EB-4)

Below the surface pavements, Boring EB-4 encountered very stiff, sandy lean clay to a depth of 3½ feet below current grades underlain by medium dense, clayey sand with gravel and cobbles to a depth of 13½ feet (corresponding to Elevation 343 and 333 feet, respectively). The clayey sand with gravel and cobbles was underlain by very stiff, sandy lean clay to a depth of 17½ feet and dense, clayey sand with gravel and cobbles to a depth of 22 feet (corresponding to Elevation 329 and 324½ feet, respectively). Below the clayey sand, our exploration encountered very dense, clayey gravel with sand and cobbles to a depth of 30 feet, the maximum depth explored (corresponding to Elevation 316½ feet).

3.3.4 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample of the subsurface soil at a depth of 14 feet below the existing grade. Test results were used to evaluate expansion potential of soils near the depths of the proposed improvements (i.e. the upper 15 feet below existing grades). The PI test resulted in a PI of 13, indicating low expansion potential to wetting and drying cycles.

3.3.5 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents likely to be encountered during the sewer main excavations, within the upper 10 feet, range from 2 to 3 percent below the estimated laboratory optimum moisture. The in-situ moisture contents of the material likely to be encountered during the jack and bore pits, within the upper 30 feet, range from 4 percent below to 3 percent above the estimated laboratory optimum moisture. Material that is above the estimated laboratory optimum moisture may need to be processed and dried out before being re-used as engineered fill.

3.4 GROUNDWATER

Groundwater was encountered in Boring EB-3 at a depth of 16 feet below current grades, corresponding to Elevation 327 feet (NAVD88). All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

The California Geologic Survey (CGS) maps historic high groundwater ranging from approximately 5 to 10 feet below the existing ground surface (Santa Clara 7.5-minute Quadrangle, 2004). We also reviewed nearby groundwater depth data obtained from the website GeoTracker (https://geotracker.waterboards.ca.gov/). Nearby monitoring well data indicates that groundwater has been measured at depths of approximately 8½ feet (Elevation 333½ feet) at wells located at 16995 Monterey Road on February 24, 2017.

Based on the above, we recommend a design groundwater depth of 8 feet below existing grade or approximately Elevation 335 feet. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.5 CORROSION SCREENING

We tested three samples collected at depths of 6 to 14 feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A.

Table 2A: Summary of Corrosion Test Results

Sample Location	Soil Type	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-1	Brown Clayey Sand (SC)	8.5	6.9	4,080	9	21
EB-2	Brown Clayey Sand (SC)	8.5	6.6	2,873	41	26
EB-4	Brown Clayey Sand (SC)	23.5	6.9	6,328	7	19

Notes:
[1] ASTM G51
[2] ASTM G57 - 100% saturation
[3] ASTM D3427/Cal 422 Modified
[4] ASTM D3427/Cal 417 Modified
[5] 1 mg/kg = 0.0001% by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute to corrosion potential.

3.5.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soil may be considered mildly to moderately corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-14 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-14, Table 19.3.1.1 below in Table 2B.

Table 2B: ACI 318-14 Table 19.3.1.1 Exposure Categories and Classes

Boring No./Soil Type	Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
EB-1 / SC	F0[1]	S0[2]	W1[3]	C1[4]
EB-2 / SC	F0[1]	S0[2]	W1[3]	C1[4]
EB-4 / SC	F0[1]	S0[2]	W1[3]	C1[4]

1 (F0) “Concrete not exposed to freezing-and-thawing cycles” (ACI 318-19)
2 (S0) “Water soluble sulfate in soil, percent by mass” is less than 0.10 (ACI 318-19)
3 (W1) “Concrete in contact with water and low permeability is required” (ACI 318-19)
4 (C1) “Concrete exposed to moisture but not to an external source of chlorides” (ACI 318-19)

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not currently mapped by the State of California, but the site is not located in a Santa Clara County Fault Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration $(PGA)_M$ was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2019 edition of the California Building Code. For our liquefaction analysis we used a PGA_M of 0.727g. We have assumed a site-specific analysis will not be required for this project; therefore, this is a code-based value of PGA_M. If a site-specific analysis is performed, this value may change.

4.3 LIQUEFACTION POTENTIAL

Based on the nature of the proposed improvements, i.e. non-habitable, we have assumed full geologic hazards evaluation is not required, including an exploration to 50 feet. In addition, the site is not located within a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). Therefore, a detailed liquefaction analysis was not performed.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to impact the proposed improvements at the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly dense to very dense sand and gravel, in our opinion, the potential for significant differential seismic settlement impacting the proposed improvements is low.

4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the southwestern portion of East 5th Street near Monterey Road is located within Zone AE, described as "special flood hazard areas subject to inundation by the 1% annual chance flood with a base flood elevation determined to be approximately 343 feet." The remainder of the site is within Zone X, described as "other flood areas, areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

As discussed, the existing sanitary sewer replacement is planned to be constructed by open-cut, pipe bursting, and jack and bore methods. Due to the existing stiffness and density of the subsurface conditions encountered, pipe bursting may be difficult in some locations due to the dense conditions and the presence of cobbles, we recommend contractors review the subsurface conditions in our boring logs to confirm the compatibility of pipe bursting and equipment proposed for use. If these methods are not feasible within various locations, open-cut methods could be performed; however, difficult excavation conditions (e.g. oversize materials) will likely be encountered. Descriptions and brief outlines of additional concerns to be addressed in the project design are listed below. Our general earthwork recommendations are provided in Section 6, following this section.

- Presence of very dense granular soil and cobbles
- Shallow groundwater
- Ground displacement and cracking
- Deflection of the jack-and-bore entry/exit pits shoring system
- Presence of existing utilities
- Residential construction areas
- Soil corrosion potential

5.1.1 Presence of Very Dense Granular Soil and Cobbles

As previously discussed in the "Subsurface" section of this report, we encountered dense to very dense, clayey sand with gravel and clayey gravel with sand and cobbles from one foot below the existing pavement to depths 30 feet below the existing grades (corresponding to Elevations 342 and 313 feet). Based on improvement plans prepared by RJA Engineers, we understand the planned sewer main improvements will include excavations into this material. Therefore, difficult excavation and shoring installation (e.g. driving sheet piles) conditions should be anticipated and planned for by the contractor. Additionally, and layers with very little fines can ravel easily during installation of temporary shoring, utility trench and jack-and-bore pit excavations, and other similar below-grade operations. We understand the excavations will extend to depths of up to 15 feet below current grades for the sewer mains and manholes and potentially up to 25 feet for the jack-and-bore pits. We recommend that trenching and shoring contractors review the subsurface conditions in our boring logs prior to bidding and selecting installation/drilling equipment and methods. We anticipate that open-cut will be used, at this time we do not anticipate trenchless methods (e.g. pipe bursting) being used for installation of new utilities, however if similar trenchless methods will be considered for installation of new utilities, these will be problematic due to subsurface conditions (i.e. cobbles and gravel), however we can provide further recommendations, if desired.

Detailed recommendations are provided in the “Earthwork” section of this report.

5.1.2 Shallow Groundwater

Groundwater was measured at a depth of approximately 16 feet below the existing ground surface in Boring EB-3 (approximate jack-and-bore pit location). As discussed, high groundwater exists at depths ranging from approximately 5 to 10 feet below the existing ground surface; therefore, we recommend a historic high groundwater depth of 8 feet be used for design. The groundwater may be either a static level or perched, or a combination thereof. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable excavation subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section of this report.

5.1.3 Ground Displacement and Cracking

The planned sewer pipe construction/replacement trenchless method at the Caltrain ROW has the potential risk of causing ground displacements that may damage existing utilities and surface improvements. For open-cut trenching, the shoring design and construction sequencing can address these potential risks.

We understand that a trenchless method, such as jack and bore, is being considered for the utility crossing of the railroad/Caltrain right of way. This method of sewer pipe construction has the potential risk of causing ground displacements that may damage existing utilities or structures. These potential risks can be addressed by the depth of cover over the horizontal alignment and by paying close attention to and monitoring the horizontal drilling pressures and the potential for ground loss; however, even with these precautions, some risks of ground displacement, settling or cracking remain.

5.1.4 Deflection of the Jack-and-Bore Entry/Exit Pits Shoring System

As discussed, we understand that a jack-and-bore method will be used to install the new sewer main under the existing Caltrain ROW. Existing utilities, pavements, and other improvements will likely be in close proximity to the jack-and-bore pits excavated to advance the casing in the undercrossing. Shoring systems should be designed with sufficient rigidity to limit detrimental deflections that result in movement of critical improvements. Good construction techniques should also be used to install and apply restraint, if necessary, in a timely manner. In no case should deflections exceed 1 inch.

5.1.5 Presence of Existing Utilities

Existing utilities appear to be in close proximity to the planned sanitary sewer upgrades, specifically at the undercrossing of the railroad ROW between Depot Street and Diana Avenue. An approximate amount of clearance, as discussed in detail in Section 4.1.4, is desirable to reduce the risk of damaging the existing utilities where installing the new sanitary sewer lines, particularly with pipe bursting methods.

5.1.6 Residential Construction Areas

The project site is located within an area of residential development where there will be concerns about construction noise and vibrations. For these reasons, our opinion is that, in addition to the difficulty of installation due to the dense gravely soils and cobbles, installing either steel piles or sheet piles using impact equipment has substantial risk of being a nuisance and difficult, and that slide rails, braced sheeting or similar methods may be preferred. Additionally, construction activities should be performed in accordance with the City’s construction ordinance requirements.

5.1.7 Soil Corrosion Potential

Our testing indicates sulfate exposure at the sites is low and therefore cement-type restrictions for buried concrete may not be needed. The corrosion potential for buried metallic structures, such as metal pipes, is considered mildly to moderately corrosive. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings. We recommend a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: TRENCHLESS METHOD CONSIDERATIONS

6.1 GENERAL

It is our understanding that a trenchless method consisting of jack-and-bore pits will be considered at the location of the new sewer main crossing the existing Caltrain ROW. Based on our understanding of the location of the new main alignment, the length, the subsurface and groundwater conditions, and the approximate invert depths, we expect that a trenchless method consisting of jack-and-bore may be used for the project; however, as discussed in Section 3, our borings encountered dense sand and gravel and cobbles. Therefore, casing will likely be needed for installation of the new sewer main. Additionally, difficult drilling conditions should be anticipated and planned for by the contractor. Recommendations for conventional open-trench methods are provided in the “Earthwork” section of this report.

6.2 POTENTIAL GROUND BEHAVIOR DURING INSTALLATION

A trenchless installation consultant should be retained to provide recommendations for trenchless pipe installation based on the subsurface conditions disclosed by our site investigation. It appears the ground conditions will consist primarily of sandy and gravelly materials with cobbles. The ground conditions should be closely reviewed prior to construction to determine the best method for completion of the new alignment. The potential for ground loss and ground behavior during installation should also be evaluated by the consultant.

6.3 CONTROL OF GROUNDWATER

Groundwater was encountered within our exploration at a depth of 16 feet, and historic high groundwater is estimated to be 8 feet below the existing ground surface. Therefore, it is anticipated that groundwater will impact the installation of the main at the Caltrain ROW.

6.4 CLEARANCE OF UTILITIES

We understand there are existing utility lines present along the proposed new sewer main alignment. An appropriate amount of clearance is desirable to reduce the risk of damaging the existing utilities when installing the new mains. The trenchless installation consultant should establish the minimum clearances of the existing utilities.

6.5 JACKING AND RECEIVING PITS

Vertical excavations on the order of about 25 feet are anticipated to construct the entry and exit pits for the installation of the new sewer main across the Caltrain ROW. These excavations will be made adjacent to existing utilities and city streets and, therefore, will require temporary support in order to avoid damaging the adjacent streets, sidewalks, utilities, and other improvements. We anticipate the excavation will predominately encounter sandy and gravelly soil with cobbles and will be below groundwater. Excavation of the pits should be readily accomplished with standard backhoes and excavators during or after shoring installations.

The Contractor should be responsible for all temporary slopes and design of any required shoring. The design of the shoring at entry and exit pits, as well as design of the jacking system, should be performed by a Registered Civil or Structural Engineer, retained by the Contractor, and submitted to the Engineer prior to its implementation. Shoring, bracing or temporary slopes should be performed by the Contractor in accordance with the strictest governing safety standards.

Vertical excavations may be temporarily shored using slide rail, braced shoring, or other shoring schemes, depending on the judgment of the shoring designer and Contractor. Based on the likely presence of dense sand and gravel and cobbles, sheet piling will likely be difficult or not feasible to install. The restrained earth pressure may also be distributed as described in Figure 24 of the FHWA Circular No. 4 – Ground Anchors and Anchored Systems.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils; however, dense sand and gravel and cobbles were encounter, which can create difficult conditions during sheet pile and soldier beam installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. The contractor is also responsible for evaluating the drilling conditions of the soils underlying the site and selecting equipment that is appropriate for the project.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created, they should be backfilled as soon as possible with sand, gravel, or grout.

We also recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

To prevent excessive surcharging of the walls from heavy construction vehicles, such as concrete trucks, we recommend that such vehicles be kept at least 15 feet from the top of the excavations; if this is not possible, the shoring must be designed to resist the additional lateral loads. In addition, all shoring schemes should be designed with sufficient rigidity to prevent detrimental displacements at the top of the shoring, particularly where excavations are completed adjacent to existing utilities, pavements or other improvements. At a minimum, the wall should be designed for a minimum surcharge of 240 psf for the upper 6 feet behind the wall to account for inadvertent surcharging. For a restrained wall, this would result in a minimum uniform lateral earth pressure of 120 psf in the upper 6 feet of wall. Where shaft or pit excavations are supported with temporary shoring, some settlement of the adjacent ground surface should be anticipated. If these shored excavations are placed in paved streets, some cracking and settlement of the adjacent pavements should be anticipated. Good design and construction techniques should greatly reduce these types of distress to improvements. The project specifications should require restoration of these damaged pavements, curbs, gutters, etc., to their preconstruction condition. A precondition survey of the area performed by the Contractor prior to construction, including photos, should be considered.

The above recommendations are for the use of the design team; the contractor, in conjunction with input from the shoring designer, should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.6 THRUST BLOCK DESIGN

Where a thrust block is required to transfer jacking loads into the soil, it shall be properly designed and constructed by the Contractor. Lateral resistance may be provided by passive pressures acting against the side of thrust blocks poured neat against competent soil. Assuming an average embedment of 10 to 15 feet, allowable uniform passive pressures of 500 to 900 psf in the design of thrust blocks, respectively. The thrust block shall be normal (square) with the proposed pipe alignment and shall be designed to withstand the maximum jacking pressure to be used with a factor of safety of at least 2.0. It shall also be designed to minimize excessive deflections in such a manner as to avoid disturbance of adjacent structures or utilities or excessive ground movement. If a concrete thrust block is utilized to transfer jacking loads into the soil, the tunnel boring is not to be jacked until the concrete or other materials have attained the required strength.

SECTION 7: EARTHWORK

The earthwork for this project is likely to consist of clearing the open trenching and jack-and-bore entry/exit pit areas of surface pavements, improvements and/or vegetation, excavating the open trenches and jacking/receiving pits, excavations for manholes,, installation and removal of temporary shoring systems, backfilling of the trenches and jack-and-bore entry/exit pits, and restoration of the surface pavement and other improvements. These items are discussed in the following sections.

7.1 SITE DEMOLITION AND PREPARATION

In the designated areas of the open-cuts, the site will likely be cleared of all surface and subsurface deleterious materials designated for removal, including existing pavements, curb and gutter, debris, shrubs, designated trees, and associated roots. All deleterious materials should be removed from the site and properly disposed of in accordance with regulatory requirements.

7.1.1 Abandonment of Existing Utilities

All utilities designated for removal should be completely removed from within planned pipeline alignments.

7.2 MATERIAL FOR FILL

All on-site soils with an organic content less than 3 percent by weight may be reused as general fill. All utility trenches and excavations should be backfilled according to the City of Morgan Hill standards and requirements. From a practical standpoint, the material near and below the anticipated groundwater table of 8 feet is anticipated to be over-optimum and may difficult to compact because of the over optimum moisture conditions. Additionally, oversize materials (e.g. cobbles) will likely be encountered. Therefore, the excavated soils may need to be processed and dried prior to reuse as engineered fill. In general, imported fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with no more than 1.5 percent larger than 2½ inches. Imported fill material should be predominantly granular with a Plasticity Index of 15 or less. To prevent significant caving during future trenching or excavations, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical and environmental reports.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

7.3 TEMPORARY CUTS

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 20 feet at the site may be classified as OSHA Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification. Cutting or sloping back the excavations is not recommended and likely not feasible due to the limited area along the new sewer alignment. Recommended soil parameters for temporary shoring are provided in the "Temporary Shoring" section of this report.

7.4 BELOW-GRADE EXCAVATIONS – OPEN TRENCHES AND MANHOLES

As discussed above, excavations with temporary slopes are likely not feasible due to the limited area along the new sewer alignment; therefore, temporary shoring may support the planned cuts up to 25 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor's judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor's responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor's scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

7.4.1 Temporary Shoring – Open Trenches and Manholes

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards.

Based on the site conditions encountered during our investigation, the cuts may be supported by braced excavations, slide rails, deep soil mixing, or potentially other methods, depending on the judgement of the shoring designer and contractor. As discussed above, due to the presence of very dense granular soils, gravels, and cobbles, we do not anticipate that the driving of sheet piles is feasible for this project. We do not recommend the use of trench boxes/shields or improvised shoring systems consisting of hydraulic speed shores, steel plates, trench boxes/shields or combinations thereof due to the presence of sand and gravel materials. Installation of soldier piles with auger assistance may be used if approved by the City Engineer.

Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 3: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	40 pcf
Restrained Wall – Uniform Earth Pressure	25H*
Passive Pressure – Starting at 2 feet below the bottom of the excavation	400 pcf up to 2,000 psf maximum uniform pressure**

* H equals the height of the excavation
** The passive pressures are assumed to act over twice the soldier pile diameter
The above pressures do not consider hydrostatic pressure due to ground water. The temporary excavations should be dewatered or the shoring designed for hydrostatic pressures.

As discussed above, we performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils; however, dense gravel and cobbles were encountered, which can create difficult conditions during soldier beam and sheet pile installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below groundwater) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. Voids created during extraction of sheet piles should be grouted during removal and should be anticipated and planned for by the contractor. Additionally, sheet pile shoring should be interlocked and continuous. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created, they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

7.4.2 Construction Dewatering Jack-and-Bore Pits

Groundwater levels are expected to be as high as 8 feet below existing grade (approximately 10 feet or more above the planned excavation bottoms for the jack-and-bore pits); therefore, temporary dewatering is anticipated to be necessary during construction for the jack-and-bore pits, and may be necessary in isolated areas for manhole excavations. Dewatering of the trenches along 5th street are anticipated to be above the design groundwater depth; however, wet and/or unstable trench bottoms may be encountered and should be anticipated and planned for by the contractor. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

Depending on the groundwater quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm or sanitary, or trucked to an off-site facility.

7.5 COMPACTION REQUIREMENTS

All backfill should be compacted in accordance with the City of Morgan Hill requirements or the recommendations contained in this section, whichever is more stringent. Pavement and aggregate base sections should be restored to their original thicknesses and grades or as required by the City of Morgan Hill. All fills should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the “Subgrade Stabilization Measures” section of this report.

Table 4: Compaction Requirements

Description	**Material Description**	**Minimum Relative[1] Compaction (percent)**	**Moisture[2] Content (percent)**
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Trench Backfill	Aggregate Base[3]	95	Optimum

Table 4 continues

Table 4: Compaction Requirements (continued)

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

7.6 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the City of Morgan Hill Standard Details U-1 through U-3. Utility lines in private improvement areas should be constructed with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded per the City of Morgan Hill Standard Details U-1 through U-3 attached to this report. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials for trenches wider than 18 inches provided they meet the requirements in the “Materials for Fill” section, and are moisture conditioned and compacted in accordance with the requirements in the “Compaction” section.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed sanitary sewer manholes may be supported directly on subgrade prepared in accordance with the recommendations provided in this report or on a layer of crushed rock (placed as a leveling course) provided the recommendations in the “Earthwork section and sections below are followed.

8.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot. Therefore, we have classified the site as Soil Classification D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the ATC Hazards by Location -line calculator (https://hazards.atcouncil.org/) based on the site coordinates presented below and the site classification. Based on the nature of the proposed improvements, we anticipate that the improvements will be designed in accordance with the Exception per ASCE Section 11.4.8. ***Recommended values in Table 5 may be used for design only if an exception will be taken in accordance with Section 11.4.8 of ASCE 7-16.*** The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 5: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	D
Site Latitude	37.1272624°
Site Longitude	-121.6496958°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.583g
1-second Period Mapped Spectral Acceleration[1], S_1	0.6g
Short-Period Site Coefficient – Fa	1
Long-Period Site Coefficient – Fv	1.7*
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.583g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.02*
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.055g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.68*

*Per Section 11.4.8 of ASCE 7-16

8.3 MANHOLE FOUNDATION

If the sewer manhole excavation bottom is stable and free of groundwater, the sewer manhole may be supported directly on subgrade prepared in accordance with the recommendations provided in this report or on 6 inches of ¾ inch clean crushed rock (placed as a leveling course) or Class 2 aggregate base (or approved equivalent), over native soil, prepared in accordance with the "Earthwork" section of this report. If groundwater is encountered and/or if the bottom of the excavation is unstable, the sewer manhole may be over-excavated an additional 12 to 18 inches and supported on clean crushed rock wrapped in stabilization fabric, Mirafi RS308i or approved equivalent. Prior to placement of crushed rock and/or aggregate base, the subgrade should be observed by a Cornerstone representative to confirm stable subgrade conditions prior to the installation crushed rock base.

8.3.1 Bearing pressures

Subgrade prepared in accordance with the "Earthwork" recommendations of this report is capable of supporting a maximum allowable bearing pressure of 3,000 psf for combined dead plus live loads. This pressure is based on a factor of safety of 2.0 applied to the ultimate bearing pressure for dead plus live loads. This pressure is a net value.

8.3.2 Manhole Foundation Support

Based on the assumed loading for the sewer manholes and the allowable bearing pressures presented above, we estimate that the total static settlement will be on the order of ¼ inch, with about ¼ inch of post-construction differential settlement across the footprint of the manholes or a horizontal distance of 30 feet along the pipelines.

8.4 MANHOLE FOUNDATION SUPPORT

Our boring EB-3 encountered groundwater at 16 feet below the existing ground surface (corresponding with Elevation 327 feet NAVD 88). CGS maps historic high groundwater as between 5 to 10 feet below existing grade. Groundwater levels for monitoring wells in the vicinity of the site provided on Geotracker website (2021), indicated groundwater depths of 8½ feet below the existing grades (corresponding with Elevation 334½ feet, NAVD 88). The anticipated depths of the sewer manholes are 8 to 11 feet below the existing grades and the depth of the jack and bore pits are 15 to 20 feet below existing grade. From a geotechnical standpoint, we would recommend a design groundwater depth of 8 feet below the ground surface for buoyancy (uplift) design.

SECTION 9: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of City of Morgan Hill specifically to support the design of the Fifth Street Sewer Main Replacement project 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 Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

City of Morgan Hill may have provided Cornerstone with plans, reports and other documents prepared by others. City of Morgan Hill understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone 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, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 10: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

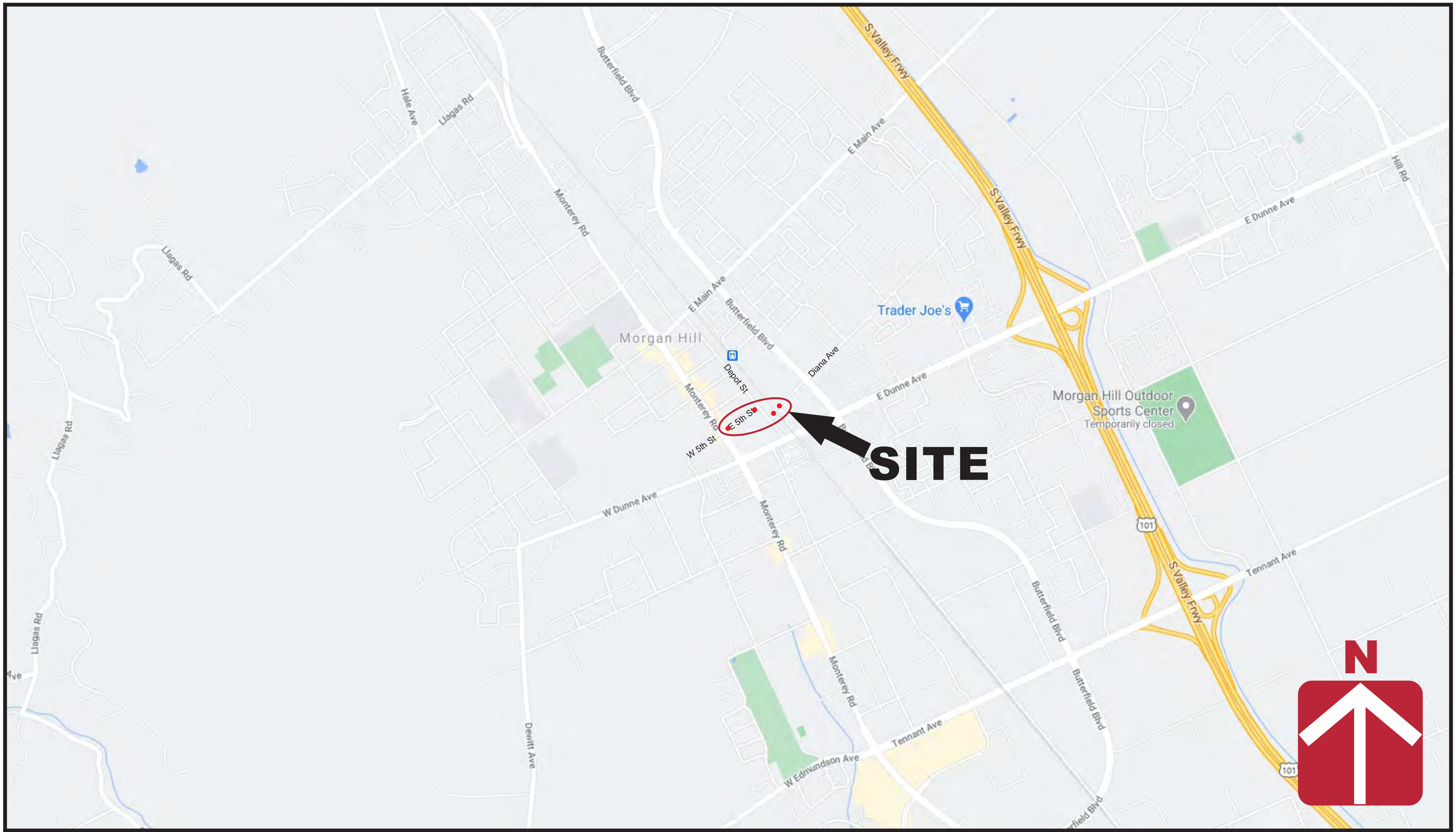
California Building Code, 201 9, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February 1998.

Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

Santa Clara County Department of Planning and Development Office, Geologic and Natural Hazards Zones – Spatial Data
https://www.sccgov.org/sites/dpd/PlansOrdinances/GeoHazards/Pages/GeoMaps.aspx

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3), USGS Open File Report 2015-3009.



CORNERSTONE EARTH GROUP

Vicinity Map

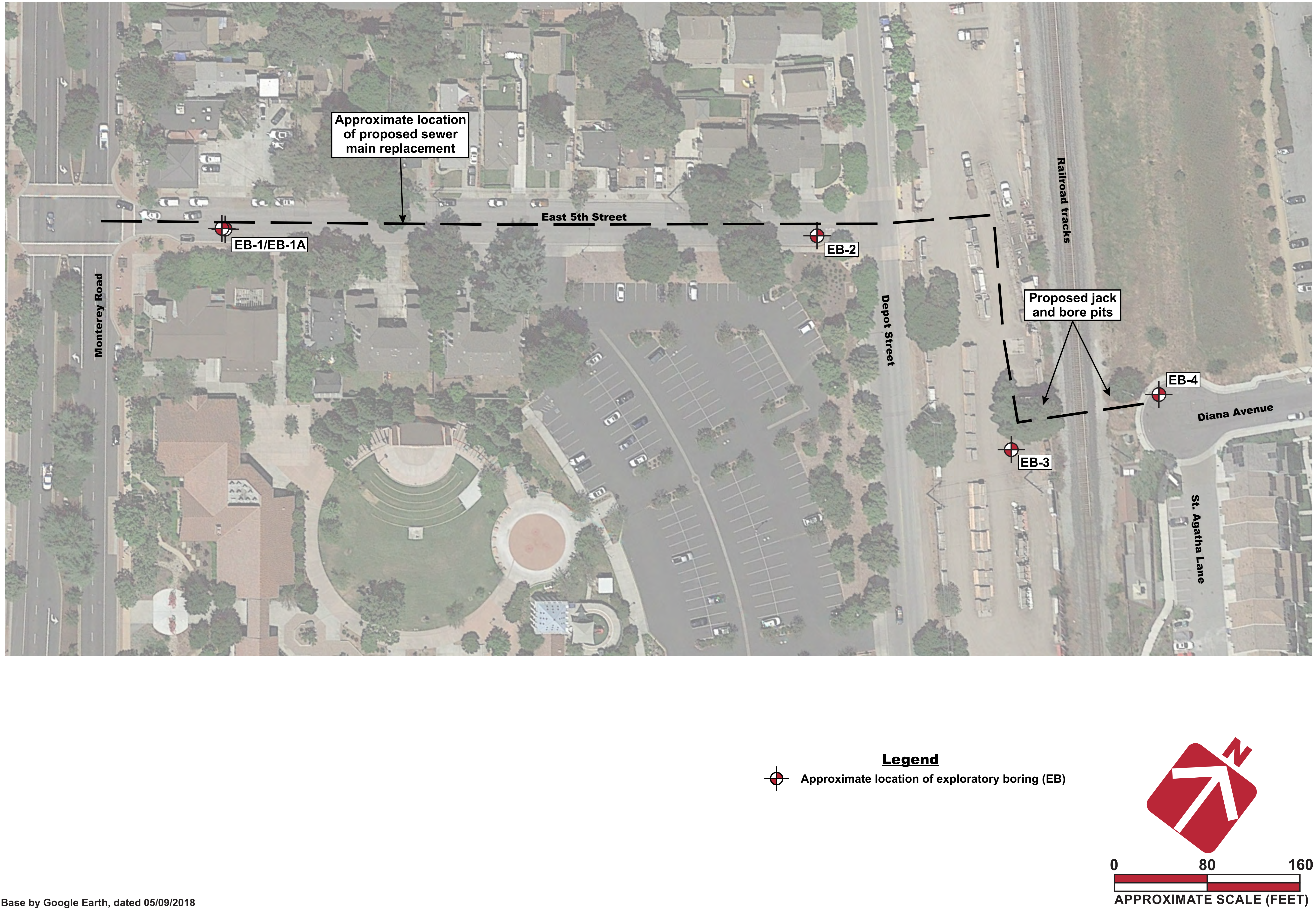
Fifth Street Sewer Main Replacement
Morgan Hill, CA

Project Number
1267-1-1

Figure Number
Figure 1

Date February 2021

Drawn By RRN



Base by Google Earth, dated 05/09/2018

Project Number
1267-1-1

Figure Number
Figure 2

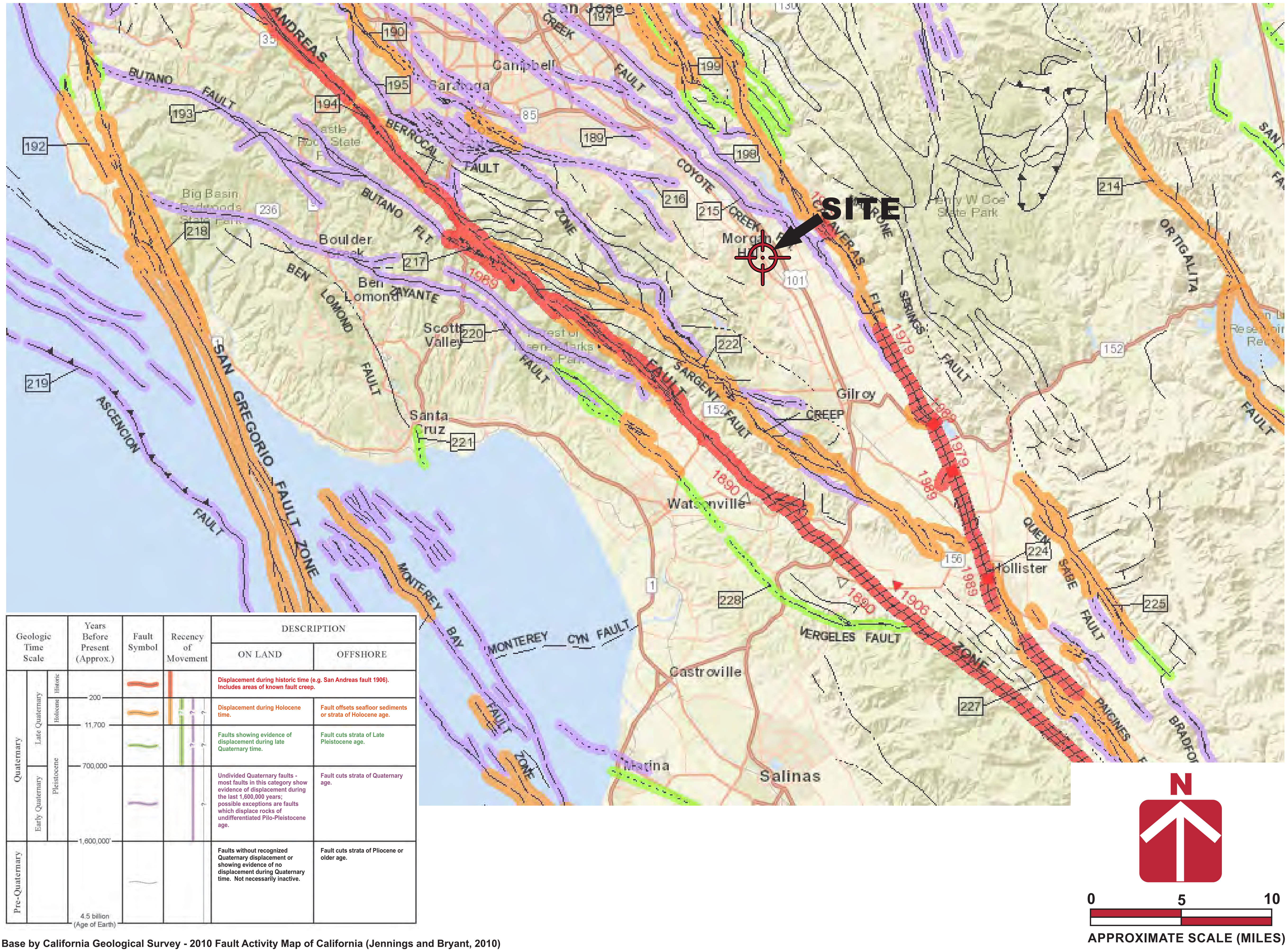
Date
February 2021

Drawn By
RRN

Site Plan

Fifth Street Sewer Main Replacement
Morgan Hill, CA

CORNERSTONE
EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
1267-1-1

Figure Number
Figure 3

Drawn By
RRN

Date
February 2021

Regional Fault Map

Fifth Street Sewer Main Replacement
Morgan Hill, CA

CORNERSTONE
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on February 9, 2021 to depths of 19 to 40 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, and other site features as references. Boring elevations were determined from the provided plan set. The elevations and locations of the borings 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. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring 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 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL	●
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL	○
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	○
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND	⁘
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND	⁘
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	▥
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	☰
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	≋
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	⁂

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	⁘	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	⁘	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	○	Gravelly Silt
⁘	Poorly-Graded Gravelly Sand	■	Asphalt
⁘	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

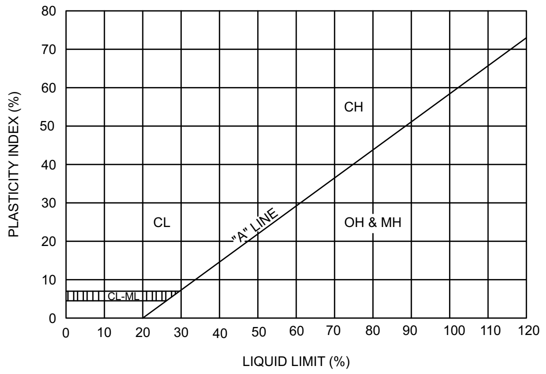
SAMPLER TYPES

☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core	✋	Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW		SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)		-	
RV	-	R-VALUE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE			
▼	-	WATER LEVEL			

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 5th Street SS Replacement

PROJECT NUMBER 1267-1-1

PROJECT LOCATION Morgan Hill, CA

DATE STARTED 2/9/21 **DATE COMPLETED** 2/9/21

GROUND ELEVATION 343 FT +/- **BORING DEPTH** 2.5 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.126675° **LONGITUDE** -121.650715°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY EA

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, 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
- ○ HAND PENETROMETER
- △ TORVANE
- ● UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
343.0	0	2 inches asphalt concrete over fabric and 3 inches asphalt concrete		GB				
340.5		**Silty Gravel with Sand (GM)** moist, reddish brown, fine to coarse subangular to subrounded gravel, fine to coarse sand, abundant cobbles Practical refusal of auger on possible cobble or boulder.		GB				
	5	Bottom of Boring at 2.5 feet.						
	10							
	15							
	20							
	25							
	30							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1A

PAGE 1 OF 1

PROJECT NAME 5th Street SS Replacement

PROJECT NUMBER 1267-1-1

PROJECT LOCATION Morgan Hill, CA

DATE STARTED 2/9/21 **DATE COMPLETED** 2/9/21

GROUND ELEVATION 343 FT +/- **BORING DEPTH** 19 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.126659° **LONGITUDE** -121.650741°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY EA

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
343.0	0	2 inches asphalt concrete over fabric and 3 inches asphalt concrete							
		Silty Gravel with Sand (GM) very dense, moist, reddish brown, fine to coarse subangular to subrounded gravel, fine to coarse sand, abundant cobbles		GB GB					
337.5	5	**Sandy Lean Clay (CL)** hard, moist, brown with light brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low plasticity	50 4"	MC-3B	111	11			>4.5 ○
334.5	10	**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles	50 5"	MC					
	15	becomes dense	48	SPT-5		10			
327.0		**Clayey Gravel with Sand (GC)** very dense, moist, brown with reddish brown mottles, fine to coarse subangular to subrounded gravel, fine to coarse sand, some cobbles	50 6"	MC-6	123	7			
324.0		Bottom of Boring at 19.0 feet.							
	20								
	25								
	30								

BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 5th Street SS Replacement
PROJECT NUMBER 1267-1-1
PROJECT LOCATION Morgan Hill, CA

DATE STARTED 2/9/21 **DATE COMPLETED** 2/9/21
GROUND ELEVATION 344.5 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.127458° **LONGITUDE** -121.649285°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
344.5	0	2 inches asphalt concrete over fabric and 3 inches asphalt concrete							
		Silty Sand with Gravel (SM) very dense, moist, reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel, trace cobbles		GB					
342.5		**Clayey Sand with Gravel (SC)** very dense, moist, reddish brown, fine to coarse sand, fine to coarse subangular to subrounded gravel, trace cobbles		GB					
341.5		**Sandy Lean Clay (CL)** hard, moist, brown with light brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low plasticity		GB					
	5		50 6"	MC-4A	110	12		69	>4.5 ○
336.5		**Clayey Sand with Gravel (SC)** very dense, moist, brown with gray mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles	64	SPT-5		9			
	10								
		decreasing clay content	50	SPT-6		8			
	15								
327.5		**Well Graded Sand with Silt and Gravel (SW-SM)** dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles	43	SPT-7		9		11	
324.5	20	Bottom of Boring at 20.0 feet.							
	25								
	30								

BORING NUMBER EB-3

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 5th Street SS Replacement

PROJECT NUMBER 1267-1-1

PROJECT LOCATION Morgan Hill, CA

DATE STARTED 2/9/21 **DATE COMPLETED** 2/9/21

GROUND ELEVATION 343 FT +/- **BORING DEPTH** 30 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.127311° **LONGITUDE** -121.648444°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

▽ **AT TIME OF DRILLING** 17 feet

▼ **AT END OF DRILLING** 16 feet

LOGGED BY EA

NOTES

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
343.0	0	**Poorly Graded Gravel with Sand (GP) [Fill]**							
		Clayey Sand with Gravel (SC) dense to very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles		GB					
	5		76	MC-2B	125	7			
	10		77	SPT					
	15		45	SPT-4		9			
	20		46	SPT					
	25		62	SPT					
315.5		**Lean Clay with Sand (CL)** very stiff, moist, reddish brown with gray mottles, fine to medium sand, moderate plasticity	78	MC					○
313.0	30	Bottom of Boring at 30.0 feet.							

BORING NUMBER EB-4

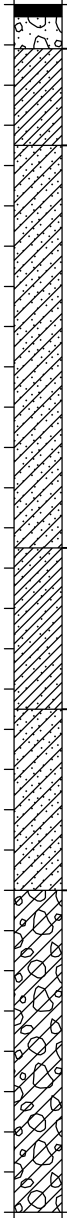
PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 5th Street SS Replacement
PROJECT NUMBER 1267-1-1
PROJECT LOCATION Morgan Hill, CA
DATE STARTED 2/9/21 **DATE COMPLETED** 2/9/21
GROUND ELEVATION 346.5 FT +/- **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.127622° **LONGITUDE** -121.648183°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, 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
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
346.5	0	4½ inches asphalt concrete over 8 inches aggregate base							
345.4		**Sandy Lean Clay (CL)** very stiff, moist, dark brown to brown, fine to medium sand, low plasticity		GB					
				GB					
343.0		**Clayey Sand with Gravel (SC)** medium dense, moist, brown with gray mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles							
	5		44	MC-3B	122	10			
			52	MC					
	10								
333.0		**Sandy Lean Clay (CL)** very stiff, moist, reddish brown with gray mottles, fine to medium sand, low plasticity Liquid Limit = 26, Plastic Limit = 13	37	MC-5B	119	15	13		○
	15								
329.0		**Clayey Sand with Gravel (SC)** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles	62	MC-6B	127	13			
	20								
324.5		**Clayey Gravel with Sand (GC)** very dense, moist, brown with reddish brown mottles, fine to coarse subangular to subrounded gravel, fine to coarse sand, some cobbles	50 6"	SPT					
	25								
			72	SPT-8		13			
316.5	30	Bottom of Boring at 30.0 feet.							

Undrained shear strength scale: 1.0 2.0 3.0 4.0

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

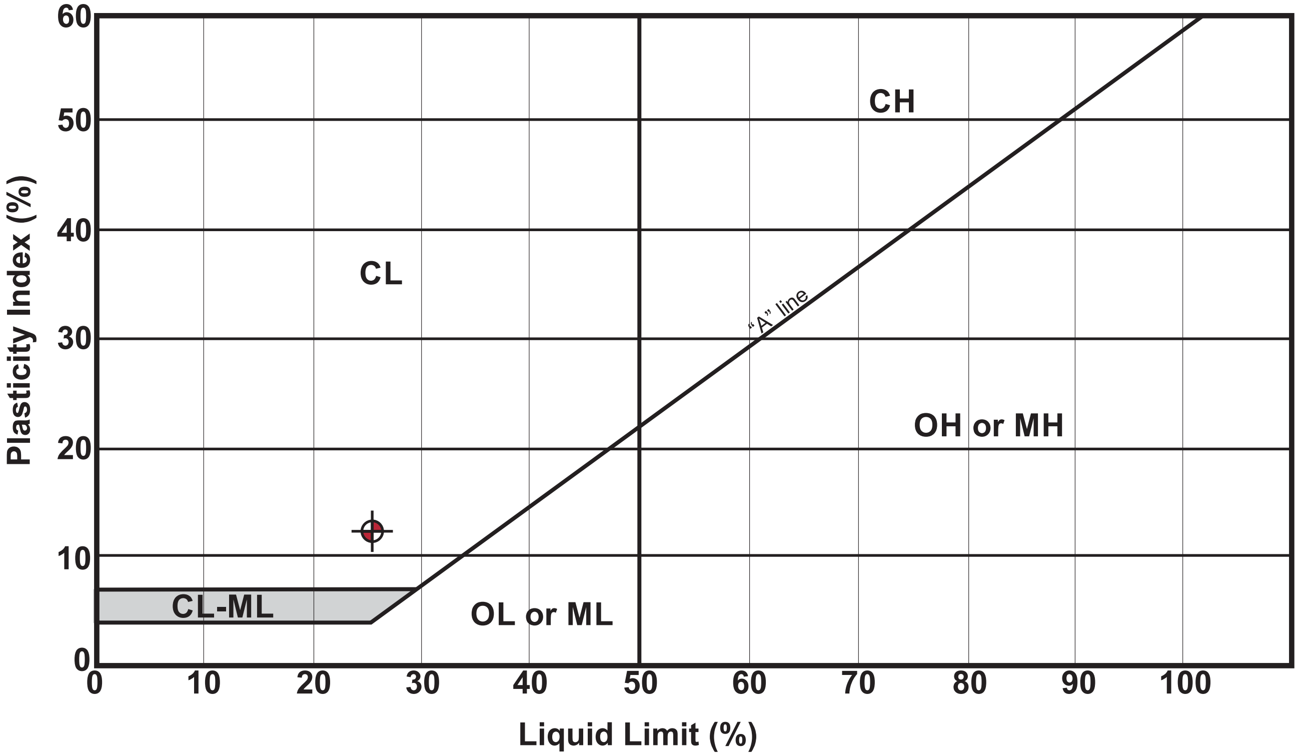
Moisture Content: The natural water content was determined (ASTM D2216) on thirteen samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one sample of the subsurface soils to aid in the classification of these soils. Results of this test are shown on the boring logs at the appropriate sample depth.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample 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 this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⌖	EB-4	14.0	15	26	13	13	—	Sandy Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Fifth Street Sewer Main Replacement
Morgan Hill, CA

Project Number 1267-1-1

Figure Number Figure B1

Date February 2021

Drawn By FLL

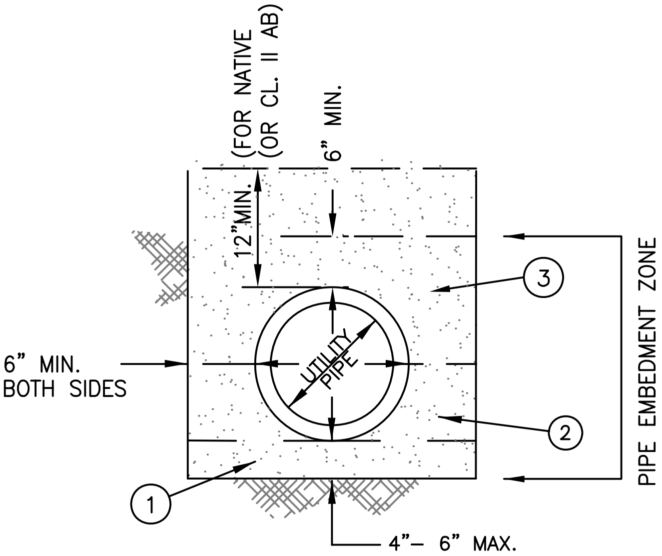
Corrosivity Tests Summary

CORNERSTONE
EARTH GROUP

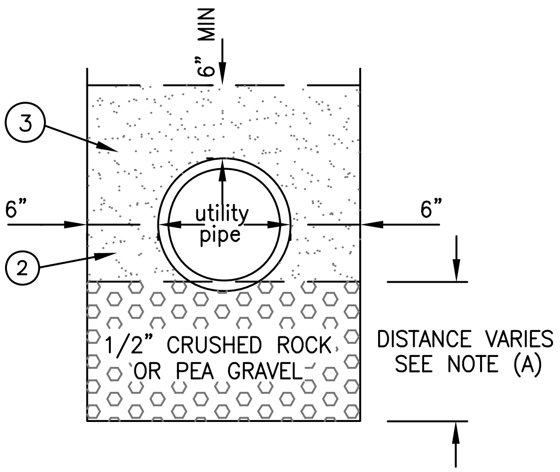
Job Number 1267-1-1
Job Name 5th Street Sanitary Sewer Replacement
Location Morgan Hill, CA
Date Tested 2/17/2021
Tested By FLL

Sample I.D.			Soil Visual Description	Moisture Content %	pH	Temp. at Testing C°	Resistivity (Ohm-cm) Corrected to 15.5 C°		Chloride Dry Wt. mg/kg	Sulfate Dry Wt. mg/kg
Boring	Sample No.	Depth, ft.					As Received	Saturated		
				ASTM D2216	ASTM G51		G57	ASTM G57	ASTM D4327	ASTM D4327
EB-1	4	8.5	Brown Clayey Sand (SC)	12.6	6.9	22.0	-	4,080	9	21
EB-2	5	8.5	Brown Clayey Sand (SC)	14.0	6.6	22.8	-	2,873	41	26
EB-4	7	23.5	Brown Clayey Sand (SC)	13.2	6.9	22.9	-	6,328	7	19

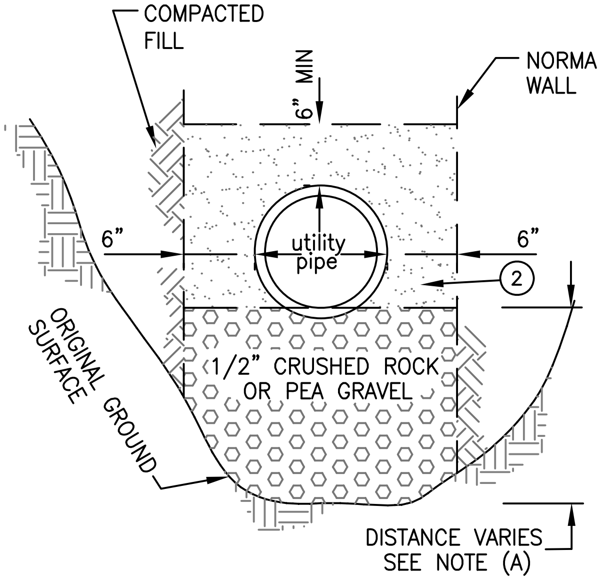
APPENDIX C: CITY OF MORGAN HILL STANDARD DETAILS



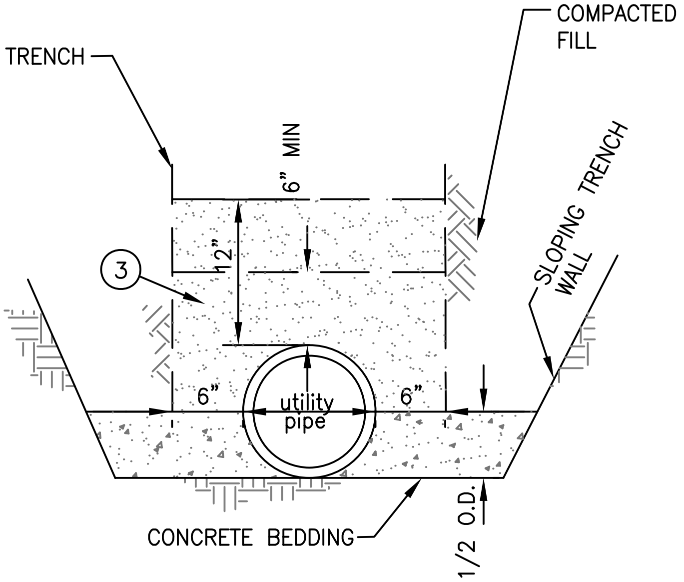
CASE 'A': NORMAL TRENCH
NTS



CASE 'B': WET / SPONGY GROUND
NTS



CASE 'C': FILLED GROUND
NTS



CASE 'D': BOTTOM OF TRENCH EXCEEDS NORMAL TRENCH WIDTH
NTS

NOTES:

(A) DISTANCE WILL VARY BASED UPON FIELD CONDITIONS, AND SOILS REPORT RECOMMENDATIONS.

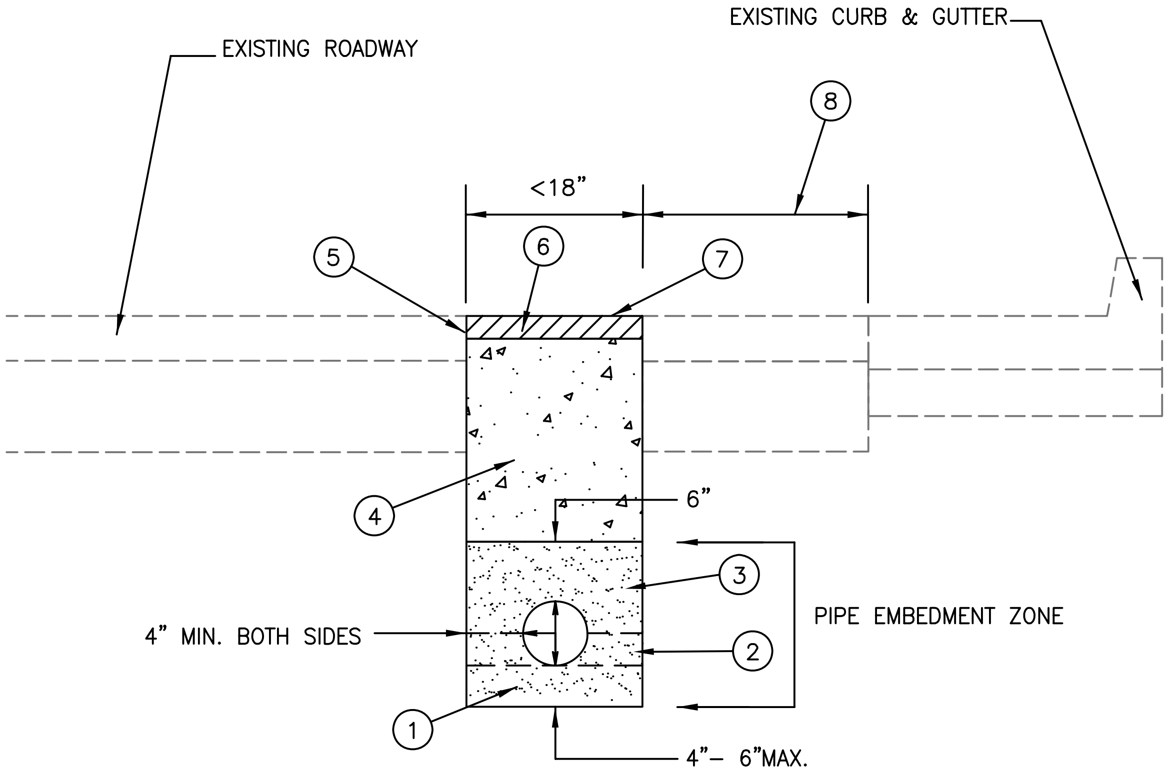
(B) PIPE EMBEDMENT SHALL CONFORM TO THE PRACTICE RECOMMENDED FOR CLASS III MATERIAL (SAND) IN ASTM D 2321 "UNDERGROUND INSTALLATION OF THERMOPLASTIC PIPE FOR SEWERS AND OTHER GRAVITY-FLOW APPLICATIONS".

(1.) SAND BEDDING, HAND PLACED AND COMPACTED TO MIN. 90% RELATIVE COMPACTION, 4" MIN. TO 6" MAX.

(2.) HAUNCHING; HAND PLACED AND COMPACTED TO MIN. 90% RELATIVE COMPACTION TO SPRING LINE OF PIPE.

(3.) INITIAL BACKFILL, INSTALL AND COMPACT TO A MINIMUM OF 6" ABOVE PIPE CROWN (12" MIN. FOR NATIVE).

City of Morgan Hill **Public Works Department**	TRENCH BEDDING	DRAWING NO.
Jim Ashcraft CITY ENGINEER 4/1/96 DATE REVISED		U-1



NOTES:

A. PIPE EMBEDMENT SHALL CONFORM TO THE PRACTICE RECOMMENDED FOR CLASS III MATERIAL (SAND) IN ASTM D 2321 "UNDERGROUND INSTALLATION OF THERMOPLASTIC PIPE FOR SEWERS AND OTHER GRAVITY-FLOW APPLICATIONS".

① SAND BEDDING, HAND PLACED AND COMPACTED TO 90% RELATIVE COMPACTION, 4" MIN. TO 6" MAX.

② HAUNCHING, HAND PLACED AND COMPACTED TO 90% RELATIVE COMPACTION TO SPRING LINE OF PIPE.

③ INITIAL BACKFILL, INSTALL AND COMPACT TO A MINIMUM OF 6" ABOVE PIPE CROWN.

④ 1.5 SACK CEMENT SLURRY BACK FILL. CEMENT SLURRY BACKFILL TO BE CURED PER MANUFACTURERS REQUIREMENTS PRIOR TO PAVING.

⑤ SAW CUT EXISTING PAVEMENT, ALL VERTICAL EDGES SHALL BE TACKED PRIOR TO PAVING.

⑥ 1 1/2" AC (1/2" TYPE B).

⑦ SURFACE SHALL BE FOG SEALED AFTER PAVING. EXISTING ROADWAY SURFACE SHALL BE REPLACED IN KIND (OIL & SCREENED, SLURRY SEAL, ETC.)

⑧ IF DISTANCE IS LESS THAN 3 FEET, PAVEMENT RESTORATION SHALL EXTEND TO LIP OF GUTTER.

City of Morgan Hill
Public Works Department

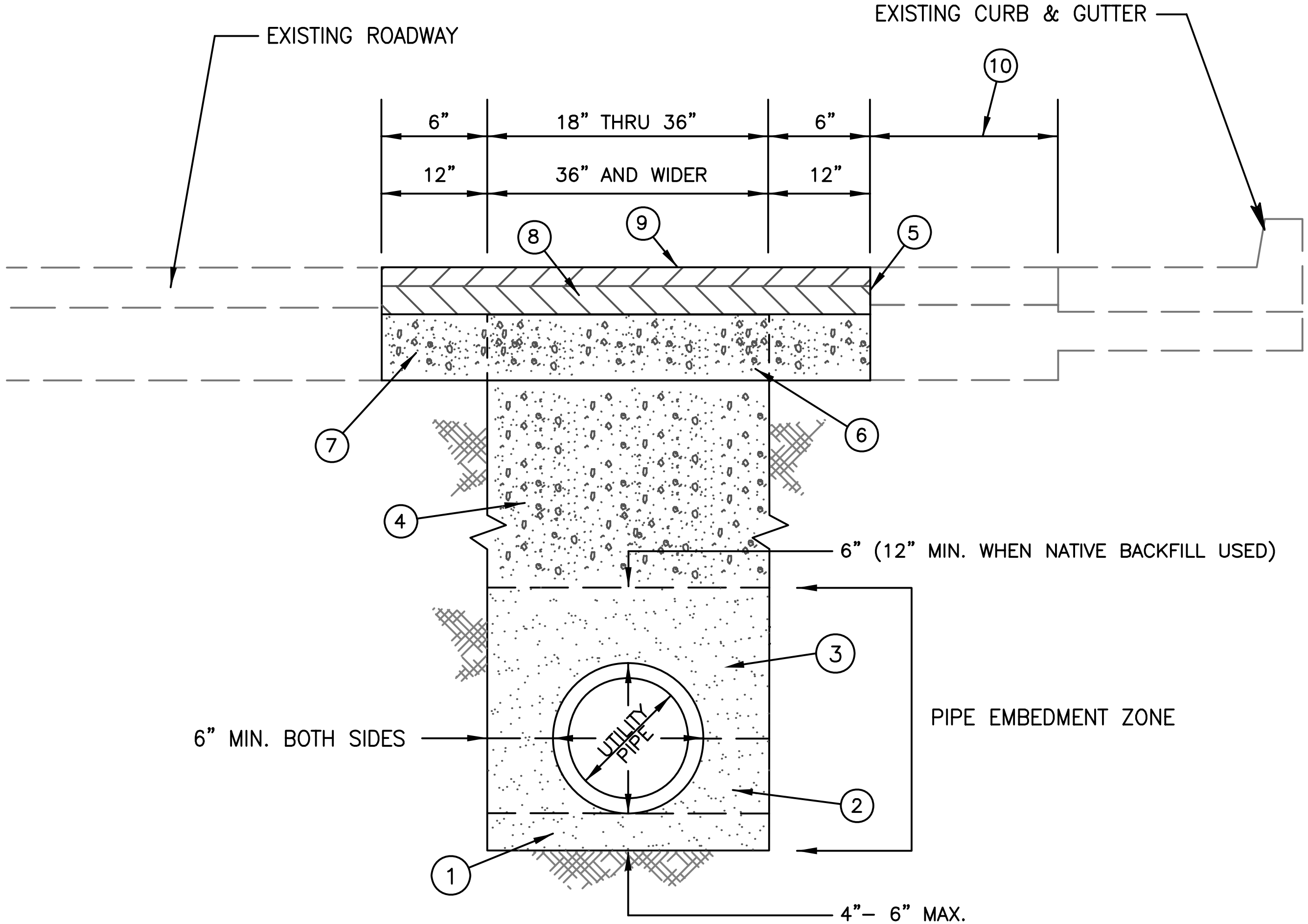
Jim Ashcroft
CITY ENGINEER

4/1/96
DATE

3/15/07
REVISED

TRENCH RESTORATION/BACKFILL
FOR
TRENCH WIDTHS LESS THAN 18"

DRAWING
NO.
U-2



NOTES:

A. PIPE EMBEDMENT SHALL CONFORM TO THE PRACTICE RECOMMENDED FOR CLASS III MATERIAL (SAND) IN ASTM D 2321 "UNDERGROUND INSTALLATION OF THERMOPLASTIC PIPE FOR SEWERS AND OTHER GRAVITY–FLOW APPLICATIONS".

① SAND BEDDING, HAND PLACED AND COMPACTED TO MIN. 90% RELATIVE COMPACTION, 4" MIN. TO 6" MAX.

② HAUNCHING; HAND PLACED AND COMPACTED SAND TO MIN. 90% RELATIVE COMPACTION TO SPRING LINE OF PIPE.

③ INITIAL SAND BACKFILL, INSTALL AND COMPACT TO A MINIMUM OF 6" ABOVE PIPE CROWN (12" MIN. FOR NATIVE).

④ 100% CLASS 2 AGGREGATE BASE ROCK BACKFILL COMPACTED IN LIFTS TO 95% RELATIVE COMPACTION. FLOODING OR JETTING SHALL ONLY BE ALLOWED UPON CITY ENGINEER APPROVAL. NATIVE BACKFILL MAY BE USED DURING THE CONSTRUCTION OF NEW STREETS ONLY AND SHALL BE USED ONLY UPON APPROVAL OF THE CITY ENGINEER AND UPON THE RECOMMENDATION OF A QUALIFIED SOILS ENGINEER/SOILS REPORT.

⑤ SAW CUT EXISTING PAVEMENT, ALL VERTICAL EDGES SHALL BE TACKED PRIOR TO PAVING.

⑥ 8" (MIN) CLASS 2 AGGREGATE BASE ROCK, COMPACTED TO 95% RELATIVE COMPACTION.

⑦ AT THE DISCRETION OF THE PROJECT INSPECTOR, THE EXISTING BASE ROCK MAY REMAIN FOR THIS TRENCH WIDTH PROVIDED THAT THE BASE ROCK IS COMPACTED AND IS NOT CONTAMINATED.

⑧ MATCH EXIST. AC SECTION OR 6" MIN. IN 2 LIFTS. BASE COURSE TO BE 3/4" TYPE B AC, AND SURFACE COURSE TO BE 1/2" TYPE B AC.

⑨ SURFACE SHALL BE FOG SEALED AFTER PAVING. EXISTING ROADWAY SURFACE SHALL BE REPLACED IN KIND.

⑩ IF DISTANCE IS LESS THAN 3 FEET, PAVEMENT RESTORATION SHALL EXTEND TO LIP OF GUTTER.

City of Morgan Hill
Public Works Department

Jim Ashcraft
CITY ENGINEER

4/1/96
DATE

3/15/07
REVISED

TRENCH RESTORATION/BACKFILL FOR TRENCH WIDTHS GREATER THAN 18"

DRAWING
NO.
U–3