August 29, 2017
Project 3700

Bellecci & Associates
2290 Diamond Blvd. Suite 100
Concord, CA 94520

Attention: Mr. Daniel Leary, P.E.

Subject: Geotechnical Investigation
Crystal Springs Complete the Gap Bike Trail
San Mateo County, California

Dear Mr. Leary:

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for proposed Complete the Gap (CTG) bike trail on Skyline Blvd. adjacent to Crystal Springs reservoir in San Mateo County, California. The site is shown on Figure 1, Vicinity Map and Figures 2A and 2B, Site Plan1.

2.0 PROJECT DESCRIPTION

The project will consist of constructing a paved bike trail along the western edge of Skyline Blvd. starting at the southern end of Lower Crystal Springs Dam and extending 800 feet south to connect to an existing bike trail segment. The construction of the bike trail will require widening of the existing roadway shoulder area in some locations by placing fill, and may necessitate that retaining walls or structures be constructed in a few areas. The location and height of the retaining structures will depend on the bike trail width and alignment alternative selected by the San Mateo County from the options presented by your firm. We understand that 6-foot wide and 8-foot wide trail sections are being considered, as well as alternative alignments that require trees to be either removed or preserved. The proposed preliminary design has the trail separated from the roadway by a concrete curb and gutter and a fence. A chain link fence is also proposed along the west edge of the trail.

Currently, the northern approximately 400 feet of the roadway and shoulder are being used to support the ongoing construction of a roadway bridge deck on the dam. As part of this project, the elevation of the Skyline Blvd. roadway and shoulder will be raised about 7 to 8 feet at the south abutment of the dam (and north end of the bike trail) and will taper to match existing grade at a point roughly 250 feet south of the abutment. The

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1 Site plan, stationing, and elevations mentioned in this report are based on Trail Improvement Plan, Sheet-3, Complete the Gap Trail Project, San Mateo County, California (30% Submittal), prepared by Bellecci & Associates, Inc., dated March 2017.
bike trail will be constructed to match these new roadway grades. A portion of the new fill was in place at the time of our fieldwork. We understand that this new fill and roadway section will be constructed prior to the construction of the CTG bike trail and that the new roadway will have minimal shoulder width and will not be wide enough to accommodate a bike trail. The replacement of a portion of the storm drain underlying the bike trail alignment is also currently in progress and has disrupted some of the fill previously placed by the bridge deck contractor. We understand that this disrupted fill will be restored after the completion of the storm drain project. Additional fill will also need to be placed as part of the bike trail project, to reestablish a wider shoulder to accommodate the bike trail.

3.0 PURPOSE AND SCOPE OF WORK

The purpose of our geotechnical investigation was to evaluate the subsurface conditions and provide geotechnical engineering recommendations for the design and construction of the bike trail. In accordance with our proposal dated August 23, 2016, we completed the following scope of work:

1. Reviewed available geotechnical and geologic information pertinent to the site, including previous geotechnical and geologic studies for the site and vicinity.

2. Performed a site reconnaissance to observe existing conditions within and adjacent to the study area.

3. Explored subsurface conditions by drilling and sampling nine exploratory test borings to depths between 10.0 and 31.3 feet below the ground surface within the approximately 800-foot long shoulder area.

4. Performed geotechnical laboratory testing to evaluate pertinent engineering and index properties of the earth materials.

5. Performed engineering analyses of the acquired data to identify and evaluate the most-appropriate retaining and/or stabilization measures to support the proposed bike trail. Developed geotechnical engineering recommendations for retaining wall design, pavement design, site grading and fill construction, and site drainage.

6. Prepared this report summarizing our findings, conclusions, and geotechnical design recommendations for the proposed bike trail.

As outlined in our proposal, we also completed supplemental tasks to assist in the preparation of design documents as follows:
1. Provided consultation and meeting attendance to assist design team in evaluating alternatives and developing the design.

2. Provided retaining wall design calculations and design sketches for soldier pile for incorporation into plan sheets prepared by your firm.

The above scope of work did not include the investigation or evaluation for the presence of hazardous materials on the site, or in the soil and groundwater beneath the site.

4.0 SITE CONDITIONS

4.1 General

The area proposed for the bike trail comprises the outer and western edge of the Skyline Blvd. roadway embankment and consists primarily of a soil and gravel shoulder of varying width (6 feet to 25 feet). At the roadway shoulder edge, the embankment slopes downhill to the west and has varying slope inclinations between about 1.3H:1V and 3.0H:1V (horizontal to vertical). Depending on the alignment alternative that is selected, the bike trail area may also encompass a small portion of the embankment upper slope and existing vegetation. The upper portion of the embankment slope has vegetation typically consisting of brush and ground cover. There are also mature trees in a few areas on the upper embankment slope, which could require trimming or removal, depending on the selected alignment.

The roadway appears to have been constructed by cut and fill with the inner northbound lane and a portion of the outer southbound lane generally supported on cut, and the remainder of the outer lane and shoulder supported on fill, except where the roadway and shoulder cross two east-west trending drainages which have been filled. Culverts were installed in these drainages to convey runoff beneath the road from areas upslope of the embankment to areas downslope of the embankment. The culverts are approximately at stations 169+20 and 173+25, and have concrete headwalls at the upstream ends.

Within the bike trail segment, the roadway and shoulder are at about a 6.5% grade. In general, the shoulder has a slight swale and outer berm roughly parallel to the roadway that directs roadway runoff along the shoulder to existing drain inlets at the roadway edge. These inlets are connected to a storm drain that is under the roadway shoulder and which consists of a 48-inch reinforced concrete pipe (RCP) upstream of 170+90, and a recently installed 48-inch HDPE plastic corrugated pipe downstream of 170+90 that replaced a deteriorated 36-inch corrugated metal pipe. The culvert and proposed bike trail alignments coincide. Our limited observations as of August 25, 2017 indicate that
the 48-inch HDPE culvert was installed in a 6-foot wide trench and that bedding and backfill placement were in progress. In several places lean concrete slurry bedding was placed to about 4 inches above the pipe and compacted soil backfill was placed above the pipe bedding. At this time, no information on inspections or testing results has been furnished. From our limited observations and conversations with the contractor, the pipe cover depth appears to be at least 27 inches below existing grade at the drain inlet at 170+90, and increases slightly going northward, except between about 168+50 and 169+75 where it is about 18 inches, according to the pipeline foreman. Beyond 169+75 the pipe cover increases with proximity to the bridge abutment. The 6-foot wide backfilled trench underlies a portion of the bike trail width, creates a non-uniform subgrade support condition, and will require limited over-excavation of the new and existing subgrade underlying the bike trail alignment.

The site features are shown on Figures 2A and 2B.

5.0 GEOLOGIC AND SUBSURFACE CONDITIONS

5.1 Geologic Conditions

The project site is located within the Coast Ranges Geomorphic province of California, a region characterized by northwest-trending mountains, intervening valleys, and northwest-trending faults. The site is located on the west-facing slope adjacent to the Crystal Springs Reservoir shoreline and is in a valley formed by the San Andreas fault, which underlies the reservoir. The site is mapped as underlain by the Franciscan Complex of Cretaceous to Jurassic age consisting of sheared rock described as containing predominately greywacke, siltstone, and shale, among other rocks. Based on our observation of test boring samples, a variable thickness of poorly lithified clayey sandstone was sporadically encountered above sheared rock.

5.2 Faulting and Seismicity

The project is located in the seismically active San Francisco Bay area, which is dominated by the active San Andreas fault and related active faults such as the San Gregorio-Seal Cove, Hayward, and Calaveras. The San Andreas fault is located about 1000 feet southwest of the project site. Consequently, the site is about 200 feet outside of and not within the State of California Earthquake Fault Zone designated for the San Andreas fault. The San Gregorio-Seal Cove is located approximately 7.6 miles southwest of the project site. The Hayward fault is located 18 miles northeast of the site. The Calaveras fault is located 27 miles northeast of the project site.

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Numerous large earthquakes have occurred in this region in the past and some have previously caused strong ground shaking at the site. The largest and most notable was the San Francisco earthquake of April 18, 1906 (Richter magnitude 7.9) with epicenter about 12 miles to the north. The most recent large earthquakes to occur in the region include the October 17, 1989 Loma Prieta earthquake (Richter magnitude 6.9), the 1984 Morgan Hill earthquake (Richter magnitude 6.2), the 1979 Coyote Lake earthquake (Richter magnitude 5.9), and the 1957 Daly City earthquake (Richter magnitude 5.3).

5.3 Subsurface Investigation

Our subsurface investigation consisted of drilling exploratory test borings, performing field penetration tests, retrieving drive samples from the borings, and completing laboratory testing of selected samples. Nine test borings, FG-1 through FG-9, were drilled in the roadway shoulder as close as practical to the embankment slope crest on June 29, 2017 to depths between about 10 feet and 31 feet below the ground surface. The locations of the borings are shown on Figure 2. A description of our subsurface methodology and the test boring logs are included in Appendix A.

The samples obtained from the exploratory borings were examined in the laboratory to confirm field classifications and to select representative samples for testing. Laboratory tests determined moisture content, dry density, unconfined compressive strength, consolidated-undrained direct shear strength, and corrosivity of soil and rock materials. Laboratory test results are summarized on the test boring logs and data sheets in Appendix A.

5.4 Subsurface Conditions

The exploratory test borings encountered a varying thickness of artificial fill ranging between about 2 feet and 21 feet thick. The fill was underlain by colluvial and residual soil in some instances, and by bedrock in other instances. Accounting for the distance of the boring from the embankment edge, it appears that that with two exceptions, the embankment edge is generally underlain by 5 feet to 10 feet of fill, which is then underlain by weathered bedrock. The exceptions are where the test borings were drilled at the two drainage crossings and a deeper fill profile was encountered. Boring FG-4 was drilled near the drainage (culvert) crossing at 169+20 and boring FG-9 was drilled near the drainage (culvert) crossing at 173+25. Boring FG-4 encountered approximately 21.5 feet of fill, which was underlain by 2 feet of colluvial soil followed by weathered bedrock. Boring FG-9 encountered approximately 8.5 feet of fill underlain by 13 feet of colluvial soil followed by weathered bedrock.
The fill materials encountered in the borings typically consisted of medium dense silty fine grained sand with gravel. Colluvial and residual soil encountered in the borings consisted of medium dense clayey sand and sandy silt with gravel. Weathered bedrock encountered in the borings typically consisted of very severely to severely weathered clayey sandstone/conglomerate and very severely to severely weathered sheared rock, previously mapped as belonging to the Franciscan Complex.

The clayey sandstone/conglomerate was encountered in most of borings except FG-7 and FG-9. The clayey sandstone was typically poorly lithified, fine grained, and with severely weathered gravels, except for a zone at 15 feet in FG-5, where slightly weathered, very hard sandstone was encountered.

The sheared rock was encountered in most of the borings except FG-1, FG-2, and FG-5, which were drilled to relatively shallow depths between 10 feet and 15 feet. The sheared rock typically has indistinct lithology and structure and in many instances we identified it as possibly shale. The sheared rock was encountered below the clayey sandstone in FG-3, FG-4, FG-6, and FG-8.

Groundwater was encountered in only FG-7 at a depth of 18 feet during drilling.

It should be noted that we interpret the poorly lithified clayey sandstone/conglomerate to possibly be a small isolated remnant of the Santa Clara formation, which has other nearby remnants, and not Franciscan greywacke that is usually well lithified within the Franciscan Complex.

Our interpretations of subsurface conditions are shown on Figures 3 through 8, Subsurface Sections.

5.5 Corrosion Potential Test Results

Tests to assess corrosion potential of the on-site soil were performed on samples from the test borings. The results are presented in Appendix A and summarized in the table below.

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Material Type</th>
<th>Minimum Resistivity (ohm-cm)</th>
<th>pH</th>
<th>Sulfate (ppm)</th>
<th>Chloride (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FG3 @ 4.0’-6.5’</td>
<td>Silty Sand (SM/SC) Fill</td>
<td>4,020</td>
<td>6.81</td>
<td>8.2</td>
<td>5.6</td>
</tr>
<tr>
<td>FG3 @ 10.5’-11.0’</td>
<td>Clayey Sandstone</td>
<td>3,480</td>
<td>6.51</td>
<td>6.4</td>
<td>4.9</td>
</tr>
<tr>
<td>FG8 @ 20.5’-21.0’</td>
<td>Sheared Rock</td>
<td>2,950</td>
<td>5.58</td>
<td>2.9</td>
<td>4.8</td>
</tr>
</tbody>
</table>
Commonly used corrosion guidelines\textsuperscript{3} for buried metals are presented as follows:

<table>
<thead>
<tr>
<th>Resistivity (ohm-cm)</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 500</td>
<td>Very Corrosive</td>
</tr>
<tr>
<td>500 – 1,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>1,000 - 5,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>5,000 - 10,000</td>
<td>Slightly Corrosive</td>
</tr>
<tr>
<td>Over 10,000</td>
<td>Non-corrosive</td>
</tr>
</tbody>
</table>

On the basis of the resistivity and pH test results, the samples can be characterized as moderately corrosive to metals.

Soil with high sulfate concentrations can cause degradation of concrete. Based on guidelines published by the Portland Cement Association\textsuperscript{4} and the American Concrete Institute (ACI)\textsuperscript{5}, there is negligible attack on concrete by soil containing less than 1,000 mg/kg (ppm) of sulfate and moderate attack (Class 1) for soil containing between 1,000 ppm-2,000 ppm of sulfate. Based on the sulfate test results, the samples can be characterized as not corrosive to concrete.

Based on the guidelines issued by Caltrans\textsuperscript{6}, chloride concentrations in soil of over 500 ppm are considered corrosive to exposed metal and reinforcing steel in concrete. If only considering chlorides, the samples would be characterized as non-corrosive; however, as previously mentioned, the samples are considered moderately corrosive to exposed metals on the basis of minimum resistivity and pH. These samples can also be characterized as exposure class C1 by ACI guidelines.

### 6.0 CONCLUSIONS AND ANALYSES

#### 6.1 General

Based on this investigation, we conclude that, from a geotechnical engineering standpoint, the proposed CTG bike trail can be constructed as planned, provided that the discussions herein are considered, and the recommendations presented in this report are incorporated into the project plans and specifications.

\textsuperscript{3} Corrosion Manual, Pacific Gas and Electric Co., 1970
\textsuperscript{4} Portland Cement Association, Design and Control of Concrete Mixtures, 1988.
\textsuperscript{5} Building Code Requirements for Structural Concrete and Commentary (ACI 318-08), Table 4.2.1, 2008
\textsuperscript{6} California Department of Transportation (Caltrans), Corrosion Guidelines, 2003.
6.2 Retaining Wall Type

We conclude that the most appropriate and cost-effective retaining wall system for the project is a steel soldier pile and timber lagging wall. Other retaining systems such as a geogrid-reinforced segmental block wall and cast-in-place concrete walls were considered but not selected because of issues that included the limited equipment access, steep slope, inadequate shallow foundation support, construction impact on existing vegetation, and cost.

6.3 Fill and Subgrade Reinforcement

We conclude that the bike trail will be underlain by variable subgrade conditions that include isolated areas of loose existing fill, differential thickness of new fill over existing fill, recent storm drain trench backfill underlying only a portion of the trail, and proximity of the trail to the outer fill slope. Consequently, we conclude that over-excavation and geogrid reinforcement of the trail subgrade are required.

6.4 Seismic Hazards

The primary seismic hazard affecting the site is strong seismic ground shaking during a large earthquake, most notably on the nearby San Andreas fault. Based on probabilistic methods, a peak horizontal ground acceleration of 1.03g (103% gravity) was estimated to have a 2% probability of being exceeded in 50 years. We conclude that the project can be designed to accommodate strong ground shaking by using the current California Building Code.

Liquefaction, a secondary seismic hazard, is estimated to have an extremely low potential to occur at the project site because groundwater is within the weathered rock zone and because the cohesionless fill and colluvial soils are unsaturated and have a low potential to become saturated.

The potential for seismically-induced compaction of soil materials and resulting ground settlement at the site is judged to be relatively low because the site is generally underlain by a limited thickness of medium dense sandy soil and bedrock.

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8 Soil liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences a temporary reduction of strength during strong cyclic loading such as that produced by earthquake shaking. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly-graded, fine-grained sand and cohesionless silt.
Based on published maps, the potential for earthquake-induced instability to occur on the slopes adjacent to, or affecting, the proposed bike trail is estimated to be low to moderate because of the steep slope inclination and the intense level of ground shaking expected during a large earthquake on the nearby San Andreas fault. Slope stability analyses were performed for an area near 169+00 where the embankment slope is steepest, the embankment fill is deepest, and the shoulder is narrow.

The results of the analyses confirm that the slope may experience shallow surface failures (FS=0.91) during intense ground shaking; however, these analyses also indicate that the slope has an adequate degree of stability against deeper failures (FS=1.11). In addition, the analyses indicate that the slope with the proposed bike trail retaining wall has an adequate degree of stability (FS=1.21) against deeper failures. While the potential for shallow surface failures may exist, we conclude that this hazard is primarily in the areas of deepest fill and steepest slopes, namely at the deeper drainage areas such as 169+20 and 173+25. With proper foundation design for the proposed retaining walls, we conclude that the bike trail can be isolated from this hazard.

The potential for ground failure from fault rupture on the project site is estimated to be relatively low because there are no known traces of the San Andreas or other active faults that cross the project site.

7.0 RECOMMENDATIONS

7.1 Site Preparation and Grading

7.1.1 General - Prior to any clearing, minor grading, or drilling, the existing underground utilities and underground improvements should be located (by calling Dig Alert-Dial 811) and checked for any conflicts with proposed excavation activities. Buried high voltage lines and storm drains are known to be among the buried improvements within the bike trail alignment.

The bike trail alignment should be stripped of vegetation and organic material, and debris and the resulting material removed from the site. Where loose soil, soft soil, deleterious materials, or voids are encountered or result from tree removal, they should be excavated to expose firm soil prior to placement of engineered fill. Engineered fill should be placed and graded to conform to adjacent slopes. We estimate that the site excavation can be accomplished using conventional excavation equipment.

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The area underlying the bike trail and any fill placed for the bike trail should be over-excavated to a depth of 18 inches below final grade, or below existing grade, whichever is lower. The lateral limits of over-excavation should extend 2 feet beyond the bike trail pavement edge to any planned fill slope edges. The base of the over-excavated areas shall be in firm material and approved by the project geotechnical engineer. It is possible that over-excavation could conflict with the existing storm drain in some areas and may require a shallower depth of over-excavation. This will be determined from conditions exposed at the time of excavation.

The base of the over-excavated areas should be scarified to a depth of 6 inches, moisture conditioned to about 2 percent over optimum moisture content, and compacted to 95 percent of the maximum dry density as determined by the ASTM test method D1557. Over-excavated areas should be backfilled to design grade using non-expansive engineered fill with geogrid reinforcement.

### 7.1.2 Engineered Fill

- On-site excavated material derived from the site appears to be generally suitable for use as engineered fill. Site excavated material and imported non-expansive materials which are used as engineered fill should not contain debris, rocks and clods larger than 4 inches in greatest dimension, or more than 2 percent by dry weight of organic material. Additionally, the material should have no more than 30 percent material passing the No. 200 sieve, have a plasticity index of no more than 12, and have a minimum R-value of 40.

Material for engineered fill should be moisture conditioned, or allowed to dry, to about 2 percent above optimum moisture content, spread in horizontal lifts not exceeding 8 inches in loose thickness, and compacted with an approved mechanical compactor to at least 95 percent relative compaction as determined by the ASTM test method D1557. Fill shall not be placed on existing slopes steeper than 2H:1V. Where fill is placed on existing slopes with inclinations of 2H:1V or flatter, a minimum 3-foot deep by 8-foot wide keyway shall be excavated to support the new fill. Minimum 18-inch wide benches shall be excavated into the existing embankment as fill is being constructed to achieve finish grade. Permanent fill slopes should be constructed no steeper than 2H:1V.

Prior to fill placement, the contractor should submit samples of proposed fill materials and the type of compaction equipment for approval by the engineer. The choice of lightweight compaction equipment is important where compaction will occur over the storm drain.

### 7.1.3 Geogrid Reinforcement

- Geogrid reinforcement should be placed at the base of the over-excavation for the bike trail and at vertical intervals not exceeding 12 inches such that at least two layers of geogrid underlie the bike trail. The geogrid should extend...
the entire width of the over-excavation, a minimum of 10 feet, and should extend to the face of fill slopes. The geogrid reinforcement should have a biaxial minimum long term design strength\(^{10}\) of 850 pounds per foot.

### 7.2 Soldier Pile and Lagging Retaining Wall

Soldier piles should consist of wide flange steel beams placed in drilled holes with concrete backfill. Design details for the soldier pile wall are presented in Appendix C of this report and are based on the following criteria:

- **Minimum Pile Length:** The minimum pile length should be determined by using an overturning factor of safety (FS) of 1.5 for static loads plus traffic surcharge and a factor of safety of 1.3 for static loads plus seismic surcharge. In addition, the pile length should also conform to embedment requirements in California Building Code section 1807.3.2.1 (pole formula).

- **Minimum Diameter of Drilled Hole:** 2 feet.

- **Maximum Spacing:** 8 feet, center to center

- **Lateral Earth Pressure:** 40 pounds per cubic foot (pcf), equivalent fluid pressure applied to the exposed wall height plus 4 feet below the exposed wall height (4-foot thick zone of loose soil potentially prone to creep or seismically-induced shallow failure).

- **Traffic Surcharge Pressure:** 100 pounds per square foot (psf) applied as a uniform lateral pressure to a depth of 5 feet below the top of the retaining wall.

- **Seismic Surcharge Pressure:** 160 psf applied as a uniform lateral pressure to the exposed wall height plus 4 feet below the exposed wall height. Seismic and traffic loads are not concurrent.

- **Minimum Timber Lagging Requirements:** 6X6 No. 1 Douglas fir pressure-treated with AZCA for long-term design life. All installed lagging shall have treated ends. Lagging should be installed with 1/2-inch gaps using durable (non-degradable) spacers. Lagging should extend 24 inches below adjacent down hill final grade.

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• Passive Resistance: 300 pcf equivalent fluid pressure applied over 3 pile diameters, starting at 4 feet below the wall or retained material height.

• Corrosion Requirements: Additional sacrificial steel thickness of 0.11 inch to be added to flanges when determining minimum pile section.

Retaining wall areas should be excavated to create a bench at the approximate bottom of wall (bottom of lagging) elevation such that the excavation extends a minimum of 18 inches behind the soldier beams. Installation of the piles should follow Caltrans standard specifications, latest edition. Pile shafts should be free of standing water and cleared of all loose debris prior to pouring of concrete. It is anticipated that groundwater may collect in some pile shafts. The water should be pumped out or the concrete should be placed by the tremie method with the concrete displacing the water from the bottom up. If casing is required to maintain excavation stability, the casings shall be removed during placement of the concrete so that the concrete will cure in contact with native soil. Uncased shafts that encounter groundwater should be poured the same day that they are drilled. All drilled shafts should be inspected and approved by the project geotechnical engineer prior to the placement of steel piles.

Resistant zones of hard rock may be encountered during drilling within the weathered bedrock at the site. The contractor should be prepared to drill in hard rock and occasionally core through these resistant zones.

7.3 Wall Backfill and Drainage

Retaining wall backfill should consist of engineered fill as described above. A drain should be installed behind the retaining wall and should consist of a minimum 12-inch wide zone of Caltrans Class 2 permeable material or clean 1/2 to ¾-inch drain rock that is completely enveloped by geotextile filter fabric such as Mirafi 140N, or approved equivalent. The filter fabric is needed to prevent migration of fine backfill material through the 1/2 inch gaps that allow drainage between wood lagging. The drain should extend the entire wall height except for the upper foot, which should consist of engineered fill separated from the lagging by additional filter fabric.

The drainage material should be compacted using vibratory compactors to 90 percent relative compaction, or to the satisfaction of the engineer. Where the lagging will be buried, a 4-inch diameter schedule 40, perforated, PVC pipe should be placed at the low point in the backfill. The perforated drain pipe should be connected to a non-perforated discharge pipe, and should drain by gravity to an appropriate area.
7.4 Construction Considerations

Soldier pile layout should avoid conflicts with underground improvements including the culverts at 169+20 and 173+25 and the existing storm drain underlying the road shoulder.

Over-excavation for the bike trail may encounter the existing storm drain. The depth to the storm drain should be determined by potholing or other methods prior to excavation so that over-excavation recommendations can be modified if necessary.

The soldier piles will be installed on a steep slope with difficult access. The equipment to be used should be intended for limited access work. While the construction means and methods will be left to the contractor to select, the County should be clear in the plans and specifications regarding the amount and limits of disturbance and excavation it is willing to tolerate in order to construct the bike trail. It may be possible to work around some the existing mature trees, provided that their branches can be trimmed to accommodate the drill rig mast.

7.5 Erosion Control

Exposed soil areas where the ground surface has been disturbed, or vegetation has been disturbed or removed should be seeded using a native seed mix approved by the County and fertilized. As much as practical, an erosion control mat and/or erosion resistant materials approved by the engineer should be placed on these areas after seeding is completed.

7.6 Pavement Design

Existing shoulder materials consisting of silty sand with gravel can be reused as engineered fill; however, this material only composes a portion of the fill material required. Because import fill will be required and the source of the import fill has not been identified, the composition of the final bike trail pavement subgrade is yet to be determined. Consequently, we recommend that import fill have a minimum R-value of 40 corresponding to Caltrans requirements for Class 3 aggregate subbase. The calculation of the minimum recommended flexible pavement section using the Caltrans Highway Design Method and a traffic index of 5.0 yields a minimum flexible pavement section of 3.0 inches of asphalt concrete (AC) over 4.0 inches of Caltrans Class 2 aggregate base (CL2AB). Recent County standards\textsuperscript{11} for bike trails require a minimum pavement section of 3 inches of AC over 4 inches to 6 inches of CL2AB aggregate base.

\textsuperscript{11} Alta Transportation Consulting, 2000, San Mateo County Comprehensive Bicycle Route Plan, Table 9-Class I Bicycle Path Specifications, prepared for City/County Association of Governments.
Therefore, we recommend that the CTG bike trail pavement section consist of 3 inches of AC over 6 inches of CL2AB.

The subgrade underlying pavement areas should be prepared according to the recommendations provided in the Site Grading section. Aggregate base materials should be compacted to a minimum of 95 percent of the maximum dry density within two percent of the optimum moisture content as determined by ASTM D1557.

7.7 Plan Review and Construction Observation

We should review the project plans and specifications to verify that the intent of our recommendations is incorporated in these documents.

The project contractors should review and reference this report prior to construction. Any questions or discrepancies in this report or between this report and the project plans should be brought to the attention of our office prior to beginning construction activities related to the item in question.

If conditions different from those described in this report are encountered during construction, our office should be notified in a timely manner so that the conditions can be evaluated and report recommendations can be modified, if necessary, to address the change in conditions.

During construction, our office should observe during over-excavation, soldier pile layout and installation, and placement and compaction of engineered fill. Our office should be notified at least 48 hours in advance of construction activities requiring inspections.

8.0 LIMITATIONS

In performing our engineering services on this project, we have employed generally accepted principles and practices of the geotechnical engineering profession in the San Francisco Bay area at this time. This warranty is in lieu of all other warranties, either expressed or implied.

The conclusions and recommendations contained in this report are based on site observations and limited background and subsurface information. It is possible that changes in site conditions, subsurface conditions different from those encountered in the test borings, or additional information that could come to light in the future could alter the conclusions and recommendations in this report.
Please call if you have any questions regarding this report.

Sincerely,

[Signature]

Raymond L. Fisher, P.E., G.E.
Geotechnical Engineer #2188

Attachments: Figure 1, Vicinity Map
Figures 2A and 2B, Site Plan
Figures 3 through 8, Subsurface Sections
Appendix A - Subsurface Investigation
Appendix B - Slope Stability Analyses
Appendix C - Retaining Wall Design Details

Distribution: Addressee (digital)
Key:
- FG1: Test Boring Location
- SD: Existing 48" Storm Drain
- PG&E HV: Existing Underground High Voltage Line

Notes:
1. The base used for this site plan is an unmarked, undated topographic survey prepared by Bellecci & Associates.
2. Test boring locations are approximate.
3. Bike trail and retaining wall locations are approximate and subject to change.
4. Underground utility and underground improvement locations are approximate and not all utilities and improvements are shown. All underground utilities and improvements should be identified and accurately located prior to final design and construction. This includes but is not limited to calling 811, Dig Alert/USA North.

SCALE 1"=30'

Fisher Geotechnical
Civil and Geotechnical Engineering

COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700
DATE AUGUST 2017
FIGURE 2A
Key:
- FG1: Test Boring Location
- SD: Existing 48" Storm Drain
- PG&E HV: Existing Underground High Voltage Line

Notes:
1. The base used for this site plan is an untitled, undated topographic survey prepared by Bellecci & Associates.
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SCALE 1"=30'

SITE PLAN

Fisher Geotechnical
Civil and Geotechnical Engineering

COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700
DATE AUGUST 2017
FIGURE 2B
Key:

FG1  Test Boring  
EG   Existing Grade  
FG   Final Grade

Notes:
2. All subsurface features or utilities not shown.
3. Preliminary trail geometry shown.

SUBSURFACE SECTION 167+00
SCALE 1"=20'
See Figure 3 for Key and Notes.

SUBSURFACE SECTION 167+75
SCALE 1"=20'

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COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700
DATE AUGUST 2017
FIGURE 4
SUBSURFACE SECTIONS 169+25 AND 169+50
SCALE 1"=20'

See Figure 3 for Key and Notes.

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COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700 DATE AUGUST 2017 FIGURE 6
SUBSURFACE SECTIONS 171+00 AND 172+50
SCALE 1"=20'

See Figure 3 for Key and Notes.
See Figure 3 for Key and Notes.

SUBSURFACE SECTIONS 173+00 AND 173+25
SCALE 1"=20'

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COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700
DATE AUGUST 2017
FIGURE 8
APPENDIX A - SUBSURFACE INVESTIGATION

Our subsurface exploration program consisting of nine test borings was carried out by Britton Exploration of Los Gatos, California on June 29, 2017. Test borings FG-1 through FG-9 were drilled using a CME 45 track-mounted drilling rig with 6.0-inch diameter solid-stem augers. The test borings were drilled to depths of between 10.0 feet and 31.3 feet below the ground surface.

The materials encountered in the test borings were continuously logged in the field by a registered geotechnical engineer. The materials are described in accordance with the Unified Soil Classification System (ASTM D-2487). The locations of the test borings are shown on Figures 2A and 2B. The test boring logs are included in this appendix. A key for classification of the soil and laboratory test results is also included. The boring logs and related information show our interpretation of subsurface conditions on the date and at the locations indicated and it is not implied that they are representative of subsurface conditions at other locations or other times.

Material samples were obtained from the test borings at various depths. The samples were obtained using: 1) a 3.0-inch outside diameter (O.D.), 2.5-inch inside diameter (I.D.) Modified California split-spoon sampler fitted with a series of 6-inch long, thin-wall brass liners, and 2) a 2.0-inch O.D., 1.375-inch I.D. Standard Penetration Test (SPT) split spoon sampler. Representative samples were taken to the laboratory for evaluation and selected testing. Laboratory test results are summarized on the boring logs and are included in this appendix.

The split-spoon samplers were driven into the soil and rock materials at selected depths in the test borings by dropping a 140-pound hammer by rope and cathead methods through a 30-inch free fall for both the truck-mounted drill rig and the portable drill rigs. The penetration resistance was obtained by counting the number of blows of the hammer required to drive the sampler, in 6-inch intervals, a total distance of 18 inches. The blow counts recorded on the boring logs represent the number of blows required to drive the sampler the last 12 inches or until refusal at less than 12 inches. Refusal is typically considered 50 or more blows to drive the sampler 6 inches or less.

The test borings were drilled and backfilled with Portland cement grout under the permitting of the San Mateo County Environmental Health Department (permit 17-1103). In the pavement area at boring FG-6, the top 6 inches of the boring was plugged with asphalt concrete cold patch.
Figure FG-1

**Laboratory Tests**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Silty Sand w/Gravel (SM), dk yel brn (10yr4/6), loose, slightly moist, fine grained, gravel to 1&quot;, FILL</td>
</tr>
<tr>
<td>73 mc</td>
<td>Rock Clayey Sandstone, yel brn and v dk gry brn (10yr5/4,5/8,3/2), severely to moderately weathered, weak, soft matrix with hard zones, w/weathered gravel, BEDROCK</td>
</tr>
<tr>
<td>50/6&quot; mc</td>
<td>at 10', lt yel brn and yel brn (10yr6/4,5/6), severely weathered, weak, soft, with gry clay seams</td>
</tr>
<tr>
<td>50/2&quot; mc</td>
<td>at 15', yel brn, dk yel brn, w/gry (10yr5/4,4/4,5/1), moderately weathered, strong, hard</td>
</tr>
</tbody>
</table>

Unconf=3,658psf @ 2.0%  4.5+  6.4  137
Unconf=6,331psf @ 5.4%  8.6  134

*Boring terminated at 15.2 feet. No ground water encountered. Boring backfilled w/Portland cement grout. San Mateo County EHD permit #17-1103.*

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**Geotechnical Invest.**
Crystal Springs CTG Bike Trail,
San Mateo County, CA

---

**Equipment:** CME 45 Track Rig
**Elevation:** 306+/- ft (elevation datum⁺)
**Drilling Date:** 6/29/17
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Crystal Springs CTG Bike Trail,
San Mateo County, CA

Figure FG-2

6” Solid Stem Auger
140 lb. Auto-Trip Hammer

Equipment: CME 45 Track Rig
Elevation: 313+/-. ft (elevation datum1)
Drilling Date: 6/29/17

Laboratory Tests
Pocket Penetrometer (TSF)
Moisture Content (%)
Dry Density (pcf)
Blows/Foot (field)
Sampler Type*

BORING FG-2

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Silty Sand w/Gravel (SM), lt yel brn (10yr6/4), medium dense, slightly moist, fine grained, gravel to 1”, FILL</td>
</tr>
<tr>
<td>5</td>
<td>Sandstone, lt yel brn, yel brn, and dk yel brn (2.5y6/3) (10yr5/8,4/4), moderately weathered w/severely weathered fine grained (fg) matrix, weak, soft, w/moderately hard subangular clasts, FeO staining, BEDROCK</td>
</tr>
<tr>
<td>10</td>
<td>at 9’, dk yel brn and yel brn (10yr4/6,5/4), weak, slightly to moderately cemented, fg</td>
</tr>
</tbody>
</table>

Unconf=3,652psf @ 5.2%
5.2 138 76 mc

Unconf=5,486psf @ 1.7%
6.4 137 50/5 mc

Boring terminated at 10.0 feet.
No ground water encountered.
Boring backfilled w/Portland cement grout.
San Mateo County EHD permit #17-1103.


*spt=Std. Penetration Test  *mc=Modified California /2.5”ID  *ca=Calif./2.0”ID  *st=Shelby tube  *cb=core barrel  *b=bulk
6" Solid Stem Auger
140 lb. Auto-Trip Hammer

Equipment: CME 45 Track Rig
Elevation: 316+/- ft (elevation datum¹)
Drilling Date: 6/29/17

Laboratory Tests

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>BORING FG-3</td>
</tr>
<tr>
<td>4.5'</td>
<td>Silty Sand w/Gravel (SM), yel brn (10yr5/4,5/8), medium dense, moist, fine grained, w/weathered sandstone gravel to 1&quot;, FILL</td>
</tr>
<tr>
<td>5</td>
<td>at 3', dk yel brn (10yr3/6)</td>
</tr>
<tr>
<td>5.5'</td>
<td>at 5', dk yel brn w/yel brn (10yr4/6,5/8), w/variously colored gravel to 1&quot;</td>
</tr>
<tr>
<td>6'</td>
<td>Clayey Sandstone, yel brn, dk yel brn, and dk gry brn (10yr4/4,6/2), very severely to severely weathered, weak, soft, w/moderately weathered clasts of variable lithology, BEDROCK</td>
</tr>
<tr>
<td>10</td>
<td>at 15', yel brn w/gry (10yr5/4,5/8,6/1), very severely weathered, clayey matrix</td>
</tr>
</tbody>
</table>

Pocket Penetrometer (TFS)

<table>
<thead>
<tr>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Sample</td>
</tr>
</tbody>
</table>

Meaning of symbols:
- Øp = 44.0°, Cφ=0 psf
- Øu = 42.1°, Cu=0 psf
- CU saturated
- 11 spt
- 4.5+ Moisture Content (%)
- 10.3 Dry Density (pcf)
- 118 Blows/Foot (field)
- 21 mc Sampler Type


Figures FG-3a


*mc=Modified California /2.0"ID
*ca=Calif./2.0"ID
*st=Shelby tube
*cb=core barrel
*b=bulk

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Boring FG-3 Cont’d

Sheared Shale, v dk gry (10yr3/1), severely weathered w/moderately weathered zones, weak, soft w/hard zones, w/FeO and Mn staining on joints

Boring terminated at 20.5 feet.
No ground water encountered.
Boring backfilled w/Portland cement grout.
San Mateo County EHD permit #17-1103.

Laboratory Tests

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Blows/Foot (field)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>ROCK</td>
<td>123</td>
<td>8.3</td>
<td>50/6” mc</td>
<td></td>
</tr>
</tbody>
</table>

8.3 123 50/6” mc


* spt=Std. Penetration Test  * mc=Modified California /2.5"ID  * ca=Calif./2.0"ID  * st=Shelby tube  * cb=core barrel  * b=bulk
**6" Solid Stem Auger**

**140 lb. Auto-Trip Hammer**

**Equipment:** CME 45 Track Rig

**Elevation:** 319 +/- ft (elevation datum)

**Drilling Date:** 6/29/17

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**Laboratory Tests**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>SM</td>
<td>4.5+</td>
<td>11.9</td>
<td>121</td>
<td>18 mc</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**BORING FG-4**

- SM Silty Sand w/Gravel (SM), yel brn, brn, and dk yel brn (10yr5/8,4/3,4/4), medium dense, slightly moist, fine grained, w/weathered sandstone gravel to 1", FILL
- at 3', alternating layers of silty and clayey sand w/trace gravel to 1/2"
- at 5', v dk gry brn, brn, w/yel brn (10yr3/2,4/4,4/6), moderately weathered gravel to 3/4"
- at 10', lt olv brn, brn, brn yel (2.5y5/4) (10yr4/3,6/8), w/very severely and moderately weathered gravel to 2", friable

**Unconf=6,552psf @ 2.7%**

- 4.5+ 11.9 121 18 mc

**Unconf=4,180psf @ 1.7%**

- 4.5+ 9.3 116 20 mc

**Unconf=4,877psf @ 4.0%**

- 12.9 121 20 mc

---


* *spt=Std. Penetration Test * mc=Modified California /2.5"ID * ca=Calif./2.0"ID * st=Shelby tube * cb=core barrel * b=bulk
6" Solid Stem Auger
140 lb. Auto-Trip Hammer

Equipment: CME 45 Track Rig
Elevation: 319± ft (elevation datum¹)
Drilling Date: 6/29/17

Laboratory Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconf=1,916psf @ 5.6%</td>
<td>15.9</td>
<td>116</td>
<td>15 mc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unconf=7,923psf @ 10.1%</td>
<td>4.5</td>
<td>15.9</td>
<td>118 32 mc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Blowcount: 50 for last 5&quot;)</td>
<td>7.9</td>
<td>132 67/11&quot; mc</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

BORING FG-4 Cont’d

- Sandy Lean Clay (CL) Cont’d), brn, blk, red brn (10yr4/3,2/1)(2.5y5/3)(5yr4/4), very stiff, moist, completely weathered gravel to 1", blk clay at bottom of sample, FILL
- Clayey Sand (SC/CL), v dk gry and blk (2.5y3/1,2.5/1), medium dense, very moist, trace subangular gravel to 3/8", COLLUVIUM
- Clayey Sandstone, dk yel brn w/gry (10yr4/6,5/1), completely weathered, weak, soft, weathered to residual soil w/visible structure, fine grained, BEDROCK

at 30', very severely weathered, w/rounded gravel to 1”

Sheared Rock (Shale?), v dk gry and blk (5y3/1,2.5/1), severely to moderately weathered, very strong, hard, sheared/altered

Boring terminated at 30.9 feet.
No ground water encountered.
Boring backfilled w/Portland cement grout.
San Mateo County EHD permit #17-1103.


* spt=Std. Penetration Test  * mc=Modified California /2.5”ID  * ca=Calif./2.0”ID  * st=Shelby tube  * cb=core barrel  * b=bulk
**Laboratory Tests**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Depth (ft)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silty Sand w/Gravel (SM), lt yel brn, bm, yel brn (10yr5/8,4/3,3/4), loose to medium dense, slightly moist, fine grained, gravel to 2&quot;, FILL</td>
<td>0</td>
<td>8.3</td>
<td>113</td>
<td>15 mc</td>
</tr>
<tr>
<td>Clayey Sandstone, lt yel brn, bm and yel brn (10yr6/4,4/3,5/8), very severely to completely weathered, weak, soft matrix with severely weathered gravel to 2&quot;, BEDROCK</td>
<td>5</td>
<td>10.4</td>
<td>121</td>
<td>21 mc</td>
</tr>
</tbody>
</table>

Unconf=1,546psf @ 1.4%

Unconf=8,846psf @ 2.7%

---

*Boring terminated at 15.0 feet. No ground water encountered.
Boring backfilled w/Portland cement grout.
San Mateo County EHD permit #17-1103.

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*st=Std. Penetration Test *mc=Modified California /2.5"ID *ca=Calif./2.0"ID *st=Shelby tube *cb=core barrel *b=bulk
6" Solid Stem Auger
140 lb. Auto-Trip Hammer

Equipment: CME 45 Track Rig
Elevation: 332+/– ft (elevation datum²)
Drilling Date: 6/29/17

Laboratory Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
</table>

Boring FG-6

Unconf=4,790psf @ 3.7%

- Asphalt Concrete Pavement, 5"
- Silty Sand w/Gravel (SM), yel brn (10yr5/8), medium dense, moist, RESIDUAL SOIL

8.7 126 51 mc

- Clayey Sandstone, dk gry brn, dk brn, and dk yel brn (10yr4/2,3/3,4/6), very severely weathered, weak, soft, w/severely and moderately weathered clasts of variable lithology, BEDROCK

- Sheared Rock (Shale?), v dk gry brn w/yel brn (10yr3/2,5/4), severely weathered, weak, soft, w/moderately weathered zones, w/wt mineralization (non-reactive to HCL), rock is altered and sheared, lithology is not obvious

- at 9', blk and v dk gry (5y2.5/1,3/1), very severely weathered, very soft, very moist

Unconf=7,061psf @ 2.3%

- Clayey Sandstone, dk gry brn, dk brn, and dk yel brn (10yr4/2,3/3,4/6), very severely weathered, weak, soft, w/severely and moderately weathered clasts of variable lithology, BEDROCK

10.9 122 60 mc

- Sheared Rock (Shale?), v dk gry brn w/yel brn (10yr3/2,5/4), severely weathered, weak, soft, w/moderately weathered zones, w/wt mineralization (non-reactive to HCL), rock is altered and sheared, lithology is not obvious

- at 9', blk and v dk gry (5y2.5/1,3/1), very severely weathered, very soft, very moist

10.9 126 20 mc

- Boring terminated at 10.5 feet.
- No ground water encountered.
- Boring backfilled w/Portland cement grout.
- San Mateo County EHD permit #17-1103.


*Boring terminated at 10.5 feet.
No ground water encountered.
Boring backfilled w/Portland cement grout.
San Mateo County EHD permit #17-1103.

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Geotechnical Invest.
Crystal Springs CTG Bike Trail,
San Mateo County, CA

Figure FG-6
6" Solid Stem Auger
140 lb. Auto-Trip Hammer

Equipment: CME 45 Track Rig
Elevation: 342+/-. ft (elevation datum¹)
Drilling Date: 6/29/17

Laboratory Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>BORING FG-7</th>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Feet (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM</td>
<td>Silty Sand w/Gravel (SM), dk brn mottled w/yel bm (10yr3/3,5/8), medium dense, moist, fine grained, w/gravel to 1&quot;, FILL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Unconf=5,492psf @ 2.7%</td>
<td>8.9</td>
<td>125</td>
<td>21 mc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td>Clayey Sand w/Gravel (SC/GC), dk gry bm, dk bm, dk gry bm (10yr4/2,3/3,4/6), medium dense, moist, severely weathered gravel of variable lithology to 1.5&quot;, chaotic appearance, FILL?/RESIDUAL SOIL?</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Unconf=7,411psf @ 2.5%</td>
<td>9.0</td>
<td>135</td>
<td>58 mc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ROCK</td>
<td>Sheared Rock (Shale?), dk gry, blk, and gry bm (2.5y4/1) (5y2.5/1) (10yr3/6), very severely weathered, weak, soft, w/zones of moderately weathered and hard rock, FeO staining at partings, BEDROCK</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Unconf=4,412psf @ 4.1%</td>
<td>9.7</td>
<td>134</td>
<td>58 mc</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


* spt=Std. Penetration Test  * mc=Modified California 2.5"ID  * ca=Calif./2.0"ID  * st=Shelby tube  * cb=core barrel  * b=bulk

*地下水* encountered at 20'

log cont'd on next page

Figure FG-7a
**Laboratory Tests**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>Sheared Rock (Shale?) (cont'd), blk (2.5y2.5/1), very severely weathered, weak, soft w/moderately hard zones, w/ wt mineralization (HCL-), indistinct lithology and structure</td>
</tr>
<tr>
<td>25</td>
<td>at 25', dk gry and blk (N4/, N2.5/) severely weathered, weak, soft</td>
</tr>
<tr>
<td>26.5</td>
<td>Boring terminated at 26.5 feet. Ground water encountered at 18.0 feet. Boring backfilled w/Portland cement grout. San Mateo County EHD permit #17-1103.</td>
</tr>
</tbody>
</table>

**Notes:**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BORING FG-8</td>
</tr>
<tr>
<td>0</td>
<td>GM Silty Gravel (GM), v dk olv gry (5y3/1), loose, slightly moist, 3/4&quot; gravel, FILL</td>
</tr>
<tr>
<td></td>
<td>GM Silty Sand w/Gravel (SM), lt and dk yel brn (10yr6/4,3/4), loose to medium dense, moist, fine grained, w/weathered sandstone gravel to 2&quot;, FILL</td>
</tr>
<tr>
<td></td>
<td>at 5', loose, severely to moderately weathered gravel of variable lithology, FILL?/ RESIDUAL SOIL?</td>
</tr>
<tr>
<td>5</td>
<td>GM Unconf=3,431psf @ 1.4% 7.5 122 13 mc</td>
</tr>
<tr>
<td>10</td>
<td>SM Clayey Sandstone, yel brn and dk yel brn (10yr5/8,4/4), completely to very severely weathered, weak, soft, w/gravel to 1&quot; in moderately cemented silty sand matrix (SM), BEDROCK</td>
</tr>
<tr>
<td>15</td>
<td>ROCK at 15', yel brn, dk yel brn, and str brn (10yr4/6,3/3) (7.5yr4/6), w/completely to moderately weathered gravel of variable lithology to 2&quot;</td>
</tr>
<tr>
<td></td>
<td>ROCK Sheared Rock, dk brn, dk yel brn, and v dk gry brn (10yr3/3,4/6,3/2), very severely to completely weathered, weak, soft w/mod. hard zones, indistinct lithology and structure</td>
</tr>
<tr>
<td>20</td>
<td>log cont'd on next page</td>
</tr>
</tbody>
</table>


| Equipment: CME 45 Track Rig |
| Elevation: 344+/ - ft (elevation datum1) |
| Drilling Date: 6/29/17 |

---

Fisher Geotechnical
Civil and Geotechnical Engineering

Geotechnical Invest.
Crystal Springs CTG Bike Trail,
San Mateo County, CA

Figure FG-8a
Equipment: CME 45 Track Rig  
Elevation: 344+/- ft (elevation datum¹)  
Drilling Date: 6/29/17

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>ROCK Sheared Rock (cont'd), dk bm, dk yel bm, v dk gry bm (10yr3/3,4/6,3/2), very severely to completely weathered, weak, soft w/mod. hard zones, indistinct lithology and structure</td>
</tr>
<tr>
<td>25</td>
<td>No ground water encountered.</td>
</tr>
<tr>
<td>30</td>
<td>Boring terminated at 21.5 feet.</td>
</tr>
<tr>
<td>35</td>
<td>Boring backfilled w/Portland cement grout.</td>
</tr>
<tr>
<td>40</td>
<td>San Mateo County EHD permit #17-1103.</td>
</tr>
</tbody>
</table>

Laboratory Tests

<table>
<thead>
<tr>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion test at 20.5'-21.0' (see data sheet)</td>
<td>4.5</td>
<td>13.3</td>
<td>122</td>
<td>37 mc</td>
</tr>
<tr>
<td>Unconf=5,202psf @ 3.4%</td>
<td>4.5+</td>
<td>13.3</td>
<td>122</td>
<td>37 mc</td>
</tr>
</tbody>
</table>

**6" Solid Stem Auger**

**140 lb. Auto-Trip Hammer**

**Equipment:**  
CME 45 Track Rig

**Elevation:**  
346 +/- ft (elevation datum²)

**Drilling Date:**  
6/29/17

---

**Laboratory Tests**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>BORING FG-9</td>
<td>6.0</td>
<td>116</td>
<td>8 mc</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>GM</td>
<td>Silty Gravel (GM), v dk olv gry (5y3/1), FILL</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td>Silty Sand w/Gravel (SM), lt yel brn (10yr6/4), loose, slightly moist, fine grained, w/sandstone gravel to 1&quot;, FILL</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>ML</td>
<td>Sandy Silt w/Gravel (ML), v dk gry brn w/yel brn (10y3/2,5/8), very stiff, moist, w/very severely to completely weathered gravel to 1.5&quot;, COLLUVIUM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>SC</td>
<td>Clayey Sand (SC), v dk gry brn (10y3/2), medium dense, moist to very moist, fine grained sand, w/subangular to subround gravel to 3/8&quot;, uniform color, COLLUVIUM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>log cont'd on next page</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

¹ Datum:  

* spt=Std. Penetration Test  
* mc=Modified California /2.5"ID  
* ca=Calif./2.0"ID  
* st=Shelby tube  
* cb=core barrel  
* b=bulk
6" Solid Stem Auger
140 lb. Auto-Trip Hammer

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>Clayey Sand (SC) (cont'd), v dk gry brn (10yr3/2), medium dense, moist to very moist, fine grained sand, w/sub-angular to sub-rounded gravel to 3/8&quot;, uniform color, COLLUVIUM</td>
</tr>
<tr>
<td>25</td>
<td>Sheared Rock (Shale?), brn, gry, blk (10yr4/3,5/1,2/1), severely weathered matrix, weak, soft, w/moderately weathered zones, sheared/ altered zones w/wt mineralization (HCL-), indistinct lithology and structure, BEDROCK</td>
</tr>
<tr>
<td>30</td>
<td>at 25', v dk gry and dk olv gry (5y3/1,3/2,4/2), very intensely fractured</td>
</tr>
<tr>
<td>35</td>
<td>at 30', blk (2.5y2.5/1), w/polished surfaces, very thinly foliated (from weathering)</td>
</tr>
<tr>
<td>40</td>
<td>Boring terminated at 31.3 feet. No ground water encountered. Boring backfilled w/Portland cement grout. San Mateo County EHD permit #17-1103.</td>
</tr>
</tbody>
</table>

Laboratory Tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Pocket Penetrometer (TSF)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Blows/Foot (field)</th>
<th>Sampler Type*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconf=3,884psf @ 3.0%</td>
<td>12.8</td>
<td>124</td>
<td>34 mc</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unconf=5,537psf @ 3.6%</td>
<td>5.7</td>
<td>143</td>
<td>73/10&quot; mc</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


Figure FG-9b
**TEST DATA**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>Sample Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content (%)</td>
<td>10.5</td>
<td>11.0</td>
<td>10.1</td>
<td>9.8</td>
<td>FG3 @ 4.0</td>
</tr>
<tr>
<td>Dry Unit Weight (pcf)</td>
<td>117.2</td>
<td>120.7</td>
<td>119.3</td>
<td>122.0</td>
<td></td>
</tr>
<tr>
<td>Saturation (%)</td>
<td>65.0</td>
<td>75.3</td>
<td>66.4</td>
<td>69.7</td>
<td></td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.44</td>
<td>0.40</td>
<td>0.41</td>
<td>0.38</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>Sample Description:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content (%)</td>
<td>14.7</td>
<td>15.2</td>
<td>13.3</td>
<td>13.2</td>
<td>Silty Sand w/Gravel (SM), Yel Brn, 10yr5/4,5/8, w/completely weathered sandstone gravel to 1&quot;, Fill</td>
</tr>
<tr>
<td>Dry Unit Weight (pcf)</td>
<td>124.4</td>
<td>123.5</td>
<td>123.1</td>
<td>128.4</td>
<td></td>
</tr>
<tr>
<td>Saturation (%)</td>
<td>100.0</td>
<td>100.0</td>
<td>0.97</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>Void Ratio</td>
<td>0.44</td>
<td>0.36</td>
<td>0.37</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>Height (in.)</td>
<td>1.1399</td>
<td>1.1728</td>
<td>1.1633</td>
<td>1.1397</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. Results somewhat variable based on differing material composition along sample shear planes.
2. Ultimate Stress taken as low point of first decrease in shear stress.

---

**Shear Stress vs. Normal Stress**

- Peak: $\theta = 44.0^\circ$, $C = 0$ psf
- Ultimate: $\theta = 42.1^\circ$, $C = 0$ psf

$y = 0.9656x$

$y = 0.9024x$

**Vertical Deformation vs. Shear Displacement**

- 1075 psf
- 2035 psf
- 2554 psf
- 3032 psf

---

**Direct Shear Test**

ASTM D 3080 Modified

Client: **Bellecci Associates**

Project: **Crystal Springs CTG Bike Trail**

Project No: **3700**  
Date: **7/18/17**
**Sample Location:**

- FG3/T5 @ 16.0'

**Sample Description:**

- Mottled Dk Yel Brn
- and Yel Brn Severely Weathered
- Clayey Sandstone

**Notes:**

- Consolidated Undrained
- 2-hour min. inundation and load
- G=2.70 assumed, strain rate 0.029/min.
- At-test density, void ratio, and saturation are approximate based on test method limitations

**Direct Shear Test**

- ASTM D 3080 Modified

---

**Client:** Bellecci Associates  
**Project:** Crystal Springs CTG Bike Trail  
**Project No.:** 3700  
**Date:** 7/25/17  
**FISHER GEOTECHNICAL**
To: Raymond Fisher  
Fisher Engineering  
345 Alameda Ave.  
Half Moon Bay, CA  94019

From: Gene Oliphant, Ph.D. \ Randy Horney  
General Manager \ Lab Manager

The reported analysis was requested for the following location:  
Location : 3700 CTG BIKE TRAIL Site ID : FG-3@4.5-6.0 FT.  
Thank you for your business.

* For future reference to this analysis please use SUN # 75056-156666.

__________________________________________________________________________  
EVALUATION FOR SOIL CORROSION

Soil pH 6.81
Minimum Resistivity 4.02 ohm-cm (x1000)
Chloride 5.6 ppm 0.00056 %
Sulfate 8.2 ppm 0.00082 %

METHODS  
ph and Min.Resistivity CA DOT Test #643  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
To: Raymond Fisher  
Fisher Engineering  
345 Alameda Ave.  
Half Moon Bay, CA 94019

From: Gene Oliphant, Ph.D. \ Randy Horney /  
General Manager \ Lab Manager /

The reported analysis was requested for the following location:  
Location: 3700 CTG BIKE TRAIL  Site ID: FG-3@10.5-11.0.  
Thank you for your business.

* For future reference to this analysis please use SUN # 75056-156667.

-----------------------------------------------
EVALUATION FOR SOIL CORROSION

Soil pH  6.51
Minimum Resistivity  3.48 ohm-cm (x1000)
Chloride  4.9 ppm  00.00049 %
Sulfate  6.4 ppm  00.00064 %

METHODS
pH and Min.Resistivity CA DOT Test #643  
Sulfate CA DOT Test #417,  Chloride CA DOT Test #422
To: Raymond Fisher  
Fisher Engineering  
345 Alameda Ave.  
Half Moon Bay, CA 94019

From: Gene Oliphant, Ph.D.  \ Randy Horney  
General Manager  \ Lab Manager

The reported analysis was requested for the following location:  
Location: 3700 CTG BIKE TRAIL  Site ID: FG-8@20.5-21.0.  
Thank you for your business.

* For future reference to this analysis please use SUN # 75056-156668.

-----------------------------------------------
EVALUATION FOR SOIL CORROSION

Soil pH  5.58

Minimum Resistivity  2.95 ohm-cm (x1000)

Chloride  4.8 ppm  00.00048 %

Sulfate  2.9 ppm  00.00029 %

METHODS
pH and Min. Resistivity CA DOT Test #643  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
APPENDIX B - SLOPE STABILITY ANALYSES

Four limit-equilibrium slope stability total-stress analyses were performed using the computer program PCSTABL5M to evaluate the degree of stability and the most probable geometry of potential failure surfaces under static and seismic conditions for the existing slope and for the slope with the new retaining wall sections. From our review of the bike trail cross sections from your firm and our test boring logs, we concluded that the most critical slope area is at 169+00, where there is the fill is deepest and where the existing slope inclination is about 1.3H:1V. This slope was used in our analyses. A seismic slope stability (pseudo-static) coefficient\textsuperscript{12} of 0.2g was used to model intense ground shaking in our seismic slope stability analyses.

Our first analyses evaluated the existing slope under seismic conditions. Using our test boring and laboratory test data, an undrained peak shear strength envelope defined by $\phi=42^\circ$, $C=0$ psf was used for the fill. An undrained peak shear strength envelope defined by $\phi=0^\circ$, $C=900$ psf was used for colluvial soil. An undrained peak shear strength envelope defined by $\phi=31^\circ$, $C=1500$ psf was used for the severely weathered bedrock that underlies the colluvial soil. The analyses considered both shallow and deep failure circles at various initiation and termination points on the slope and behind the slope crest. The results of this analysis indicate that the slope has surface instability ($FS=0.91$) during the maximum earthquake; however, the slope appears to have an adequate degree of stability ($FS=1.11$) against deeper failures.

Our second analyses evaluated the existing slope with the proposed retaining wall under seismic conditions and using the same undrained, peak shear strength parameters. The analysis also incorporated a tieback force of 2,400 lbs. per foot of wall width to artificially model a restraint to active wall pressure. The results of this analysis indicate that the slope appears to have an adequate degree of stability ($FS=1.21$) against failure under seismic conditions.

Our third analyses evaluated the existing slope under static conditions. An undrained ultimate shear strength envelope defined by $\phi=42^\circ$, $C=0$ psf was used for the fill. An undrained shear strength envelope defined by $\phi=20^\circ$, $C=0$ psf was used for colluvial soil. An undrained ultimate shear strength envelope defined by $\phi=42^\circ$, $C=0$ psf was used for the severely weathered bedrock that underlies the colluvial soil. The analyses considered both shallow and deep failure circles at various points on the slope and behind the slope crest. The results of this analysis indicate that the slope appears to be stable by a reasonable margin under static conditions, but appears to have a degree of stability ($FS=1.33$) slightly lower than the accepted standard of $FS=1.50$ against failure under static conditions.

\textsuperscript{12}Caltrans Bridge Design Specifications, August 2004, Section 5.2.2.3
Our fourth analyses evaluated the existing slope with the proposed retaining wall under static conditions and using the same undrained, peak shear strength parameters. The analysis also incorporated a tieback force of 2,400 pounds per foot of wall width to artificially model a restraint to active wall pressure. The analyses considered both shallow and deep failure circles at various points on the slope and behind the slope crest. The results of this analysis indicate that the slope appears to be stable by a reasonable margin under static conditions, but appears to have a degree of stability (FS=1.39) slightly lower than the accepted standard of FS=1.50 against failure under static conditions.

A ground water surface was included in each of the analyses above and was conservatively assumed to be above the native colluvial soil, although no groundwater was encountered in test boring FG-4 drilled near 169+00. Summary plots of the stability analyses are included in this appendix.
CTG Bike Trail - Orig. Grade - Seismic Profile FG4, undrained strength, K=0.2g
All surfaces evaluated. C:CTG1.PLT  By: rif  08-22-17  5:18pm
CTG Bike Trail - New Ret. Wall - Seismic Profile FG4, undrained strength, K=0.2g
Ten Most Critical. C:CTG2.PLT By: rlf 08-22-17 5:28pm

<table>
<thead>
<tr>
<th>#</th>
<th>No.</th>
<th>Soil</th>
<th>TotWt</th>
<th>SatWt</th>
<th>C</th>
<th>Phi</th>
<th>Ru</th>
<th>Pore</th>
<th>Piez.</th>
<th>Param</th>
<th>Press</th>
<th>Surf</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1.21</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>42</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>W1</td>
<td>0</td>
<td>W1</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>1.22</td>
<td>120</td>
<td>120</td>
<td>900</td>
<td>0</td>
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<td>0</td>
<td>0</td>
<td>W1</td>
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<td>W1</td>
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<tr>
<td>3</td>
<td>3</td>
<td>1.26</td>
<td>130</td>
<td>130</td>
<td>1500</td>
<td>31</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>W1</td>
<td>0</td>
<td>W1</td>
</tr>
</tbody>
</table>

Elev. (ft)

PCSTABL5M FSmin=1.21 X-Axis (ft)
CTG Bike Trail - Orig. Grade - Static Profile FG4, undrained ultimate strength
All surfaces evaluated. C:CTG4.PLT By: rlf 08-23-17 9:35pm
CTG Bike Trail- New Ret. Wall-Static Profile FG4, undrained ultimate strength
Ten Most Critical. C:CTG3.PLT By: rlf 08-23-17 9:41pm

<table>
<thead>
<tr>
<th>No.</th>
<th>#</th>
<th>FS</th>
<th>Soil Tot Ht Sat Ht</th>
<th>C</th>
<th>Phi</th>
<th>Ru</th>
<th>Pore Press</th>
<th>Piez.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1.39</td>
<td>1</td>
<td>130</td>
<td>130</td>
<td>0</td>
<td>42</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>1.42</td>
<td>2</td>
<td>120</td>
<td>120</td>
<td>0</td>
<td>20</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>1.42</td>
<td>3</td>
<td>130</td>
<td>130</td>
<td>0</td>
<td>42</td>
<td>0</td>
</tr>
</tbody>
</table>

Elev. (ft)

PCSTABL5M FSmin=1.39 X-Axis (ft)
APPENDIX C - RETAINING WALL DESIGN DETAILS
See Figure C-2 for Retaining Wall Data Table and Notes

RETAINING WALL DETAIL
NO SCALE

Fisher Geotechnical
Civil and Geotechnical Engineering

COMPLETE THE GAP BIKE TRAIL
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 3700
DATE AUGUST 2017
FIGURE C-1
## RETAINING WALL DATA

<table>
<thead>
<tr>
<th>Wall Height Above Existing Grade (ft)</th>
<th>Wall Design Height (W/4' Deep Loose/Creen zone) (ft)</th>
<th>Minimum Beam Size* ASTM A992</th>
<th>Flange Width (in)</th>
<th>Minimum Beam Length (ft)</th>
<th>Beam Spacing (ft)</th>
<th>Wood Lagging Height (ft)</th>
<th>Final Grade Depth Below Top of Wall-Front Face (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>W8x35</td>
<td>8</td>
<td>14</td>
<td>6</td>
<td>5</td>
<td>0 to 3</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>W10x54</td>
<td>10</td>
<td>18</td>
<td>6</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>9</td>
<td>W10x60</td>
<td>10</td>
<td>22</td>
<td>6</td>
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<td>W12x79</td>
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<td>25</td>
<td>6</td>
<td>11</td>
<td>9</td>
</tr>
<tr>
<td>9</td>
<td>13</td>
<td>W12x96</td>
<td>12</td>
<td>30</td>
<td>6</td>
<td>13</td>
<td>11</td>
</tr>
</tbody>
</table>

*Note: For uniformity, the use of one or two beam sizes for a wall that tapers in height is more practical than using multiple beam sizes. Minimum beam size includes additional sacrificial thickness for corrosion protection.

### Notes:

1. Subsurface obstructions such as rocks, tree roots, and debris may be encountered during excavation or drilling for pile installation. Obstructions shall be cored through or removed with minimal disturbance to adjacent ground as approved by the engineer. Hard bedrock zones may also be encountered.

2. The contractor shall provide proper configuration and/or shoring of excavations in accordance with occupational safety laws. In addition to occupational safety requirements, excavation method shall ensure preservation of adjacent underground utilities.

3. Prior to placing the first course of wood lagging, tops of soldier pile concrete backfill shall be adjusted to the specified elevation by removing concrete between beam flanges to create a uniform level footing for the lagging. (The contractor may elect to block-out the area between flanges to the specified elevation prior to placing concrete backfill.) Wood lagging shall be installed level and plumb. ¼” thick spacers shall be installed between lagging courses. Spacers shall have a nominal dimension of 4”x6” and shall be pressure treated wood or an approved alternative durable material.

4. All earth fill, graded surfaces, and any other areas disturbed by the contractor’s operations shall be hydro-seeded as directed by the engineer.