Structural Evaluation

Five Wounds Trail – Trestle Bridge

City of San Jose

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→ The Power of Commitment



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Executive Summary

The trestle bridge at Five Wounds Trail is located north of the intersection of Story Road and Senter Road where it crosses Coyote Creek in San Jose, CA. A structural evaluation of the existing bridge has been performed and is summarized in this report. This report is subject to, and must be read in conjunction with, the limitations set out in section 1 and the assumptions and qualifications contained throughout the Report.

The existing trestle exhibits the following issues:

- Bridge structure is only adequate to resist its own weight and short-term wind loading.
- Inadequate lateral force resisting system.
- Trestle supports are at risk of partial collapse during a relatively high probability, low severity seismic event, quantifiable as a seismic event with a 20% risk of occurrence within 50 years.
- Further deterioration of the structure will occur, and existing structural deficiencies will get worse.

If the existing structure is to remain in place with no retrofit, the vertical capacity of the bridge will eventually deteriorate to zero and portions of the structure will be at risk of partial collapse. In the bridge's current state there is the potential for partial collapse.

If preservation of the existing structure is desired, to mitigate the ongoing risk of collapse due to a seismic event a retrofit of the bridge support is recommended. Two retrofit approaches are looked at in this report:

- Retrofit Collapse Prevention/Self-Sustained Retrofit
    - Replace members of trestle/bridge supports using a similar material.
    - Strengthen connections between new and/or existing members of trestle/bridge supports.
    - Pedestrian/Bicycle access will be prohibited.
    - This is the most feasible option per report Section 2.3.2.
    - If a multi-use path is desired, a new steel bridge could be constructed adjacent to the existing structure.
- Retrofit for Multi-Use Path
    - Designing steel lateral frames to replace the insufficient lateral system of the existing structure.
    - Option would need to be designed for a higher seismic load than the collapse prevention alternative per report section 2.3.3.
    - Numerous non-structural elements such as concrete decking, guardrails, signage, etc. would be required as part of this retrofit for multi-use access.

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Appendices

Appendix A Wood Assessment by Porter & Associates
Appendix B Geotechnical Investigation
Appendix C Draft Hydraulic Analysis Technical Memorandum
Appendix D Conceptual Sketches of Analysis Options 2 & 3

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1. Introduction

The trestle bridge at Coyote Creek is located north of the intersection of Story Road and Senter Road, in San Jose, CA. Originally built in the 1920s to serve railway service, the bridge crosses Coyote Creek along the master planned Five Wounds Trail alignment. The most recent fire below the bridge has caused significant charring and damage to supports.

1.1 Purpose of this report

This report presents the results of an evaluation of the structural stability of the existing bridge and the feasibility of structural retrofit. This evaluation is intended to aid in decision-making related to repair or replacement of the bridge. Information has been provided related to the ongoing risks associated with leaving the bridge in its current condition, preservation efforts, and the potential effort involved in a structural retrofit for the purpose of utilizing the bridge as a multi purpose trail crossing.

1.2 Scope and limitations

This report has been prepared by GHD for City of San Jose and may only be used and relied on by City of San Jose for the purpose agreed between GHD and City of San Jose as set out in section 1.1 of this report. GHD otherwise disclaims responsibility to any person other than City of San Jose arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible. The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The conclusions and recommendations in this report are based on an analysis of the structure performed in accordance with the American Society of Civil Engineers (ASCE) 41-17, “Seismic Evaluation and Retrofit of Existing Buildings”. This provision provides analysis methods and acceptance criteria for a variety of building types, lateral force resisting systems, and seismic hazard*s*. Based on the age of the existing bridge structure and the expected scope of retrofit efforts, the evaluation methods outlined in the standard are applicable for the seismic evaluation of this bridge structure. The assumptions and limitations of the ASCE-41 evaluation are summarized further in the methodology section of this report. GHD disclaims liability arising from any of the assumptions being incorrect.

The analyses, opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring after the date that the report was prepared. GHD has prepared this report based on information provided by City of San Jose and others who provided information to GHD, which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

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1.3 Methodology and Assumptions

1.3.1 Assessment of Existing Conditions

Information regarding the current condition of the structure has been determined by a visual inspection of the bridge. As-built drawings or original plans are not available. The bridge layout has been determined via 3D aerial scan, verified by visual observations and measurements at accessible portions of the bridge. The primary structural elements of the bridge and structural connections are exposed to view, and inspection from below has been sufficient to determine the general size and state of deterioration at primary structural elements. A comprehensive inspection of the existing wood elements of the bridge was performed by Porter and Associates, summarized in a report dated 08/17/2022 and attached as Appendix A. This inspection included visual grading of wood species and grade based on criteria in the National Design Specification for Wood Construction (NDS) and from the Western Wood Products Association (WWPA). A geotechnical investigation report has been prepared by Parikh Consultants, Inc. dated 10/18/2022, attached as Appendix B. The report summarizes existing soil conditions and evaluates pile capacities in terms of kips vs feet of penetration below grade.

1.3.2 Seismic Evaluation

For the ASCE-41 evaluation, a tier 1 checklist-based screening has been completed for a typical bridge support bent. A Risk Category of II has been assumed for the structure, and the structural system has been approximated as wood light-framing for the purposes of the tier 1 evaluation.

Based on deficiencies found in the Tier 1 evaluation, a Tier 2 deficiency-based analysis has been used to determine which elements of the structure require would require retrofit. A K-factor, accounting for the uncertainty of field observations and material testing, is outlined in the ASCE 41 for different types of evaluations and structural classifications. A value of 0.9 has been assumed for this structure, reflecting a 10% reduction in structural capacity of existing elements. The tier 2 seismic evaluation has been developed for three potential retrofit scenarios, identified in this report as options one, two and three. Different performance levels and seismic hazards, as defined by ASCE 41, have been selected for each scenario.

Performance levels relevant to this analysis are Collapse Prevention (CP) and Life Safety (LS):

- CP performance is defined as the post-earthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin of safety against collapse.
- LS performance is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin of safety against the onset of total or partial collapse.

The seismic hazards relevant to this analysis are BSE1E and BSE2E:

- BSE1E corresponds to an earthquake event that has a 20% chance of occurrence within 50 years, or a mean recurrence interval of 250 years. This level of seismic hazard is significantly less severe than what would be considered for new construction.
- BSE2E corresponds to an earthquake event that has a 5% chance of occurrence within 50 years. This level of seismic hazard is roughly comparable in magnitude to the design basis earthquake for new buildings

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Analysis 1 - No Retrofit/Existing Condition

Based on the existing condition of the bridge, a BSE1E seismic hazard is applied to the structure to determine whether the structure as-is satisfies the CP performance goal, or if not, to quantify the existing structural deficiencies. This combination of hazard and performance level qualifies as a limited performance objective in the ASCE 41. This option should only be considered if the risk of collapse determined by analysis is acceptable for the foreseeable lifespan of the structure.

Analysis 2 - Retrofit for Collapse Prevention/Self-Sustained

Based on a BSE1E seismic hazard, the existing structural layout is analyzed to determine what replacements of damaged material and modifications to existing connections would be required to achieve a CP performance level. This combination of hazard and performance level qualifies as a limited performance objective in the ASCE 41. This retrofit option should be considered if historical preservation of the structure's character is desired, but pedestrian access will be prohibited on the bridge.

Analysis 3 - Retrofit for Multi-purpose Trail Use

In order to retrofit the bridge for multi-purpose trail use (pedestrians, bicycles, etc.), the bridge would need to be designed for a combination of seismic hazard and performance level that is called the "Basic Performance Objective". This consists of a BSE2E seismic hazard and a retrofit of structural deficiencies resulting in LS performance. This retrofit scenario would be significantly more costly than the retrofit for collapse prevention because a larger seismic hazard must be used, and the structural elements must achieve a larger margin of safety against collapse. It is anticipated that a supplementary support system such as steel braced frames with a similar layout to the existing bents would be necessary to achieve this basic performance objective.

1.3.3 Hydraulic Analysis

A hydraulic analysis of Coyote Creek was conducted in the vicinity of the bridge to provide the following:

- A hydraulic assessment to estimate the water surface profile, flow depth, and flow velocity in Coyote Creek in the vicinity of the existing bridge
- An evaluation of the scour at the existing bridge

The Santa Clara Valley Water District's (SCVWD) existing steady-flow one-dimensional HEC-RAS model for Coyote Creek was used as the basis for the hydraulic modeling. The flow rates used for the analysis were the 100-year design peak flow rates for Coyote Creek, which are based on the 2020 SCVWD Design Flows Manual.

Additional information on the methods and assumptions for the hydraulic analysis is included in the Technical Memorandum (TM) in Appendix C.

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2. Evaluation Results

2.1 Assessment of Existing Conditions

The bridge consists of timber post and beam construction. The bridge deck consists of Square wooden ties spanning across two multi-ply wood bridge girders. Girders span 32’-6” typically, with supports consisting of timber bents, varying in height, with sawn lumber cap beams supported on wood pole columns. Poles are assumed to be pinned at the base and are braced by diagonal and horizontal braces extending the full width of the bent, varying from one to three levels, bolted to each wood pole column. An image of the 3D scan of the bridge, and a sketch of a typical bent elevation are included below.

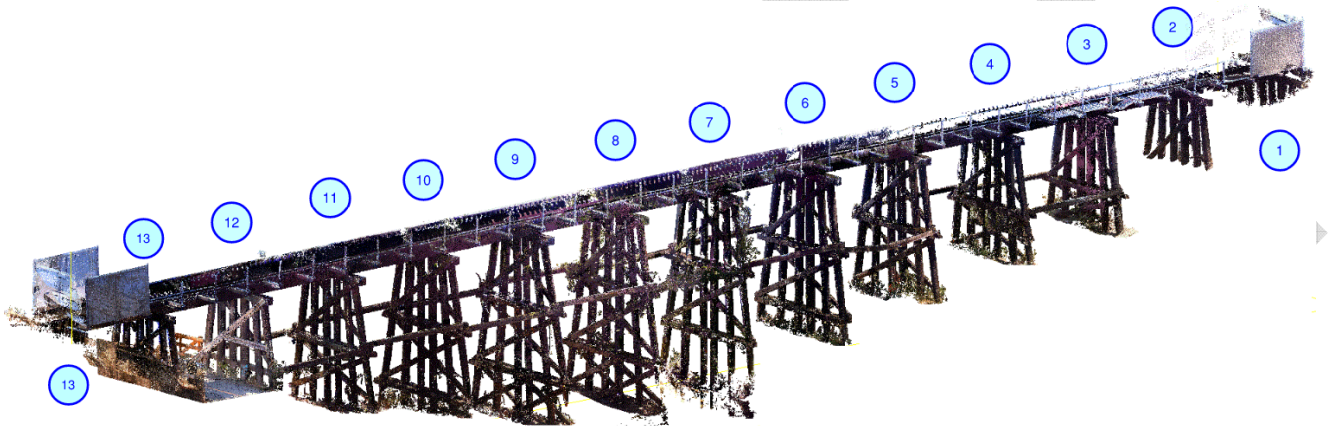


Figure 1 Isometric View, looking Southeast

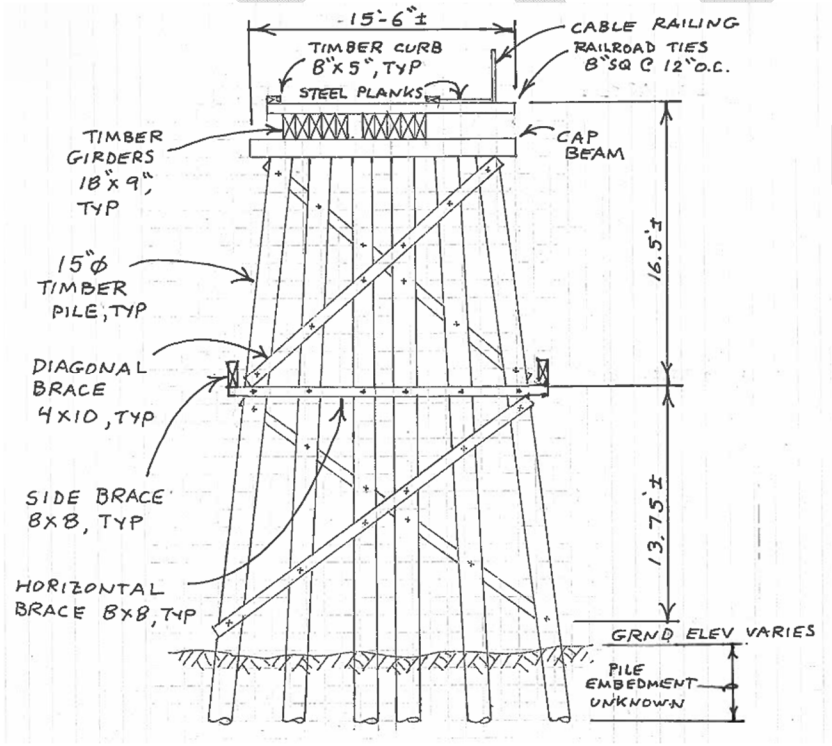


Figure 2 Typical Bent Elevation

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2.1.1 Timber Deterioration

At all wood elements, the long service life of the existing material has resulted in some degree of deterioration primarily due to wood decay fungi. This damage is most notable at the cap beams at the top of the bents, the longitudinal side braces, and the ends of the diagonal braces (as shown in Figure 2). These locations see the most severe rainwater exposure, and numerous moisture-trapping conditions occur at connections that would accelerate the rate of fungal decay. The following elements have deterioration that is likely to result in more than 50% reduction to their load carrying capacity.

- Cap Beams at bents 5 and 10
- Girder 9 in Span 4
- Longitudinal side braces in all spans
- Diagonal braces at end bolted connections in all bents
- Horizontal braces at end bolted connections in all bents

2.1.2 Fire Damage

A fire appears to have happened below the bridge near spans 1 through 4, spreading to piles, sway braces, longitudinal braces, cap beams, girders, and deck ties. At these elements, char depth was measured to be ¾", and a uniform thickness of lost material can be assumed as 1" thickness on all exposed faces. Longitudinal sway braces have lost all capacity at their end connections, and in span 4 the longitudinal braces have collapsed. Pile 1 at Bent 4 has lost approximately half of its thickness in the lower level of bracing.

A smaller fire has caused damage in Span 13 of the bridge, but damage is localized to the deck ties and span girders in this area.

2.1.3 Localized Damage or Unusual Construction

The following elements have been damaged to the point of lost capacity or have been replaced with non-typical elements that may impact their expected capacity.

- At Bent 8, pile 1 the soil around the pile has been scoured away by the adjacent creek. The remaining integrity of the pile material where moisture exposure is frequent and intermittent is likely significantly reduced.
- At Bent 12 and 13, the wood diagonal sway brace and horizontal brace have been replaced with a steel vehicle guardrail section at the span 12 side, where an adjacent paved trail occurs. The horizontal brace is not aligned with the opposite wood brace or the diagonal brace.
- At Bent 12 piles 2 and 3, the wood section has been fully cut approximately 1'-6" from the base, and no positive reattachment has been made.
- At Bent 13, the soil on the span 12 side has been excavated to allow a paved trail, and the piles are now retaining approximately 4ft of soil on the span 13 side.

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2.2 Seismic Evaluation - ASCE-41 Tier 1 Screening

Based on the collapse prevention performance level for a risk category II building, the Basic Configuration Checklist and the Wood Light Frame Checklist have been evaluated for the existing bridge condition. The following items have been found “Non-Compliant” with Checklist standards.

2.2.1 Basic Configuration Deficiencies

Load Path

At Trestle support bents 2 through 13 the structure does not contain a complete, well-defined lateral load path. Diagonal wood braces occur to stiffen the wood pole columns at each bent against lateral load, but the braces and their bolted connections are significantly deteriorated. Based on wood deterioration and fire damage, wood material in various elements and bolted connections at some braces must be considered insufficient.

Weak Story / Soft Story / Vertical Irregularities

At bents 2, 3, 4, 5, 6 ,7, 8, 9, and 11, the diagonal braces terminate above the foundation with no horizontal tie or with several feet clear between the bottom of braces and existing grade. No lateral system exists between the existing braces and grade level. At Bent 12, an existing wood pole was previously cut and repaired with nailing insufficient to serve as a seismic connection.

2.2.2 Wood Light Frame Deficiencies

Redundancy

The Lateral load path relies on diagonal braces that occur on only one side of bent framing. The loss of any single brace could eliminate the existing lateral load path. The bolted connections consist of a single bolt at each column.

Girder-Column Connection

The Hardware that occurs between girders, cap beams, and piles is insufficient to provide positive attachment. An example of typical connection detailing for this style of construction is shown below. The presence of highlighted fasteners was unable to be confirmed and their remaining capacity is likely negligible due to wood deterioration.

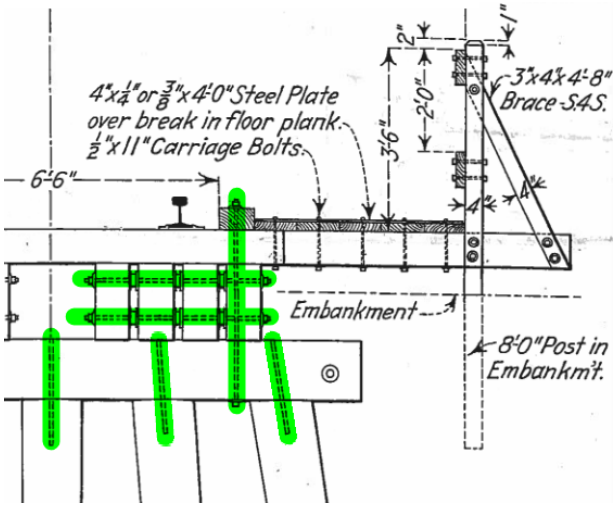


Figure 3 Example Connection from Bridge to Piles

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2.3 Tier 2 Deficiency-Based Analysis

The deficiencies noted in the tier 1 screening all relate to the overall deficiency of an insufficient load path, which arises from the deterioration of wood elements and the trestle bridge layout. There is no simplified procedure for a tier 2 analysis of this deficiency, so an analysis has been performed using the guidelines for a tier 3 Systematic Evaluation. Modern light timber construction relies on ductile yielding in connections, with layouts sufficient to provide redundant resisting elements. Elements prone to sudden failure such as beams and columns are typically designed to resist seismic forces without failure.

This trestle bridge layout relies on non-redundant wood elements for horizontal stiffening, and connections typically rely on a single bolt. If wood elements fail during a seismic event or if bolts fail by splitting at wood elements, the seismic load will redistribute to the remaining intact elements and cause progressive failure of those elements until the structure has lost all load resisting capacity. In order to achieve either LS or CP performance, the structure must avoid this progressive collapse failure mode. Progressive collapse can be avoided by ensuring that a structure with failing elements maintains residual capacity in a redundant load resisting system, or that non-redundant failing elements fail in a manner that provides some residual capacity such as the yielding of steel connection bolts.

Analysis has been carried out using a RISA-3D model to represent a typical trestle support at one of the taller bents, such as bent 8 or 9. Based on the approximately equal loads supported at each bent, these conditions can be taken as indicative of other bents by looking exclusively at upper levels of the trestle to represent shorter bents.



Figure 4 Typical Analysis Bent



Figure 5 Bent with Fire Damage

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2.3.1 Analysis 1 - No Retrofit / Existing Condition

Based on BSE1E loads and member sizes including a 1” char thickness on all faces, wood elements can be expected to fail in numerous locations, indicated in red on the diagram below. At all bolted connections, bolt capacities are governed by steel yielding modes, but demands from seismic loads far exceed these capacities in addition to the bolt capacities based on wood splitting.

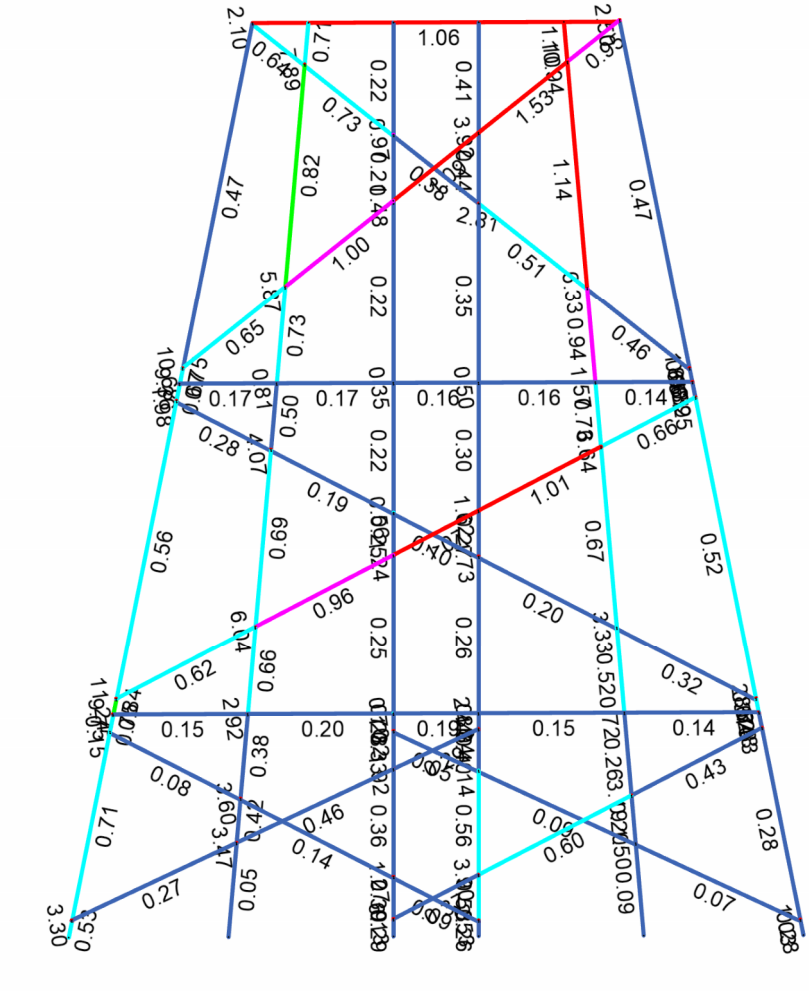


Figure 6 Existing Member Stresses at BSE1E

Based on the failures of non-redundant load resisting elements, the structure would be expected to deflect without resistance horizontally due to a BSE1E Seismic Hazard. Without a positive attachment between cap beams and top of piles, there is a significant risk that the horizontal deflection would shift the piles away from cap beams enough to cause a collapse of vertical framing.

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2.3.2 Analysis 2 - Retrofit for Collapse Prevention / Self-Sustained

Assuming that existing elements that are damaged can be replaced with similar or larger components, the trestle has been analyzed for BSE1E loads to determine a layout that results in failures governed by bolt yielding, with no failures in wood elements. Approximate required sizes of retrofit elements are indicated in the diagram below.

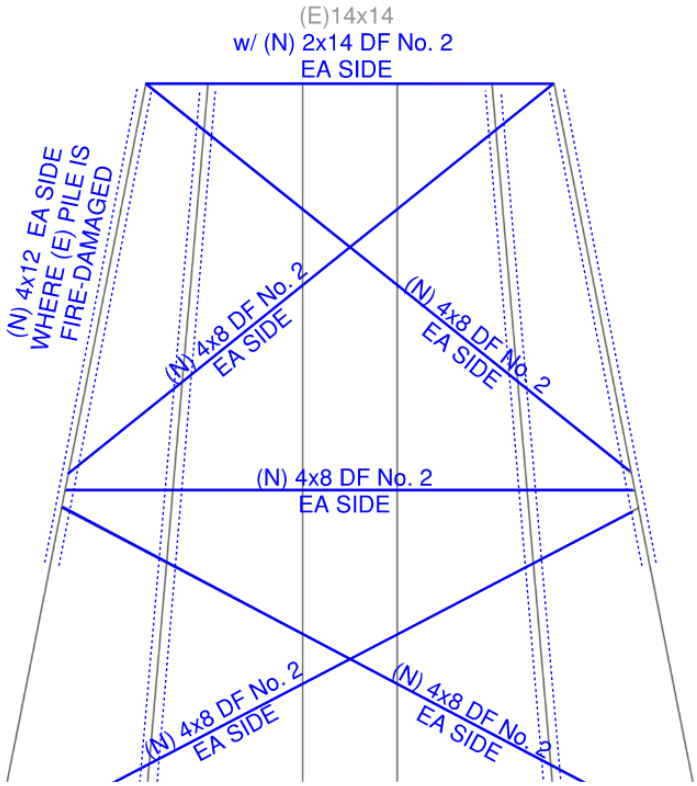


Figure 7 Collapse Prevention Retrofit Sketch

Where significant fire damage at piles occurs, the capacity of piles is insufficient and additional wood members will be required with through-bolting attachment along the length. At grade, the embedment of existing piles is unknown, and significant overturning loads must be resisted by the outermost piles. A Retrofit in-kind may be able to use existing piles in place if embedment depth can be verified in field or supplemented with the installation of helical pier tiedowns.

At connections, braces and pile caps are not adequate as noted in section 2.3.1. The bolted connections at braces and pile to cap beam attachments will require significant strengthening, as indicated in figures 8 through 11.

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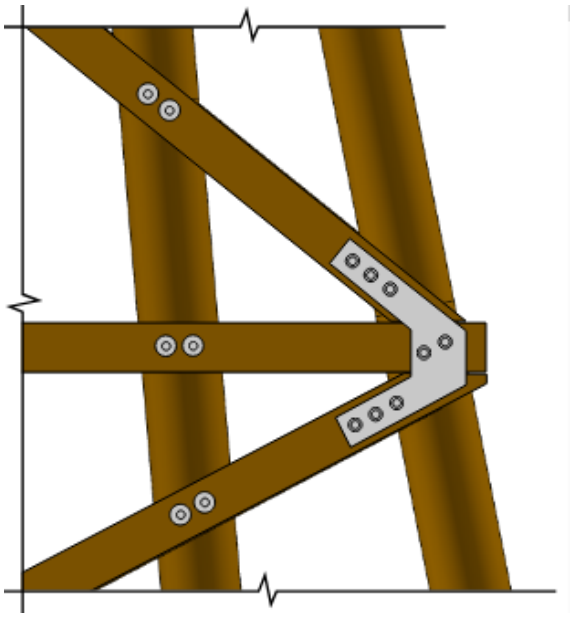


Figure 8 Schematic Brace Strengthening

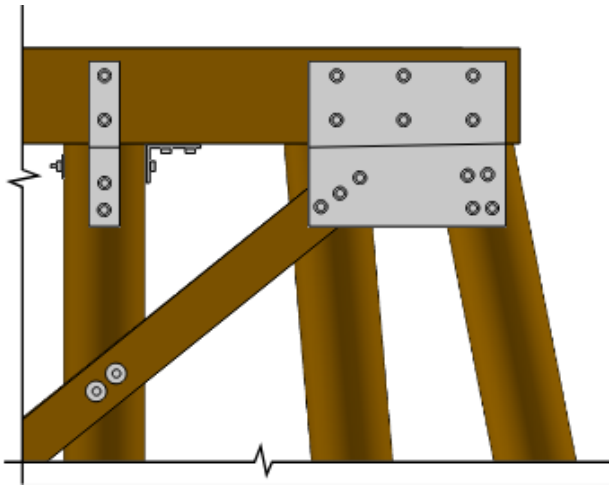


Figure 9 Schematic Cap Beam Attachment



Figure 10 Existing Brace Condition



Figure 11 Existing Cap Beam Condition

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2.3.3 Analysis 3 – Retrofit for Multi-purpose Trail

For LS performance, a retrofit similar to that outlined in section 2.3.2 would need to resist loads caused by BSE2E with a margin of safety remaining at elements resisting vertical loads due to occupancy.

Based on the risk of progressive collapse at this structural layout, strength for resisting lateral loads must be limited by bolted connections failing in yielding of the steel. Based on the more stringent requirements for loads and capacities at LS level, the schematic sizes noted in section 2.3.2 would need to increase by approximately 75%. The bolted connection retrofit would require a similar increase in capacity, and given available geometry a connection using bolts, timber, and steel side plates may not be feasible. Considering the required member sizing and connection constraints, a retrofit “in-kind” for pedestrian use is likely not feasible.

The retrofit would more likely consist of supplementary tube steel frames and concrete foundations, installed either adjacent to the existing trestles or spaced between where clearances may be required. These frames would support horizontal bracing at the bridge deck to resist lateral load, allowing the existing bridge elements to be repaired as required for the purpose of supporting gravity loads only. The steel frames could be spaced at every bent or intermittently, with a profile that matches the existing condition as closely as possible. An example schematic layout is indicated in figure 12, showing the south half of the bridge with frames at every other span.

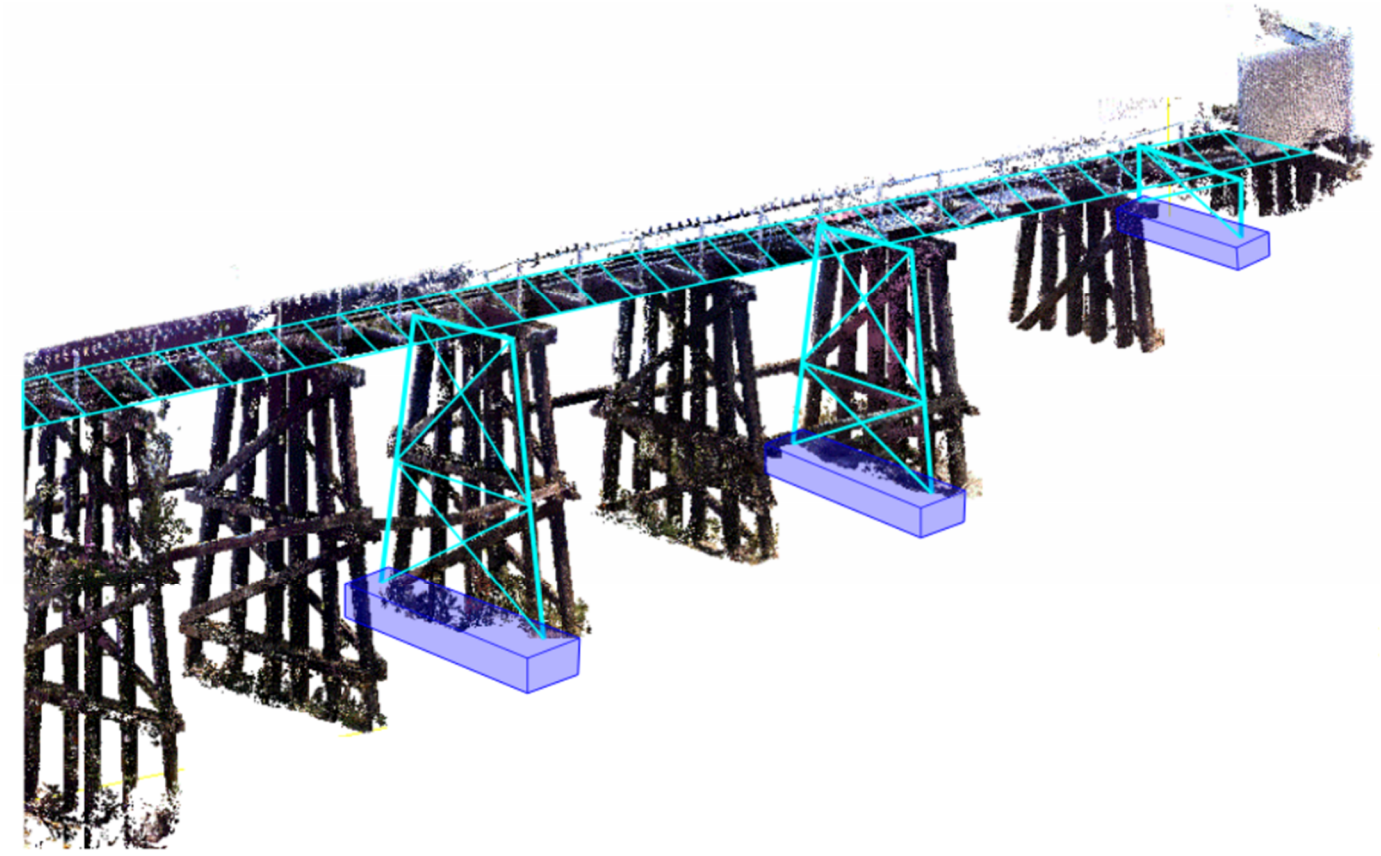


Figure 12 Schematic Retrofit Frame Layout

The existing bridge girders are adequate for expected loads from pedestrian use and a single maintenance vehicle, which is consistent with the structure’s original service as a railway bridge. The existing bridge deck would need to be replaced where deteriorated, or replaced with an alternative support system for the desired bridge deck. Repurposing of the bridge for pedestrian use would require additional retrofit for non-structural requirements, such as guard rails, accessibility, decking surface, and adequate signage. These new elements would need to meet all current building code requirements for new construction.

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2.4 Hydraulic Analysis

The results of the hydraulic analysis, including the scour analysis, are included in Appendix C. For the peak 100-year flow considered in the analysis, the maximum depth of flow at the upstream side of the bridge was calculated to be 33.9 feet and the average flow velocity at that location was calculated to be 2.2 feet/second.

The scour analysis considered general scour due to contraction and bends in the channel. The calculated maximum general scour at the bridge was 5.8 feet.

The scour analysis also considered local scour at the piers and abutments. The calculated pier scour depth at the bents ranged from 1.9 to 3.2 feet. The calculated abutment scour depth was 2.6 feet at Abutment 1 on the left bank (looking downstream) and 9.1 feet at Abutment 14 on the right bank (looking downstream).

Depending on the preferred retrofit option, a hydrodynamic structural and debris impact analysis would need to be performed during the final design / Contract Document phase to determine the existing trestle's structural capacity versus hydraulic loading.

3. Conclusions and Recommendations

The existing trestle bridge structure is adequate to support its own weight, and to resist short-term wind loading. It is not currently used for pedestrian traffic, but it was historically used for rail traffic and still has significant capacity to resist vertical loads; however, trespassers have dismantled City installed perimeter fencing to use the bridge to illegally cross Coyote Creek. Without repair and replacement of deteriorated elements, the vertical capacity of the bridge will eventually deteriorate to zero and portions of the structure will be at risk of partial collapse. Therefore, leaving the structure in its existing condition is not recommended.

Based on the structural evaluation and seismic analysis outlined in section 2.3.1, the two options for retrofit of the existing trestle are as follows:

- Analysis 2: Retrofit for Collapse Prevention
    - Recommended option if historical preservation of the structure's character is desired.
    - Designed for a high probability, low severity seismic event, quantifiable as a seismic event with a 20% risk of occurrence within 50 years.
    - Retrofit of bridge supports using similar material and strengthened connections.
    - Pedestrian/multi-purpose trail access will be prohibited.
- Analysis 3: Retrofit for Multi-purpose Trail Use
    - Recommended option if re-use of trestle is desired for a multi-purpose trail.
    - Designed for a seismic event, quantifiable with a 5% chance of occurrence in 50 years.
    - Would involve supplementary support system such as steel braced frames to support lateral loads.
    - This alternative would be more costly than the retrofit resulting from Analysis Option 2.

This document is in draft form. The contents, including any opinions, conclusions or recommendations contained in, or which may be implied from, this draft document must not be relied upon. GHD reserves the right, at any time, without notice, to modify or retract any part or all of the draft document. To the maximum extent permitted by law, GHD disclaims any responsibility or liability arising from or in connection with this draft document.

Appendices

Appendix A

Wood Assessment by Porter & Associates

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August 17, 2022

COYOTE CREEK TRESTLE: SUMMARY FINDINGS AND RECOMMENDATIONS

1. The wood species of the structural members in the trestle (piles, pile caps, stringers, bracing, abutment planks, and ties forming the deck is Douglas-fir *(Pseudotsuga menziesii),* with the exception of one diagonal brace from the south abutment, which is redwood *(Sequoia sempervirens),* sample #19. It is not possible from a wood assessment to determine if the redwood plank is original or a replacement.

2. The structural grade of the timber piles is given in the *National Design Specification for Wood Construction* for Pacific Coast Douglas Fir piles, based on grading and dimensions criteria defined in ASTM D25 *Standard Specification for Round Timber Piles* and ASTM D2899 *Practice for Establishing Allowable Stresses for Round Timber Piles*. The piles were pressure treated with creosote to resist biological deterioration and increase service life. The creosote treatment has largely been effective.

3. The structural grade of the pile caps is Select Structural Douglas Fir -Larch Posts and Timbers, based on criteria established in ASTM D245 and the *Western Lumber Grading Rules 2017* promulgated by the Western Wood Products Association (WWPA). The pile caps were pressure treated with creosote. The creosote treatment has largely been effective in protecting the pile caps against biodeterioration.

4. The structural grade of the timber stringers is Select Structural Douglas Fir - Larch Beams and Stringers, based on criteria established in ASTM D245 and the WWPA grading rules. The stringers were pressure treated with creosote. The creosote treatment has largely been effective in protecting the stringers against biodeterioration.

5. The structural grade of the sway braces varies from Select Structural Douglas Fir – Larch Visually Graded Dimension Lumber to No. 2 Douglas Fir – Larch Visually Graded Dimension Lumber.

6. The structural grade of the longitudinal braces varies from Select Structural Douglas Fir – Larch Visually Graded Dimension Lumber to No. 2 Douglas Fir – Larch Visually Graded Dimension Lumber.

7. The structural grade of the ties is not relevant to this project, only the condition, as the ties will likely be covered with a pan and continuous wearing surface, such as a concrete deck.

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8. The condition of the timber piles is, generally, good-to-excellent, meaning that some minor deterioration due to wood decay fungi or insects is present in the pile, with some exceptions (namely Bent 1, Pile 1; Bent 3, Pile 4; Bent 5, Pile 5; Bent 12, Pile 2; Bent 13, all piles)

9. The condition of the pile caps is, generally, poor-to-excellent, with deterioration due to wood decay fungi resulting in a) reduction in connection capacity where a brace is attached at the west end of the cap at Bent 7, b) section losses of up to 35% at Bents 4 and 12, and c) severe deterioration of caps at Bents 5 and 10; these caps are in poor condition and are not able to perform their intended function.

10. The condition of the stringers is, generally, good-to-excellent, meaning that some minor deterioration due to wood decay fungi or insects is present, except in span 4, where stringer 9 has section loss ≤50%.

11. The condition of the sway braces is, generally, poor-to-fair, meaning that some severe deterioration due to wood decay fungi, insects, fire damage, or physical erosion of the ends of the braces has resulted in losses in connection capacity. Members in fair-to-poor conditions will not be able to perform their intended function for an extended period of time.

12. The condition of the ties making up the decking is, generally, good-to-excellent, meaning that some minor deterioration due to wood decay fungi or insects is present in the ties with few exceptions. Because of the close spacing of the ties, ties with minor deterioration can either be replaced or remain in service so long as remedial preservative treatment is applied and the deck is covered with a continuous wearing service.

13. The condition of the guardrails is, generally, poor-to-good, meaning that some severe deterioration due to wood decay fungi, insects, fire damage, and/or weathering has occurred in many of the members. Because exposure to the elements is expected to continue, guardrail members in poor-to-fair conditions will not be able to perform their intended function for decades.

RECOMMENDATIONS

1. The Coyote Creek Trestle is built with, generally, high-quality piles and timbers. With some repairs and limited replacement of members, the trestle can provide service as a pedestrian (rather than a railroad) bridge for decades with minimal maintenance. This, of course, is subject to the structural analysis conducted by GHD comparing load-carrying capacity to anticipated loads.

2. The timber piles can remain in service. Repairs to the base of deteriorated piles can be made using a timber splice to the existing pile or other means to maintain the visual and structural integrity of the trestle.

3. Most of the pile caps can remain in service. At the very least, replacement of the two severely deteriorated caps at Bents 5 and 10 should be considered by the structural

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engineer to provide adequate bearing for the stringers they support. Replacement is relatively straightforward, and the structural engineer may want to consider replacement of the caps at Bents 4 and 12 as well, as a way to reduce maintenance of these caps in the decades following rehabilitation of the trestle.

4. The stringers can remain in service. Repairs or replacement of deteriorated stringers should be considered by the structural engineer to provide adequate bearing for the deck, recognizing that there is considerable redundancy provided by the 10 stringers spanning between each bent.

5. The sway braces and longitudinal braces are of lower quality lumber and were not pressure treated with creosote. Due to their limited remaining service life, they should be replaced.

6. Replace fire-damaged dimension lumber (nominal thickness of 4 inches or less). These typically include the sway braces and abutment planking, which are recommended for replacement based on reduced connection capacity and limited remaining service life.

7. Remove char from timber piles, pile caps, and stringers to expose the remaining sound timber. The remaining cross section can be considered the size of the un-charred member dimensions for structural analysis purposes. Char that developed into pillowing is typically ½ to ¾ inches thick on each face exposed to fire.

8. Piles and caps used in the construction of abutments are typically in good condition and require no repair. Most of the four-inch planking used in the construction of retaining walls and bracing is in contact with the soil, and reductions in cross section are common. The structural engineer can consider wholesale replacement of planking; alternatively, soils behind the retaining walls can be excavated and a new layer of pressure treated planking can be added behind the existing walls, preserving most of the historic material.

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Appendix B

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION REPORT

FIVE WOUNDS TRAIL – TRESTLE EVALUATION

SAN JOSE, CALIFORNIA

Prepared For:

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October 18, 2022

PARIKH Project No. 2022-104-GEO

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GEOTECHNICAL INVESTIGATION REPORT
FIVE WOUNDS TRAIL - TRESTLE EVALUATION
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This "Geotechnical Investigation Report" (REPORT) presents the results of geotechnical engineering investigation for the proposed "Five Wounds Trail – Trestle Evaluation" in San Jose, California, hereinafter referred to as "PROJECT". The work was performed in accordance with the scope of work outlined in our proposal to GHD, Inc. (Designer).

2.0 PROJECT UNDERSTANDING

The City of San Jose (CITY) has requested consulting services to provide an immediate structural assessment of an old Railroad trestle across Coyote Creek that has recently caught on fire and is in the danger of collapsing. Parikh Consultants, Inc. (PARIKH) is to provide a geotechnical investigation to support the structural design by the designer.

3.0 SCOPE OF WORK

PARIKH has performed geotechnical investigation for the evaluation of the pile foundation of the existing bridge structure. The intent is to prepare a 'draft report' that can provide enough data for designer to evaluate the capacity of the existing pile foundation. The as-built pile lengths are unknown according to designer. The 'draft report' may need to be updated with the existing pile length and design conditions as required at a later date. This scope excludes that task.

The scope of work performed for this investigation included a review of the readily available soils and geologic literature pertaining to the project site, site reconnaissance, obtaining representative samples and logging soil materials encountered in exploratory borings, laboratory testing of the representative soil samples, performing engineering analyses based on the field and laboratory data, and preparation of this geotechnical investigation report.

4.0 FIELD INVESTIGATION AND TESTING PROGRAM

4.1 Field Exploration Performed

Based on the preliminary plan and discussions with the designer, two soil borings (Borings B-1 and B-2) were drilled at the project site to verify the subsurface soil materials. Two bulk samples were collected by hand tools from the creek bed for gradation for scour analyses.

Hammer test was included in the scope to get the information of a rough estimate for the strength

of the surface concrete at the abutments. However, according to the field engineer of PARIKH, the existing bridge structure is a wooden structure with no concrete for the hammer test. The following is the photo taken at the existing abutments during the field exploration:



Approximate locations of these borings are presented in the "Boring Location Map" (Plate No. 3). Details of the field exploration are included in Appendix A. The depth and approximate ground elevation of these borings are summarized in the table below.

TABLE 1-SUMMARY OF FIELD EXPLORATIONS

Boring No.	Boring/Hole Depth (feet)	Approx. Ground Elev. (feet)	Hammer Energy Ratio (%)	Completion Date
B-1	61.5	104.0	54	3/28/2022
B-2	61.5	106.0	61	3/29/2022

The descriptions of the soil materials encountered in the field exploration and relevant boring information are presented on the LOTB in Appendix A.

4.2 Field Testing

Soil samples were obtained from the borings during drilling at various depths by driving a 2.5 inches

Inside Diameter (I. D.) Modified California Sampler or a 1.375 inches I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140 pounds hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the LOTB, Appendix A. (When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be converted to equivalent Standard Penetration Test sampler by multiplying a factor of 0.65).

Pocket penetration tests were also performed on clay samples to evaluate their consistency.

4.3 Details of Field Exploration

The soil borings were drilled by Exploration GeoServices Inc. (EGS) using 7.5-inch diameter hollow-stem auger. The hammer efficiency was 54% for the B61 drill rig and 61% for the B53B drill rig per the driller. All borings were drilled under the technical supervision of our geologist/engineer, who classified and continuously logged the soils encountered during drilling and supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were classified in the field according to the Caltrans "Soil and Rock Logging, Classification and Presentation Manual" (2010 Edition) and then transported to our laboratory for further evaluation and testing. Upon completion of drilling, the boreholes were backfilled with cement grout.

The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

It should be noted that the descriptions of the soil encountered, and relevant boring information presented on the LOTB depict subsurface conditions only at the locations indicated on the "Boring Location Map" and on the particular date noted on the LOTB. Because of the variability from place to place within soil in general, subsurface conditions at other locations may differ from conditions occurring at the boring locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

5.0 LABORATORY TESTING PROGRAM

The following laboratory tests were performed on selected soil samples collected during field



exploration to evaluate the physical and engineering properties of the subsurface soils at the project site to support the recommendations:

a) Laboratory determination of Moisture-Density.
b) Atterberg Limits.
c) Grain Size Classifications.
d) Unconfined Compression Test.
e) Corrosion Test.

The laboratory test methods and laboratory test results for moisture content, dry unit weight, unconfined compression, plasticity index and grain size classification of the soil samples are summarized in the table in Appendix B. The laboratory test methods and laboratory test results are presented in Appendix B.

6.0 SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

6.1 Site Geology

The project is located within the Coast Ranges Geomorphic Province on the plain bordering southern San Francisco Bay. Geologic unit extents and descriptions have been derived from Witter et al., (2006), "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, USGS OF-06-1037".

Project site geology is shown in Plate No. 4 Geologic Map. Surficial geological units relevant to the project are described below:

- Artificial levee fill (alf), is mapped to border both banks of Guadalupe Creek at the Project site. The Guadalupe River Trail runs along the eastern levee of Guadalupe creek. The origin and composition of this material was not revealed during this investigation.
- Latest Holocene stream terraces (Qhty). This unit is mapped to underlie the majority of the project and is described as being composed of sand, gravel, silt, and minor clay, is moderately to well sorted, and is moderately to well bedded.

Depth to bedrock was not revealed during this investigation, however a well drilled about 1,800 feet south of the project site (615 Moreland Way; Well # 0915322) did not intersect bedrock to a maximum depth of about 930 feet below surface.

6.2 Surface Conditions

The area surrounding the project site is relatively flat and slopes gently towards Coyote Creek. The Project site slopes gently to the north-west towards San Francisco Bay. Coyote Creek is the main topographic and hydrological feature of the site. Coyote Creek forms a bend directly at the site which turns from east-west to north-south. Areas adjacent to the Project site appear to flow to Coyote Creek. The existing ground elevation is estimated to be between Elev. 104 feet and 106 feet based on Google Earth. There is water flowing in the creek during the field exploration. Based on the photo below, it appears that the water of the creek is approximately 20 feet below the ground surface of the creek bank.



The historic Western Pacific Railroad passed directly through the site and appears to have been decommissioned prior to about 1980.

Geologic and elevation data relative to the site does not indicate the presence of geologic hazards such as landslides, slope failure, rockfalls, or debris flows.

6.3 Subsurface Soil Conditions

Based on the available boring data, the subsurface soil conditions within the boring depth generally consist of stiff to hard lean clay, with interbedded layers or intermittent pocket of stiff to very stiff silt. Loose poorly graded gravel and medium dense silty sand were encountered in the top 18 feet of Boring B2.

The descriptions of the subsurface soil materials encountered in the exploratory borings are presented in Table 2 below.

TABLE 2-SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	Boring Depth (ft)	Subsurface Soil Descriptions
B-1	61.5	Approximately 23 feet of very stiff to hard lean clay, underlain by approximately 20 feet of stiff to very stiff lean clay/silt, underlain by approximately 10 feet of soft to very stiff lean clay, underlain by approximately 9 feet of very stiff silt/lean clay to the bottom of boring,
B-2	61.5	Approximately 12 feet of very loose poorly graded gravel with sand, underlain by approximately 6 feet of medium dense silty sand with gravel, underlain by approximately 15 feet of very stiff to hard lean clay, underlain by approximately 5 feet of very stiff silt, underlain by approximately 20 feet of very stiff lean clay, underlain by approximately 4 feet of stiff to very stiff silt to the bottom of boring.

The subsurface soil condition for the evaluation of the existing foundation of the "Five Wounds Trail – Trestle" has been based on the interpretation of the geologic formations and available boring information from Borings B-1 and B-2.

It is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

6.4 Groundwater

Hollow-Stem Auger drilling was used to measure groundwater. Groundwater encountered in Parikh's borings from the current investigation is summarized in the table below

.

TABLE 3 - SUMMARY OF GROUNDWATER LEVEL

Boring No.	Ground Surface Elevation (ft)	Measured Groundwater		Date Measured
		Depth, feet	Elevation, feet	
B-1	104.0	29.0	75.0	3/28/2022
B-2	106.0	38.0	68.0	3/29/2022

*The elevations are approximate and based on NAVD88 datum.

Design groundwater elevation of 75 feet is assumed for the engineering analyses in this report only.

The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, variations in yearly rainfall, water elevations in the nearby Guadalupe River, surface and subsurface flows, ground surface run-off, and other environmental factors that may not be present at the time of the investigation.

It is imperative that the contractor is aware of such potential changes in groundwater level and should not solely rely on such transient measurements from the boring data. It may be prudent to make conservative assumptions in the construction program to account for the potential variation and/or to verify the groundwater level prior to the construction.

7.0 SCOUR EVALUATION

Bulk sample was collected by hand tools from the creek bed for grain size distribution analyses for scour analyses. The grain size distribution curve is included in Appendix B.

8.0 CORROSION EVALUATION

The corrosion investigation for this project was also performed on the selected samples from the borings drilled in 2021 in general accordance with the provisions of California Test Methods 417, 422 and 643. A summary of the corrosion test results is presented in the following table.

TABLE 4-SUMMARY OF CORROSION TEST RESULTS

Boring No.	Depth (ft)	Min. Resistivity (ohm-cm)	pH	Sulfate (ppm)	Chloride (ppm)
B-1	31	2,170	7.74	46.0	15.1
B-2	41	1,370	7.52	71.5	13.2

The Caltrans Corrosion Guidelines May 2021 (Version 3.2) states that soil and water that have a minimum resistivity equals to or less than 1100 ohm-cm, the sample are required to be tested by a certified lab for chlorides and sulfates per CT 417 and CT 422. Caltrans currently considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water

samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration is 1500 ppm or greater, or the pH is 5.5 or less. Based on the corrosion test results, the project site is considered non-corrosive per Caltrans corrosion design guidelines.

9.0 SEISMIC RECOMMENDATIONS

9.1 Seismic Sources

The regional seismic context is an important consideration because forces that affect the project area are regional in nature: that is, they are generated off-site, outside the immediate area, or outside Santa Clara County. However, the effects of these forces must be accommodated within the limits of the project in compliance with regulations and guidelines established by the State and County.

Santa Clara County and the rest of the Bay Area are in one of the most active seismic regions in the United States. Many active faults exist in the regional area and can produce earthquakes that may cause strong ground shaking at the project site. Each year, low- and moderate-magnitude earthquakes occurring within or near the Bay Area are felt by residents. Since the mid-nineteenth century, hundreds of earthquakes have been felt in Santa Clara County. The “Loma Prieta Earthquake” of October 17, 1989, originated within the San Andreas Fault Zone and caused severe damage throughout much of the Bay Area. The major fault zones of the San Andreas Fault System (including the Hayward and Calaveras faults) have been the source of other earthquakes and are expected to be a source of future earthquakes. A fault map showing the active faults within about sixty miles of the project site is shown in Plate No. 5**.**

The major faults in the vicinity of the project site and their distances to the project site, fault types and the maximum credible earthquake magnitudes associated with each fault are summarized in the Table 5. These maximum credible earthquake magnitudes represent the largest earthquakes that could occur on the given fault based on the current understanding of the regional tectonic structure.

TABLE 5-EARTHQUAKE DATA

Fault	Age	Maximum Magnitude M_{Max}	Fault Type	Approx. Distance Rx (miles)
Hayward, South-east Extension	latest	6.7	strike slip	4.6
Hayward, Southern	historic	7.3	strike slip	4.9
Calaveras, Central	latest	6.9	strike slip	8.0
Calaveras, Northern	latest	6.9	strike slip	9.0
Monte Vista-Shannon	latest	6.4	reverse	9.0
San Andreas, Peninsula	latest	8.0	strike slip	12.6
San Andreas, Santa Cruz Mountains	historic	8.0	strike slip	12.6
Sargent	latest Quaternar	7.0	strike slip	18.2

Notes: Magnitudes are derived from UCERF 3 (Field et al. 2013); approximate fault distances were derived from the USGS Fault and Fold Database (2006); Rx = Horizontal distance to the fault trace or surface projection of the rupture plane.

Based on available geological and seismic data, the potential of the project site to experience strong ground shaking exists.

9.2 Seismic Design Criteria

The recommended design response spectrum for the proposed "River Oaks Stormwater Capture Project" was determined by using the Caltrans ARS Online tool (V3.0.2). which is consistent with the Caltrans SDC V2.0.

For SDC 2.0, the Design Spectrum is based on the USGS 975-year uniform hazard spectrum only. Effective December 1, 2019, the USGS hazard spectrum is based on the 2014 National Hazard Map per the memorandum from the State Bridge Engineer. The updated Design Spectrum continues the use of near-fault adjustment factors and basin amplification factors. The only change to these factors is the use of the Campbell-Bozorgnia (2014) and Chiou-Youngs (2014) basin amplification factors, updated from their 2008 models.

The development of the design ARS curve is based on several parameters, including site location (longitude/latitude), average shear wave velocity for the top 30m/100 feet (Vs30m), and other site parameters, such as fault characteristics and site-to-fault distances.

Average shear wave velocities for the top 100 feet of soils at the site were estimated by using established correlations and procedure provided in Caltrans Methodology for Developing Design Response Spectrum for use in Seismic Design Recommendations (2012). Shear wave velocity calculations are attached in Appendix C.

Based on the subsurface data, the site is classified as "Class S2 Soil" per Caltrans SDC, V2.0. The site location and the relevant parameters are summarized in the table below, and the recommended design curve is presented in Appendix C.

TABLE 6 – RECOMMENDED GROUND MOTION PARAMETER FOR GEOTECHNICAL DESIGN

Site Parameter			Design Ground Motion Parameter (Return Period = 975 years)		
Locations		Shear-Wave Velocity V_{s30},m/sec	Horizontal Peak Ground Acceleration $(\mathbf{HPGA})^{(1)}$, g	Mean Earthquake M, Moment Magnitude	Mean Site-to-Fault/ Rupture Surface $\mathbf{Distance}^{(2)}$ Rrup, (mile)
Latitudes degrees	Longitude, degrees				
37.4020	-121.9461	220	0.72	6.86	8.2

1. An adjustment factor for the near-fault effect was applied to the calculated spectral acceleration values. The increase of 20% to the spectral acceleration values corresponds to periods longer than 1 second and linearly tapes to zero at a period of 0.5 seconds.
2. No adjustment was needed for basin effect.

9.3 Seismic Hazard

Primary seismic hazards include ground shaking and surface fault rupture. Secondary seismic effects resulting from soil responses to ground shaking includes liquefaction. These hazards may cause deformation of man-made structures. These hazards are discussed in the following paragraphs.

Primary Seismic Hazards

9.3.1 Seismic Ground Shaking

A peak ground acceleration of 0.58g was calculated for the project site using Caltrans ARS Online tool (version 3.0.2).

9.3.2 Surface Fault Rupture

A review of the Milpitas Alquist-Priolo Earthquake Fault Zone Map shows the project is not located within an Alquist-Priolo Earthquake Fault Zone. The USGS Quaternary Fault and Fold Database shows that the project is not located within 1,000 feet of an unzoned fault that is Holocene/Latest Pleistocene (15,000 years) or younger in age. In assessing seismic risks, we consider that surface fault rupture does not contribute to the seismic hazards at the site during the useful life of the project. The preceding statements do not preclude the existence of unknown active faults or make inferences on the potential for aseismic surface cracking.

9.3.3 Tsunami Risk

The Project is located about 6.1 miles from the edge of San Francisco Bay and greater than

about 45 miles from the Pacific Ocean coast; at an elevation of about 4 feet at its lowest and is not within the CGS Tsunami Inundation Area. Given the distance from the Pacific Ocean coast, the Project is not considered at risk of tsunami inundation.

Secondary Seismic Hazards

9.3.4 Liquefaction Susceptibility

The "Liquefaction Susceptibility Map" (Plate No. 5) was referred to in the evaluation of the liquefaction potential for the proposed earth retaining systems. This map was developed from Witter, R.C. et al. (2006). According to this map, the liquefaction susceptibility at the project site is classified as "High".

9.3.5 Liquefaction Potential

Based on the boring logs, the subsurface soil generally consists of mainly medium stiff lean clay, underlain by medium dense to dense and poorly graded sand with gravel, underlain by very dense silty sand. Groundwater was encountered within the reach of hollow stem auger during the field exploration in 2022 at the depths between 29 and 38 feet.

According to "MNO-12 *Soil Liquefaction during Earthquakes*, by I.M. Idriss and R.W. Boulanger, 2008", fine-grained soils can reasonably be expected to exhibit clay-like behavior if they have a Plasticity Index >7 in engineering practice. The clays and plastic silts will not liquefy as the cohesionless sand but may undergo "cyclic strength softening" during earthquake.

The results of the liquefaction analyses are summarized in the table below.

TABLE 7 - SUMMARY OF ESTIMATED POST-LIQUEFACTION SETTLEMENT

Boring No.	Estimated liquefiable Soil Depth (ft)	Approx. Thickness (ft)	Estimated liquefiable Soil Top Elev.(ft)	Estimated liquefiable Soil Bottom Elev.(ft)	$(N_1)_{60,CS}$	Estimated Post-liquefaction Settlement (inches)
B-2	57.5	4.0	48.5	44.5	21.2	0.64

Based on the results of the liquefaction analyses, it appears that:

a) The loose silty sand/poorly graded sand encountered in Boring B-2 appears to be isolated and discontinuous.

b) The potentially liquefiable soil materials is at the depth of 57.5 feet, which should be well

below the tip of the existing timer pile. The potentially liquefiable soil at the project site is generally covered by predominantly cohesive soils. The cohesive soil tends to serve as a "soil mat" and should reduce the potential impact of liquefaction.

The post-liquefaction settlement probably would be random and localized and could spread out over some distance/area. The potential of liquefaction impact on the existing bridge structure is considered relatively insignificant. Liquefaction analyses are included in Appendix C.

Lateral Spreading

Liquefaction-induced spreading has been defined as the *"lateral displacement of large surficial blocks of soil as a result of liquefaction in a subsurface layer"*. Lateral spreading refers to the more moderate movements of gently sloping ground due to soil liquefaction. As described by Bartlett and Youd (1992a; 1992b), liquefaction-induced lateral spreading occurs on mild slopes of 0.3% to 5% underlain by loose sand and shallow water. The geologic conditions conducive to lateral spreading (gentle surface slope, shallow water table, and liquefiable cohesionless soils) are frequently found along streams and other waterfronts in recent alluvial or deltaic deposits, as well as in loosely packed, saturated, sandy fills.

The potential for lateral spreading does not exist at the project site since the saturated loose sand/silt appears to be localized in form of pockets/lenses and discontinuous.

10.0 GEOTECHNICAL EVALUATIONS

This report was prepared to evaluate the capacity of the existing pile foundation. Normal construction procedures were assumed throughout our analysis and represent one of the bases of evaluation presented herein. Our evaluation has been based upon the materials and subsurface soil conditions encountered in the soil borings at the project site.

The lateral earth pressures for the abutments/abutment retaining walls, the vertical capacity of the existing pile have been evaluated and discussed in the following sections.

10.1 Lateral Earth Pressures and Friction Coefficient for Cantilever Retaining Walls

The following is the photo of the existing railroad tie retaining wall taken during the field exploration.



The following were assumed in the evaluation of the lateral earth pressure for abutment wall/abutment retaining wall of the existing bridge structure:

a) Level ground behind the retaining wall.

b) No hydrostatic pore pressure buildup behind the wall.

c) Native soil was backfilled behind the retaining walls.

The following are the estimated "Applied Lateral Earth Pressures (Equivalent Fluid Pressures-EFP)".

Static Lateral Earth Pressure

(a) Native Soil Backfill γ =120 pcf, ϕ=30˚

(b) Active Pressure Coefficient ka = 0.333

(c) Recommended active pressure is 36 pcf EFP.

(d) Recommended at-rest pressure is 60 pcf EFP.

The calculations of lateral earth pressures are included in Appendix C.

10.2 Capacity of Existing Pile Foundation

According to the designer, the existing pile is the 15-inch timber pile. Since the pile length of the existing pile is unknown, the vertical pile capacity (based on AASHTO LRFD design criteria) and lateral pile capacity versus the pile depth has been developed for the 15-inch timber pile. The pile capacity analysis was performed to estimate the pile capacity based on the following:

(a) The estimation of the driven pile capacity of the timber pile was based on the Federal Highway Administration Method. The pile capacity was derived from frictional resistance along the pile shaft.

(b) Only axial capacity analysis in compression has been performed.

The output plots of the vertical pile capacity analyses are included in Appendix C.

11.0 INVESTIGATION LIMITATIONS

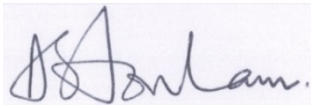
Our services consist of professional opinions and evaluations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater, or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge.

Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.



Alston Lam, P.E., G.E. 2605
Project Engineer

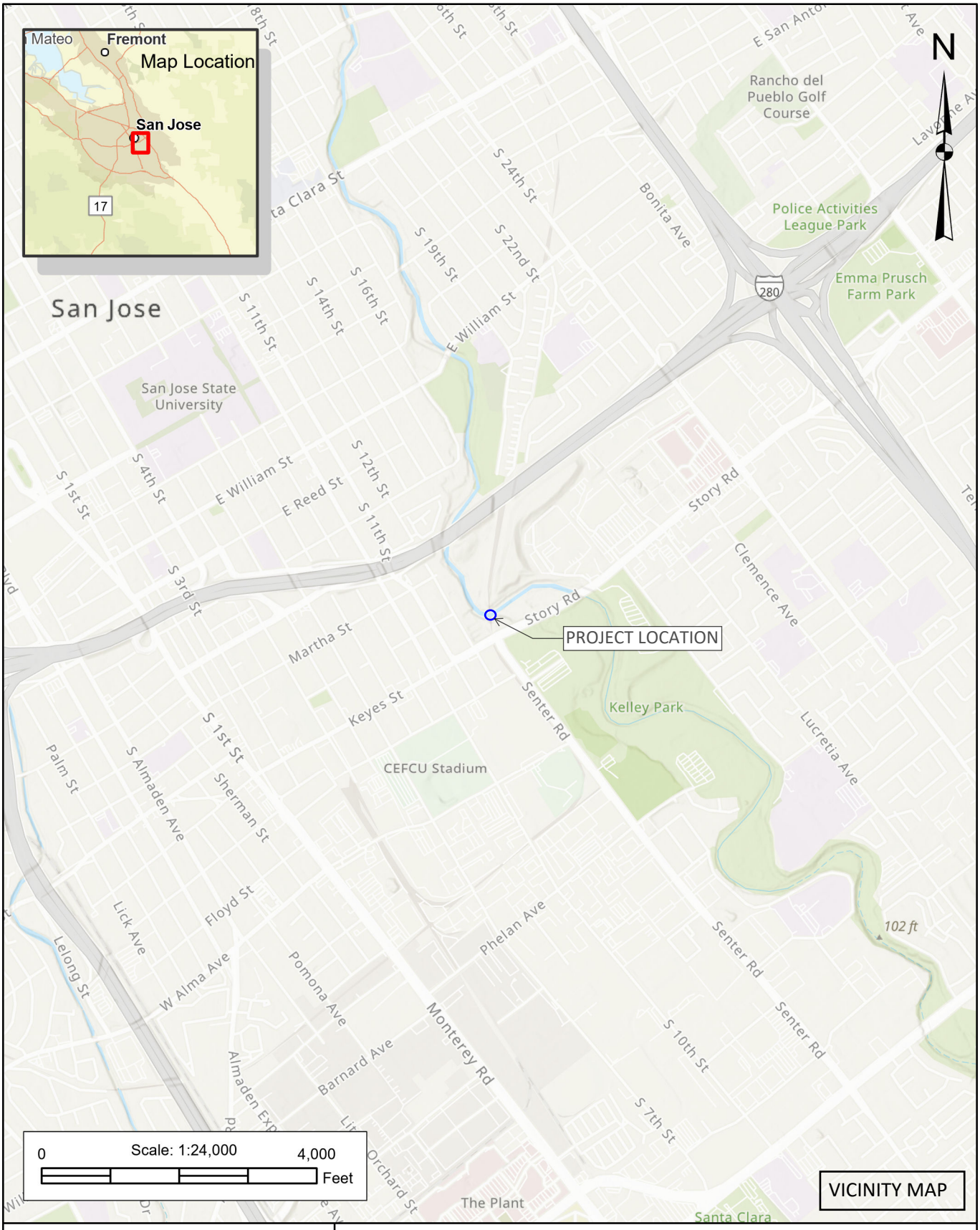


Y. David Wang, PhD., P.E. 52911
Senior Project Engineer



REFERENCES

1. California Department of Transportation, 2010, Soil and Rock Logging, Classification and Presentation Manual.
2. California Department of Transportation, 2012, Corrosion Guidelines, Version 3.2, May 2021.
3. California Department of Transportation, 2017, California Amendments to the AASHTO LRFD Bridge Design Specifications – Eight Edition.
4. California Department of Transportation, 2016, Caltrans ARS Online Tool, Version 3.0.2, http://arsonline.dot.ca.gov.
5. California Geological Survey. 1982, Official Maps of Earthquake Fault Zones: GIS files of Official Maps of Alquist-Priolo Earthquake Fault Zones - Milpitas, Sacramento, CA. Department of Conservation, California Geological Survey. http://maps.conservation.ca.gov/cgs/informationwarehouse/. Accessed April 28, 2021
6. State of California, 2020. Tsunami Hazard Area, Alameda and Santa Clara Counties; produced by the California Geological Survey and the California Governor's Office of Emergency Services; dated 2021.
7. United States Geological Survey and California Geological Survey, 2006. "Quaternary fault and fold database for the United States." Accessed April 28, 2019. http//earthquake.usgs.gov/hazards/qfaults/.
8. USGS NED ned19_n37x50_w122x00_ca_sanfrancisco_topobathy_2010 1/9 arc-second 2013 15 x 15-minute IMG
9. Witter, R.C., Knudsen, K.L, Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C. E., Brooks, S.K., and Gans, K.D., 2006. "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California." U.S. Geological Survey Open-File Report 06-1037. Accessed April 28, 2019. http://pubs.usgs.gov/of/2006/1037/.

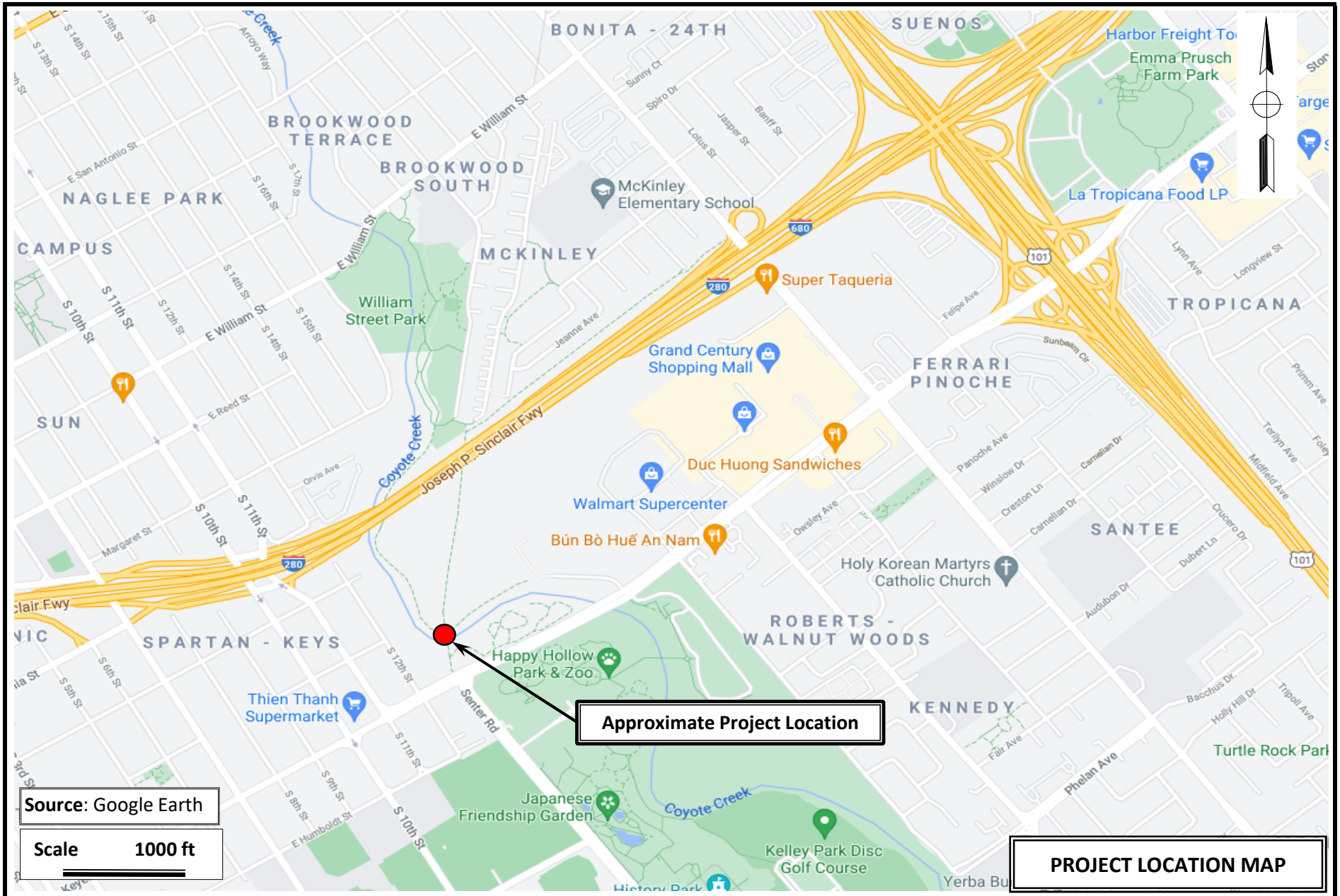


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SAN JOSE, CALIFORNIA

JOB NO.: 2022-104-GEO

PLATE NO.: 1



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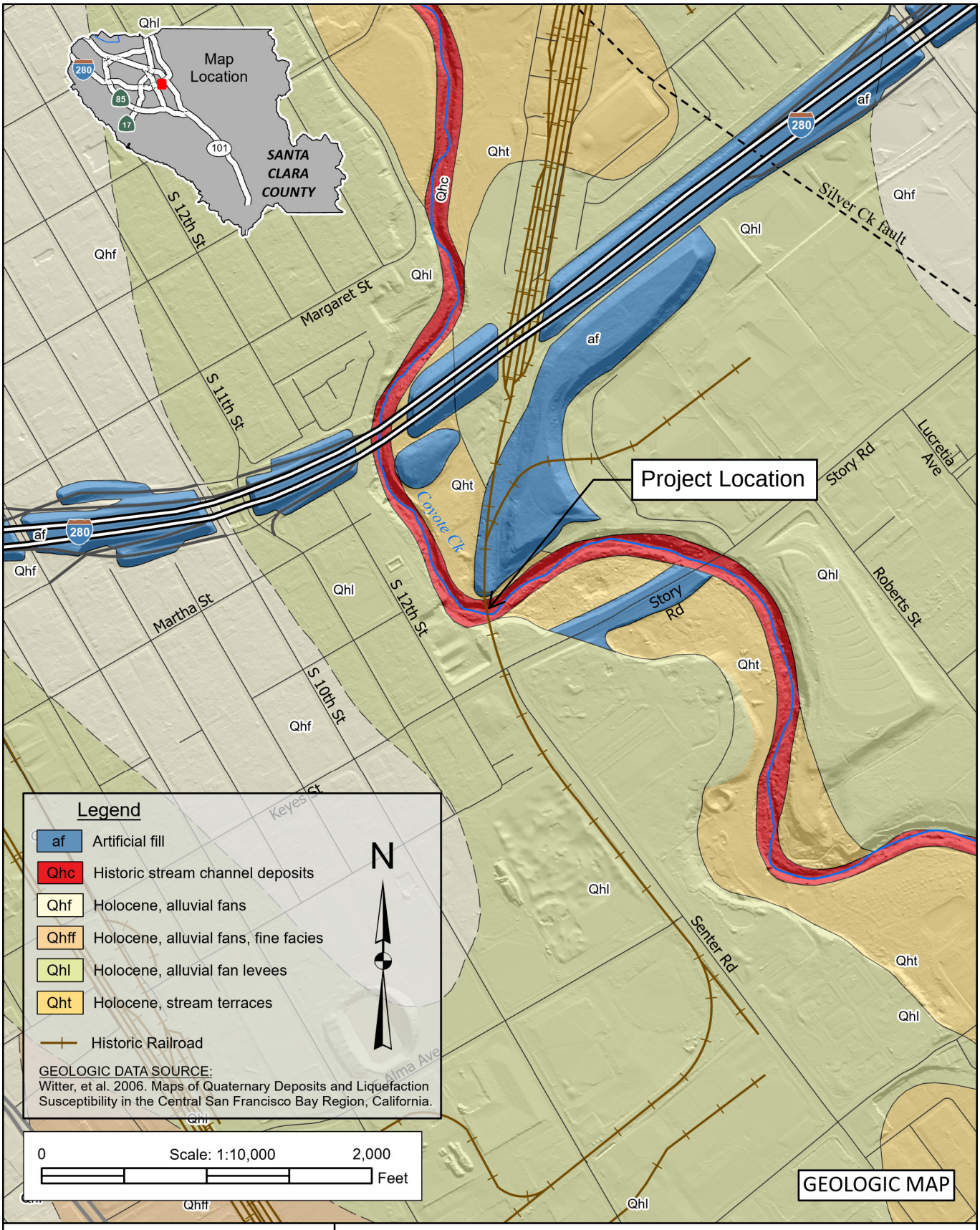
JOB NO.: 2022-104-GEO | **PLATE NO.: 2**



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SAN JOSE, CALIFORNIA

JOB NO.: 2022-104-GEO | **PLATE NO.: 3**



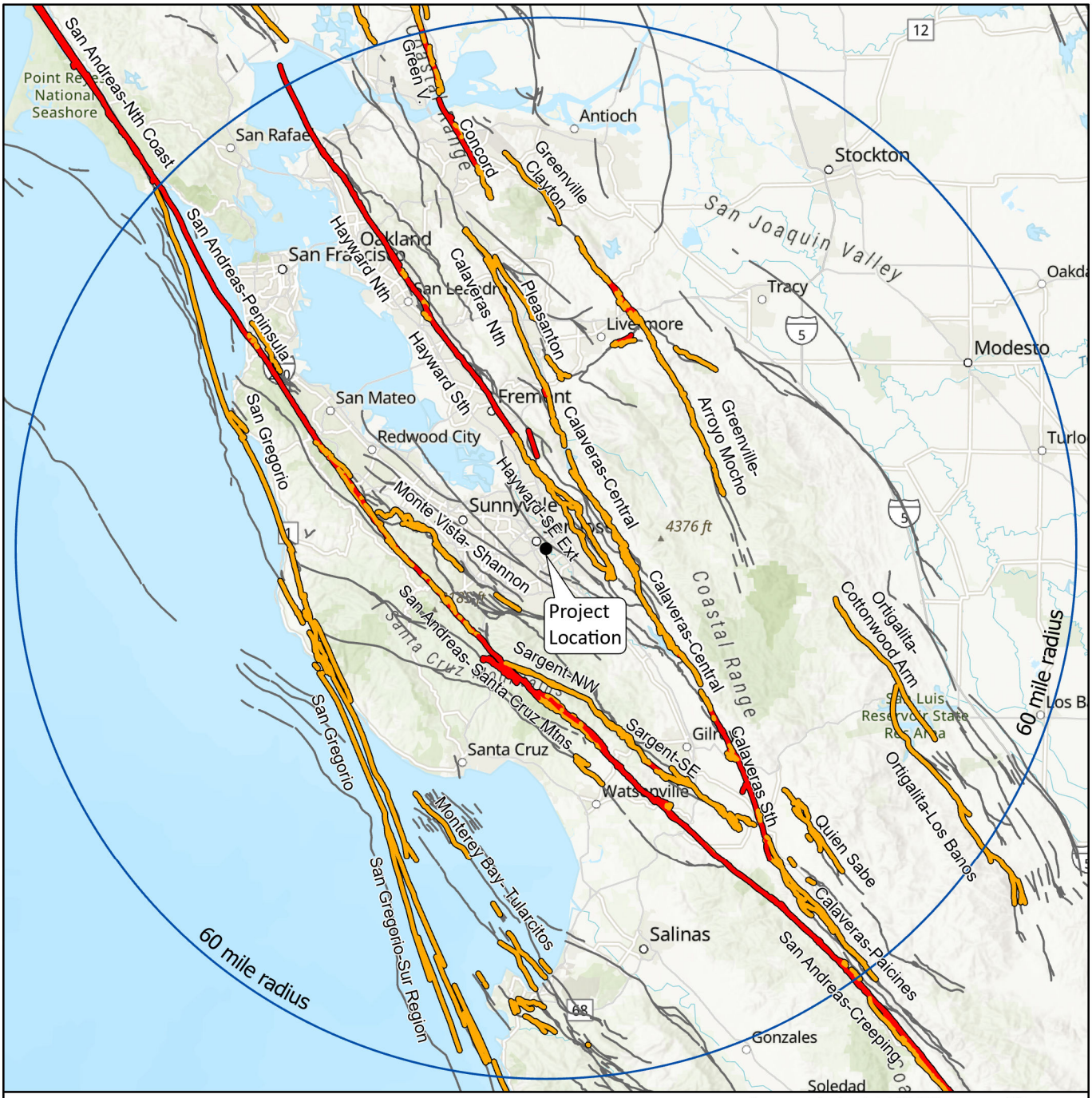
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SAN JOSE, CALIFORNIA

2022-104-GEO	O.: 4

PLATE NO.: 1

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Legend

Fault trace

Fault name

Cascade

Historic <150 yrs

Latest Quaternary <15,000 yrs

Quaternary <1.6 million yrs

MAP LOCATION



N

0 Scale: 1:1,100,000 30 Miles

Fault data source:

USGS & CGS. Quaternary Fault and Fold Database for the United States. Accessed 4/28/2019.

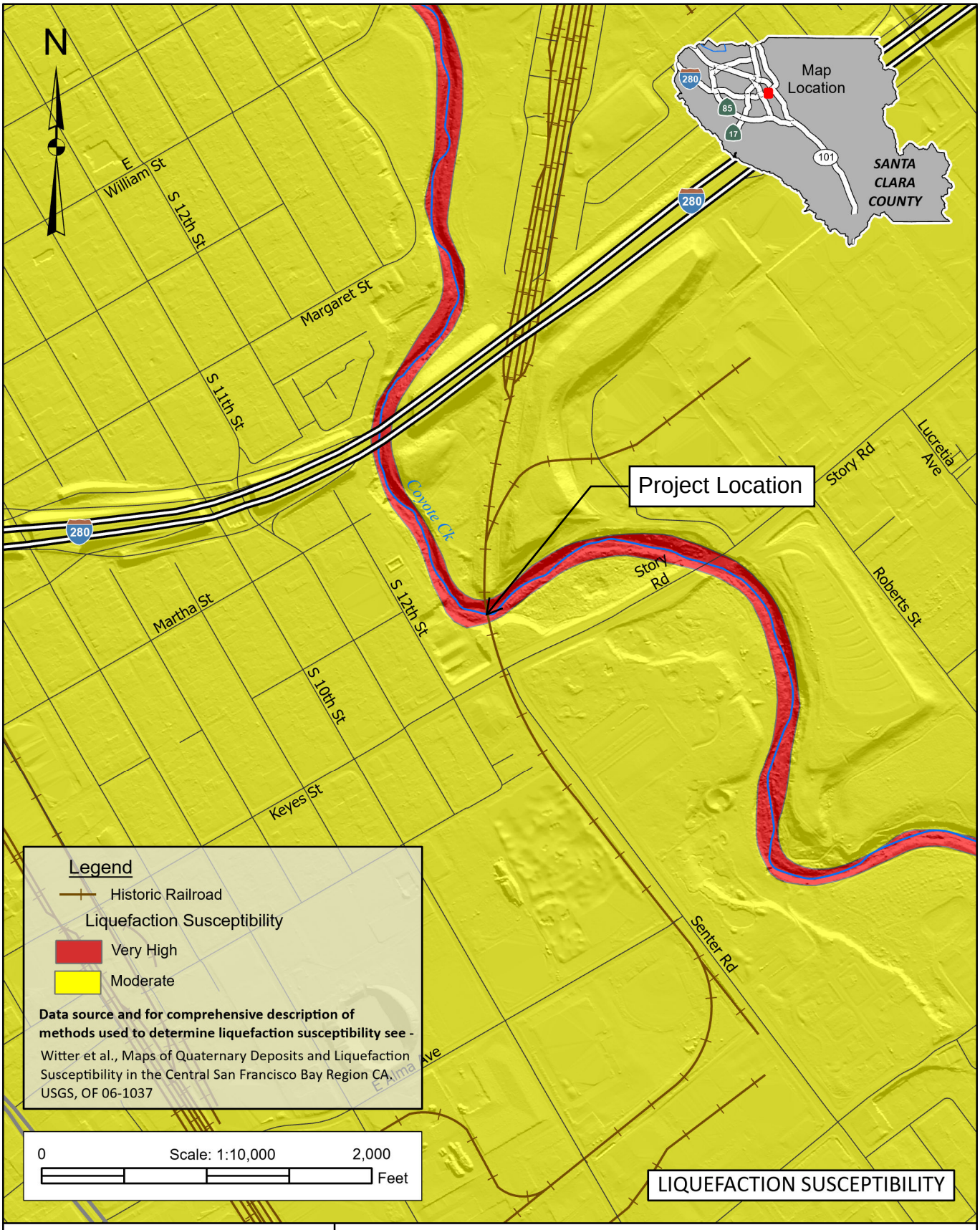
FAULT MAP

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PLATE NO.: 5



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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT
SAN JOSE, CALIFORNIA

2022-104-GEO	JO.: 6

APPENDIX A

LOG OF TEST BORINGS

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

BORING RECORD LEGEND

PARIKH
Practicing in the Geosciences

FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date:

Job No.: 2022-104-GEO

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Plate:

CONSISTENCY OF COHESIVE SOILS				
Descriptor	**Unconfined Compressive Strength (tsf)**	**Pocket Penetrometer (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS	
Descriptor	**SPT N_{60} - Value (blows / foot)**
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS	
Descriptor	**Criteria**
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE		
Descriptor		**Size**
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS	
Descriptor	**Criteria**
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION	
Descriptor	**Criteria**
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

BORING RECORD LEGEND

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Plate:

ROCK GRAPHIC SYMBOLS


IGNEOUS ROCK


SEDIMENTARY ROCK


METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

CORE RECOVERY CALCULATION (%)

$$\text{Core Recovery (\%)} = \frac{\text{Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

RQD CALCULATION (%)

$$\text{RQD (\%)} = \frac{\text{Length of intact core pieces} > 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

BORING RECORD LEGEND

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Plate:

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
El Bhango	**3-28-22**	**3-28-22**	**37° 19' 34.5936" / -121° 51' 57.3402"**	**B-1**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Exploration Geoservices Inc			**1.00' Rt Sta Refers to the "Boring Location Map"**	**104.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Hollow-Stem Auger			**B61**	**7.5 in**
SAMPLER TYPE(S) AND SIZE(S) ID			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT & MC			**Auto 140 lbs / 30 inches**	**54%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Cement Grout			**29.0 ft**	**61.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0		SANDY lean CLAY (CL); hard; dark yellowish brown; moist; medium plasticity fines; a few fine subrounded gravel, PP>4.0 tsf (UC=1.5 ksf).												
102.00	2				1	11 24 23	47	9	108	UC = 0.75	78				UC
	3		(UC=1.5 ksf).												
100.00	4														
	5				2	12 20 25	45	9	104		72				
98.00	6		PP>4.0 tsf.												
	7														
96.00	8														
	9														
94.00	10				3	9 10 12	22	9			78				
	11		Lean CLAY (CL); hard; dark yellowish brown; moist; medium plasticity fines; trace fine SAND.												
92.00	12														
	13														
90.00	14														
	15		Yellowish brown; PP>4.0 tsf.		4	12 24 32	56	15	106		78				
88.00	16														
	17														
86.00	18														
	19														
	20														

(continued)

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-1 | Job No.: 2022-104-GEO

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Plate: A-A

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	20		Lean CLAY (CL); hard; dark yellowish brown; moist; medium plasticity fines; trace fine SAND. *layer description continued from previous page*		5	14 18 19	37	12	100	UC = 1.6	72				UC
	21		Olive brown; high plasticity fines; PP>4.0 tsf.												
82.00	22		Lean CLAY (CL) *(continued)*. (UC=3.2 ksf).												
	23														
80.00	24														
	25		Stiff; dark yellowish brown; low to medium plasticity fines.		6	4 8 10	18	22			78				
78.00	26														
	27														
76.00	28														
	29														
74.00	30				7	6 7 8	15	24	104		89				
	31		Grayish brown; PP=1.5 tsf.												
72.00	32														
	33		SANDY SILT (ML); very stiff; dark greenish gray; moist; low plasticity fines; fine SAND; (LL=24%, PI=3%).												
70.00	34														
	35				8	12 24 24	48	23			78				PI
68.00	36														
	37														
66.00	38		SILT (ML); stiff; greenish gray; moist; low plasticity fines; (LL=22%, PI=2%).												
	39														
64.00	40				9	6 7 7	14	22			89				PI
	41														
62.00	42														
	43		Lean CLAY (CL); soft; greenish gray; wet; medium plasticity fines; PP=0.5 tsf.												
	44														

(continued)

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-1 | Job No.: 2022-104-GEO

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Plate: A-B

PCI-CT 5 BR 2022-104-GEO.GPJ TEMPLATE 7-22-11.GDT 4/27/22

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
			Lean CLAY (CL) *(continued)*. Lean CLAY (CL); soft; greenish gray; wet; medium plasticity fines; PP=0.5 tsf. *layer description continued from previous page*												
	45				10	8 10 12	22	27	97		89				
58.00	46														
	47														
56.00	48														
	49														
54.00	50		Very stiff; moist; PP=2.0 tsf.		11	10 16 32	48	23	102	UC = 0.6	78				
	51		(UC=1.2 ksf).												UC
52.00	52														
	53		SILT (ML); very stiff; greenish gray; moist; low plasticity fines; pockets of silt with sand; (LL=29%, PI=5%).												
50.00	54														
	55				12	5 7 11	18	25			89				
48.00	56														PI
	57														
46.00	58														
	59														
44.00	60		Lean CLAY (CL); very stiff; greenish gray; moist; medium plasticity fines; PP=1.5 tsf.		13	13 18 18	36	34			89				
	61														
42.00	62		Bottom of borehole at 61.5 ft bgs/Elev. 42.5 ft Borehole backfilled wth cement grout. This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2010) except as noted on the Soil or Rock Legend or below.												
	63														
40.00	64														
	65														
38.00	66														
	67														
36.00	68														

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-1 | Job No.: 2022-104-GEO

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-C

PCI-CT 5 BR 2022-104-GEO.GPJ TEMPLATE 7-22-11.GDT 4/27/22

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
El Bhango	**3-29-22**	**3-29-22**	**37° 19' 37.4052" / -121° 51' 57.8442"**	**B-2**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Exploration Geoservices Inc			**1.00' Rt Sta Refers to the "Boring Location Map"**	**106.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Hollow-Stem Auger			**B53B**	**7.5 in**
SAMPLER TYPE(S) AND SIZE(S) ID			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT & MC			**Auto 140 lbs / 30 inches**	**61%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS DURING DRILLING AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Cement Grout			**38.0 ft**	**61.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	0														
	1		Poorly graded GRAVEL with SAND (GP); very loose to loose; gray; moist; mostly fine subangular gravel, about 3/8" dia..												
104.00	2				1	2 2 2	4	1			67				
	3														
102.00	4														
	5		(+#4=71.3%, -#200=5.0%).		2	4 3 4	7	1			78				
100.00	6														PA
	7														
98.00	8														
	9														
96.00	10				3	4 8 9	17	1			78				
	11														
94.00	12		SILTY SAND with GRAVEL (SM); medium dense; brown; moist; fine SAND; fine angular gravel; (+#4=32.1%; -#200=20.9%).												
	13														
92.00	14														
	15				4	5 5 8	13	7	95		78				
90.00	16														PA
	17														
88.00	18		SANDY lean CLAY (CL); hard; dark yellowish brown; moist; medium plasticity fines; PP>4.0 tsf; (UC=0.6 ksf).												
	19														
	20														

(continued)

LOG OF TEST BORING

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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-2 | Job No.: 2022-104-GEO

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Plate: A-A

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
	20		SANDY lean CLAY (CL); hard; dark yellowish brown; moist; medium plasticity fines; PP>4.0 tsf; (UC=0.6 ksf). *layer description continued from previous page*		5	6 10 15	25				78				
	21		SANDY lean CLAY (CL)*(continued)*.					13	82	UC = 0.3					UC
84.00	22														
	23		Lean CLAY (CL); hard; olive brown; moist; medium plasticity fines; PP>4.0 tsf.												
82.00	24														
	25				6	17 26 35	61				72				
80.00	26							13	116						
	27														
78.00	28		SANDY lean CLAY (CL); hard; olive brown; moist; medium plasticity fines; PP>4.0 tsf; (UC=6.2 ksf).												
	29														
76.00	30				7	20 19 29	48				72				
	31							13	117	UC = 3.1					UC
74.00	32														
	33														
72.00	34		SANDY SILT (ML); very stiff; olive brown; moist; nonplastic to low plasticity fines; (+#4=0.1%; -#200=54.7%).												
	35				8	25 23 20	43				78				
70.00	36							14	110						PA
	37														
68.00	38		Lean CLAY (CL); very stiff; greenish gray; moist; medium plasticity fines; PP=2.5 tsf.												
	39														
66.00	40				9	5 9 13	22				78				
	41							28	93						
64.00	42														
	43														
	44														

(continued)

LOG OF TEST BORING

PARIKH Practicing in the Geosciences

FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-2 | Job No.: 2022-104-GEO

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Plate: A-B

PCI-CT 5 BR 2022-104-GEO.GPJ TEMPLATE 7-22-11.GDT 4/27/22

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Depth	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	UC/UU in Shear. Str. (tsf)	Recovery (%)	RQD (%)	Drilling Method	Casing Depth	Remarks
			Lean CLAY (CL) *(continued)*.												
	45		Lean CLAY (CL); very stiff; greenish gray; moist; medium plasticity fines; PP=2.5 tsf.*layer description continued from previous page*		10	5 9 12	21				78				
60.00	46		Medium stiff to stiff; high plasticity fines; PP=1.0 tsf.					31	92						
	47														
58.00	48														
	49														
56.00	50				11	7 9 11	20				78				
	51		PP=1.0 tsf; (UC=0.7 ksf).					23	104	UC = 0.35					UC
54.00	52														
	53														
52.00	54														
	55				12	10 12 12	24				78				
50.00	56		PP=2.0 tsf.					27	98						
	57														
48.00	58		SANDY SILT (ML); stiff to very stiff; gray; moist; nonplastic fines; fine SAND; PP=0.75 tsf.												
	59														
46.00	60				13	11 22 25	47				72				
	61							27	97						
44.00	62		Bottom of borehole at 61.5 ft bgs/Elev. 44.5 ft Borehole backfilled wth cement grout. This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2010) except as noted on the Soil or Rock Legend or below.												
	63														
42.00	64														
	65														
40.00	66														
	67														
38.00	68														

LOG OF TEST BORING

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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

Date: 4/22/2022 | Boring ID: B-2 | Job No.: 2022-104-GEO

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Plate: A-C

PCI-CT 5 BR 2022-104-GEO.GPJ TEMPLATE 7-22-11.GDT 4/27/22

APPENDIX B

LABORATORY TEST SUMMARY

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Unconfined Shear Strength (tsf)
B-1	1	3.0	CL	9.4	107.9						0.75
B-1	2	6.0	CL	9.1	103.6						
B-1	3	11.0	CL	9.2	-						
B-1	4	16.0	CL	14.8	106.3						
B-1	5	21.0	CL	12.1	99.5						1.6
B-1	6	26.0	CL	22.3	-						
B-1	7	31.0	CL	23.5	104.4						
B-1	8	36.0	ML	23.1	-	24	21	3			
B-1	9	41.0	ML	21.8	-	22	20	2			
B-1	10	46.0	CL	27.1	96.6						
B-1	11	51.0	CL	23.0	102.1						0.6
B-1	12	56.0	ML	25.2	-	29	24	5			
B-1	13	61.0	CL	33.7	-						
B-2	1	3.0	GP	1.1	-						
B-2	2	6.0	GP	1.3	-				71.3	5.0	
B-2	3	11.0	GP	1.2	-						
B-2	4	16.0	SM	6.7	94.5				32.1	20.9	
B-2	5	21.0	CL	12.8	81.8						0.3
B-2	6	26.0	CL	13.1	116.1						
B-2	7	31.0	CL	12.9	116.7						3.1
B-2	8	36.0	ML	13.8	110.2				0.1	54.7	
B-2	9	41.0	CL	28.4	93.2						
B-2	10	46.0	CL	30.6	91.5						
B-2	11	51.0	CL	22.9	103.6						0.35
B-2	12	56.0	CL	26.9	97.5						
B-2	13	61.0	ML	26.9	96.8						

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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT
SAN JOSE, CALIFORNIA

JOB NO: 2022-104-GEO | **PLATE NO:** B-1A

Borehole	Sample Number	Depth	Classi-fication	Water Content	Dry Density	Liquid Limit	Plastic Limit	Plasticity Index	% > Sieve 4	% < Sieve 200	Compressive Strength (tsf)
Creek Bed Sample		0.0	GW	-	-				55.8	3.9	

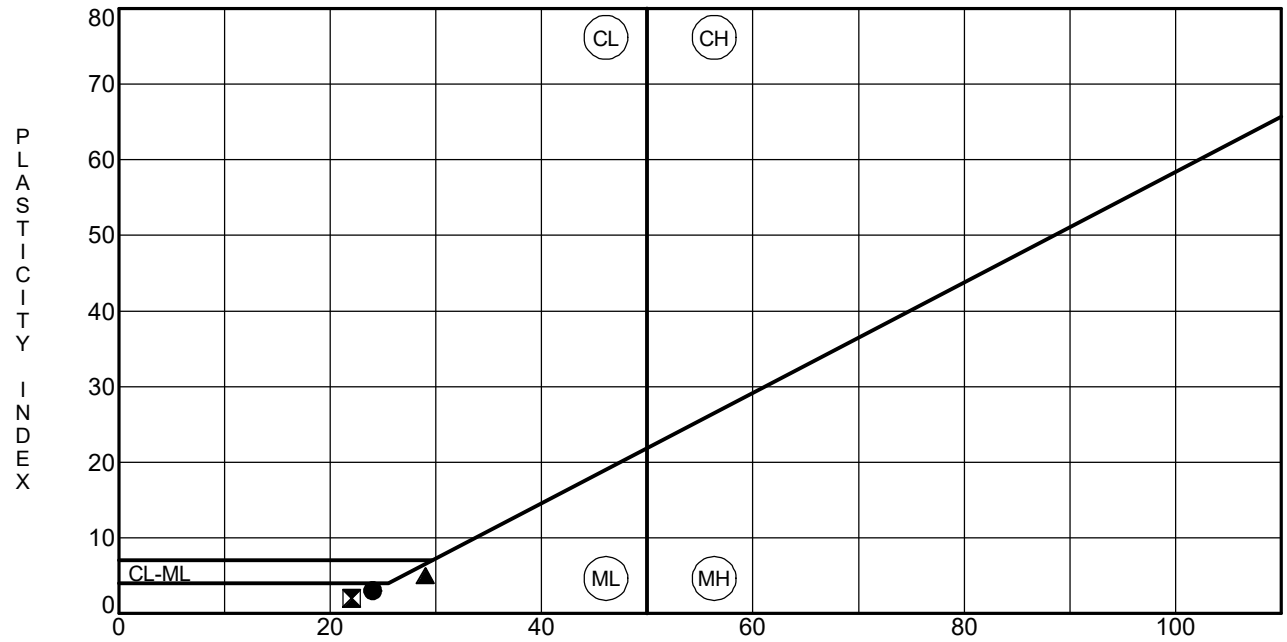
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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

JOB NO: 2022-104-GEO | **PLATE NO: B-1B**

ATTERBERG LIMITS



PLASTICITY

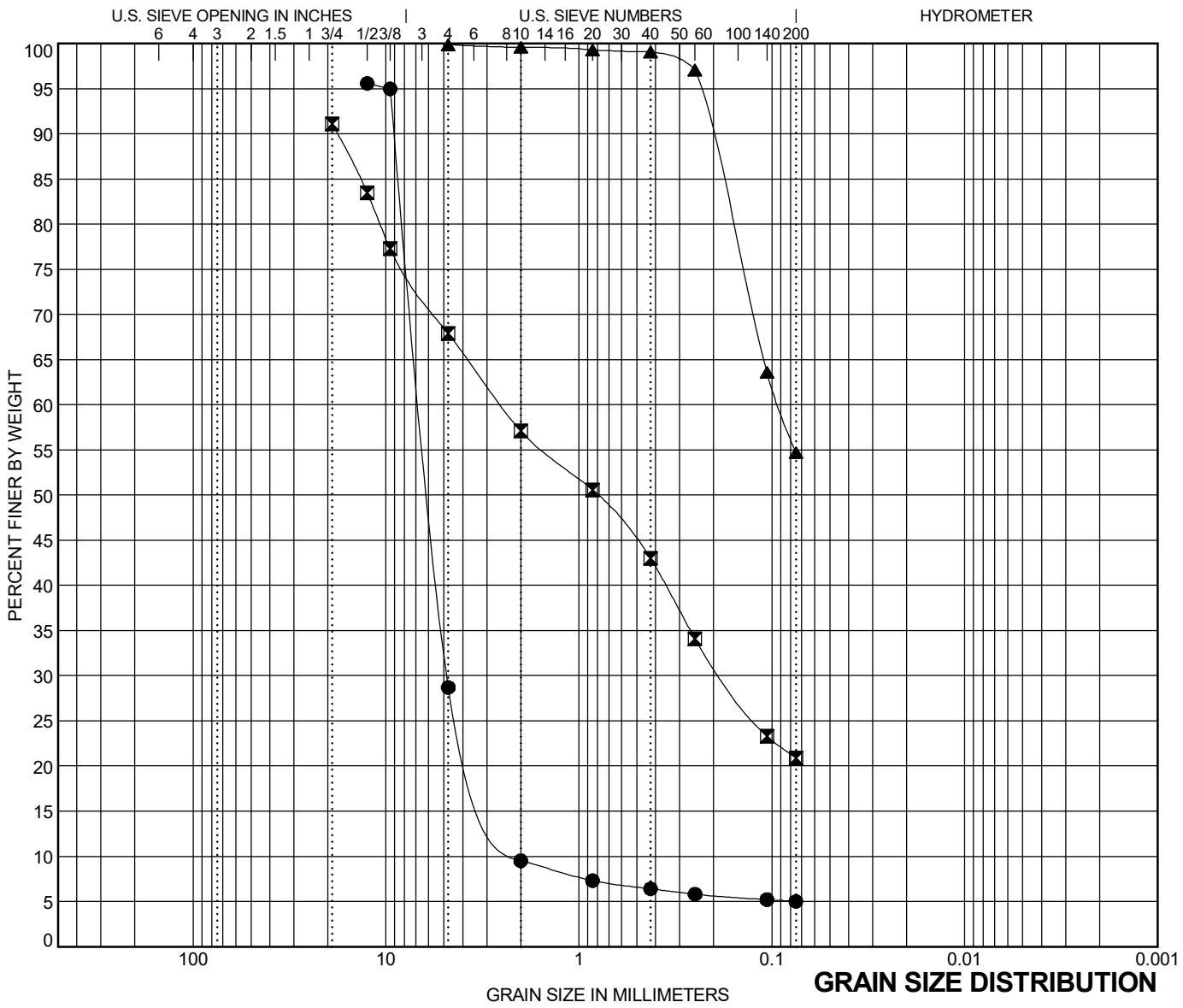
	BOREHOLE	SAMPLE #	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**8**	**36.0**	**24**	**21**	**3**		**SANDY SILT**
⊠	**B-1**	**9**	**41.0**	**22**	**20**	**2**		**SILT**
▲	**B-1**	**12**	**56.0**	**29**	**24**	**5**		**SILT**

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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT
SAN JOSE, CALIFORNIA

JOB NO: 2022-104-GEO | **PLATE NO: B-2**

GRAIN SIZE DISTRIBUTION CURVE



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

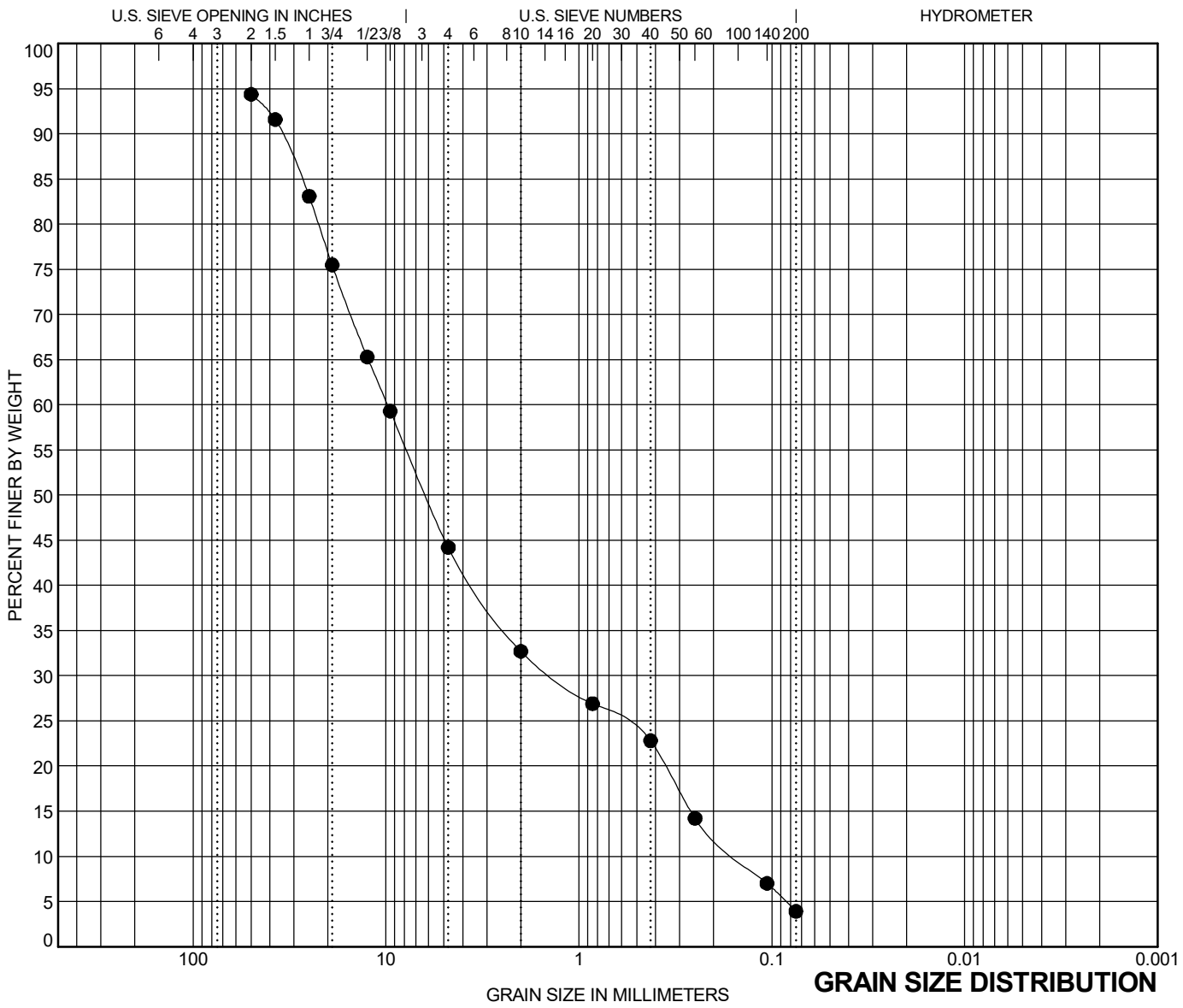
	BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	B-2	2	6.0	Poorly graded GRAVEL with SAND				1.72	3.22
⊠	B-2	4	16.0	SILTY SAND with GRAVEL					
▲	B-2	8	36.0	SANDY SILT					

	BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-2	2	6.0	12.5	6.589	4.815	2.046	66.9	23.7	5.0	
⊠	B-2	4	16.0	19	2.523	0.18		23.2	47.0	20.9	
▲	B-2	8	36.0	4.75	0.092				45.2	54.7	

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FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT
SAN JOSE, CALIFORNIA

JOB NO: 2022-104-GEO | **PLATE NO: B-3A**



GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BORING	SAMPLE #	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**Creek Bed Sample**	**0.0**	**Well-graded GRAVEL with SAND**				**1.21**	**64.72**

BORING	SAMPLE #	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**Creek Bed Sample**	**0.0**	**50**	**9.809**	**1.343**	**0.152**	**50.2**	**40.3**	**3.9**	

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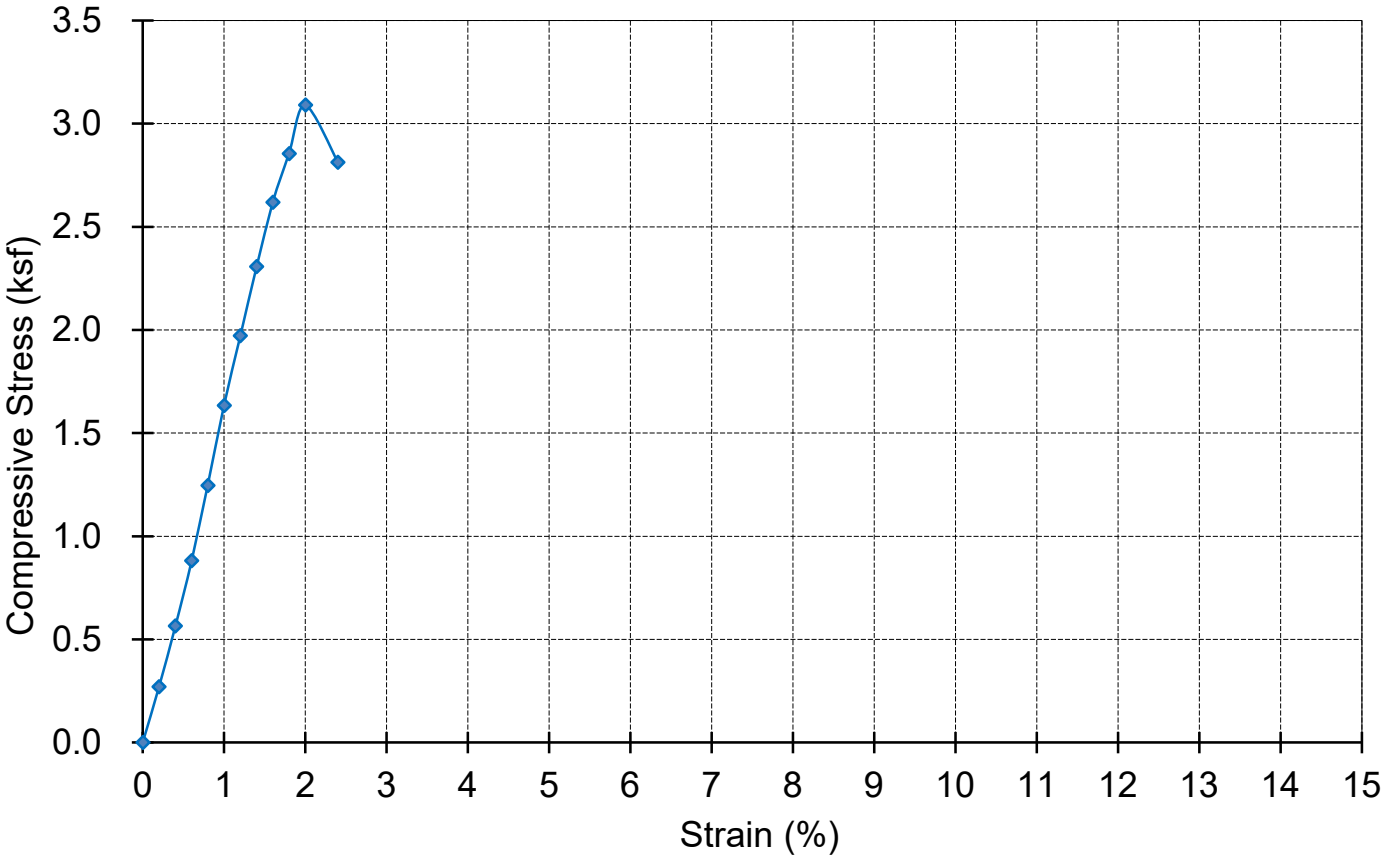
FIVE WOUNDS TRAIL - TRESTLE EVALUATION PROJECT

SAN JOSE, CALIFORNIA

JOB NO: 2022-104-GEO | **PLATE NO: B-3B**

UNCONFINED COMPRESSION TEST

UNCONFINED COMPRESSION TEST



Boring No.:	B-1	**Unconfined Compressive Strength (ksf):**	3.1
Sample No. :	1	**Shear Strength (ksf)**	1.5
Depth (feet):	3	**Strain @ Failure (%):**	2.0
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	108
Test Method	ASTM D2166	**Water Content (%):**	9
Material Type:	CL		
Material Description:	Sandy Lean CLAY, (CL), Dark Yellowish Brown		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

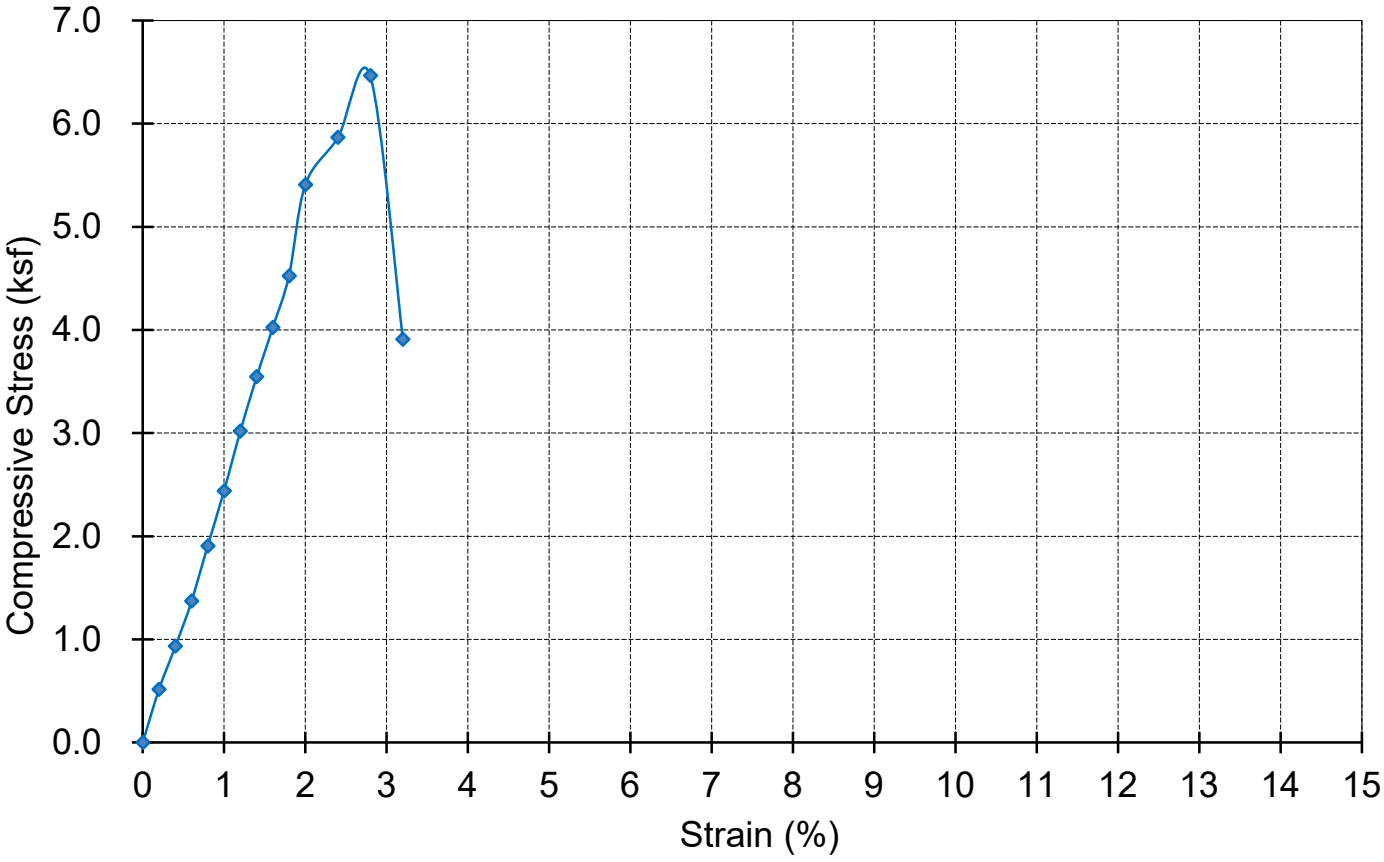
Remarks:

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Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4A

UNCONFINED COMPRESSION TEST



Boring No.:	B-1	**Unconfined Compressive Strength (ksf):**	6.5
Sample No. :	5	**Shear Strength (ksf)**	3.2
Depth (feet):	21	**Strain @ Failure (%):**	2.8
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	100
Test Method	ASTM D2166	**Water Content (%):**	12
Material Type:	CL		
Material Description:	Lean CLAY, (CL), Olive Brown		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

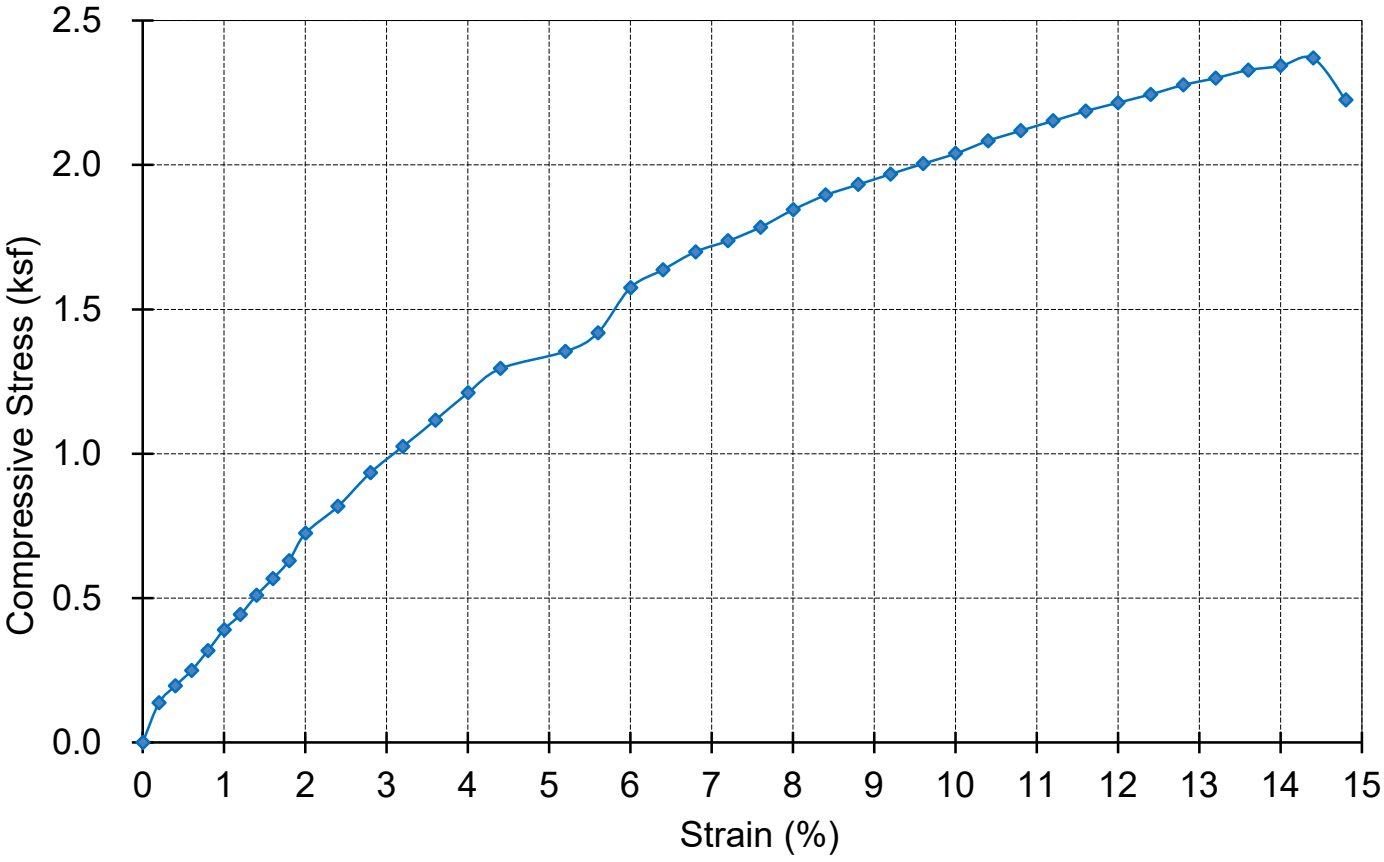
Remarks:

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Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4B

UNCONFINED COMPRESSION TEST



Boring No.:	B-1	**Unconfined Compressive Strength (ksf):**	2.4
Sample No. :	11	**Shear Strength (ksf)**	1.2
Depth (feet):	51	**Strain @ Failure (%):**	14.4
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	102
Test Method	ASTM D2166	**Water Content (%):**	23
Material Type:	CL		
Material Description:	Lean CLAY, (CL), Greenish GRAY		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

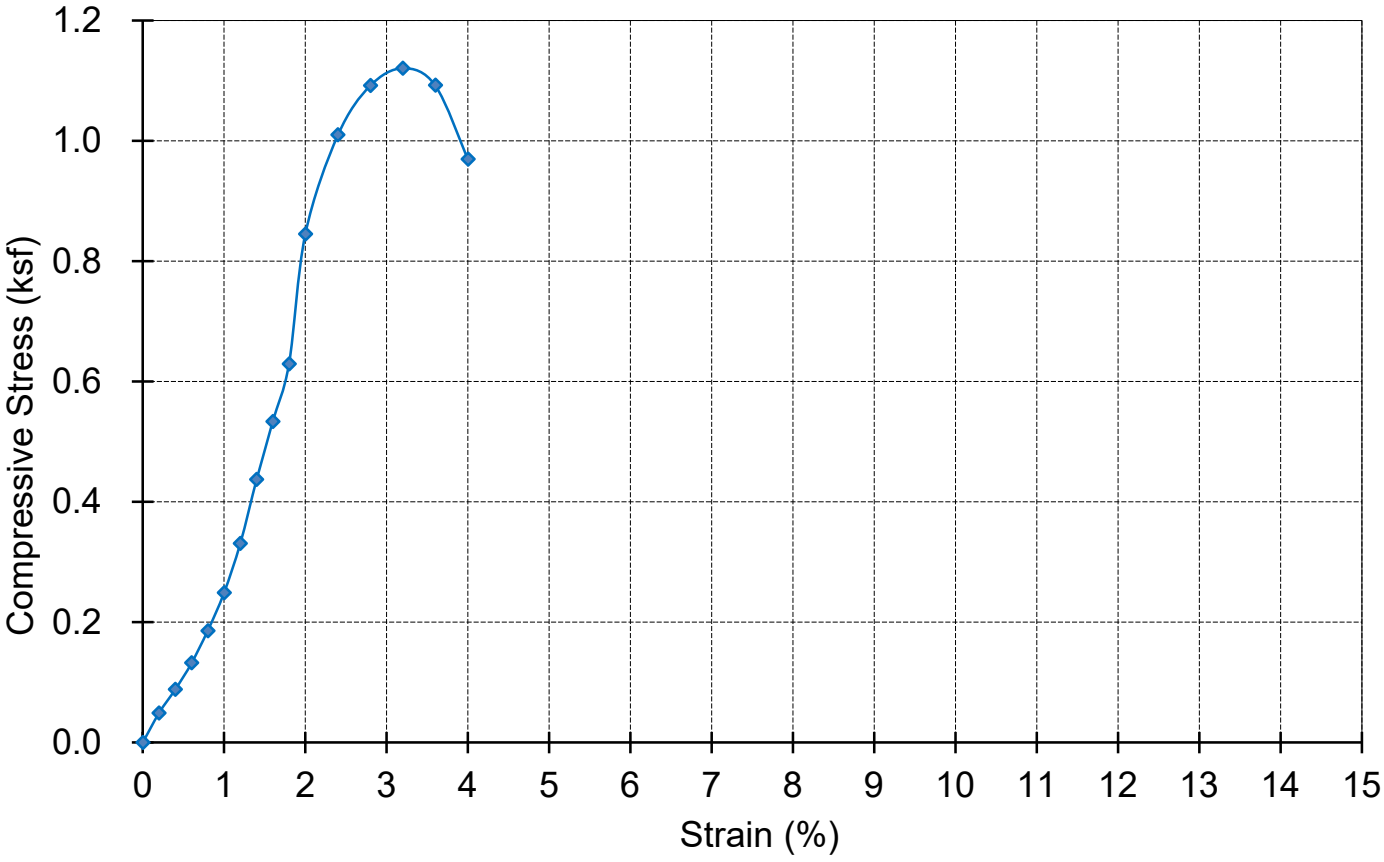
Remarks:

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Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4C

UNCONFINED COMPRESSION TEST



Boring No.:	B-2	**Unconfined Compressive Strength (ksf):**	1.1
Sample No. :	5	**Shear Strength (ksf)**	0.6
Depth (feet):	21	**Strain @ Failure (%):**	3.2
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	82
Test Method	ASTM D2166	**Water Content (%):**	13
Material Type:	CL		
Material Description:	Sandy Lean CLAY, (CL), Dark Yellowish Brown		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

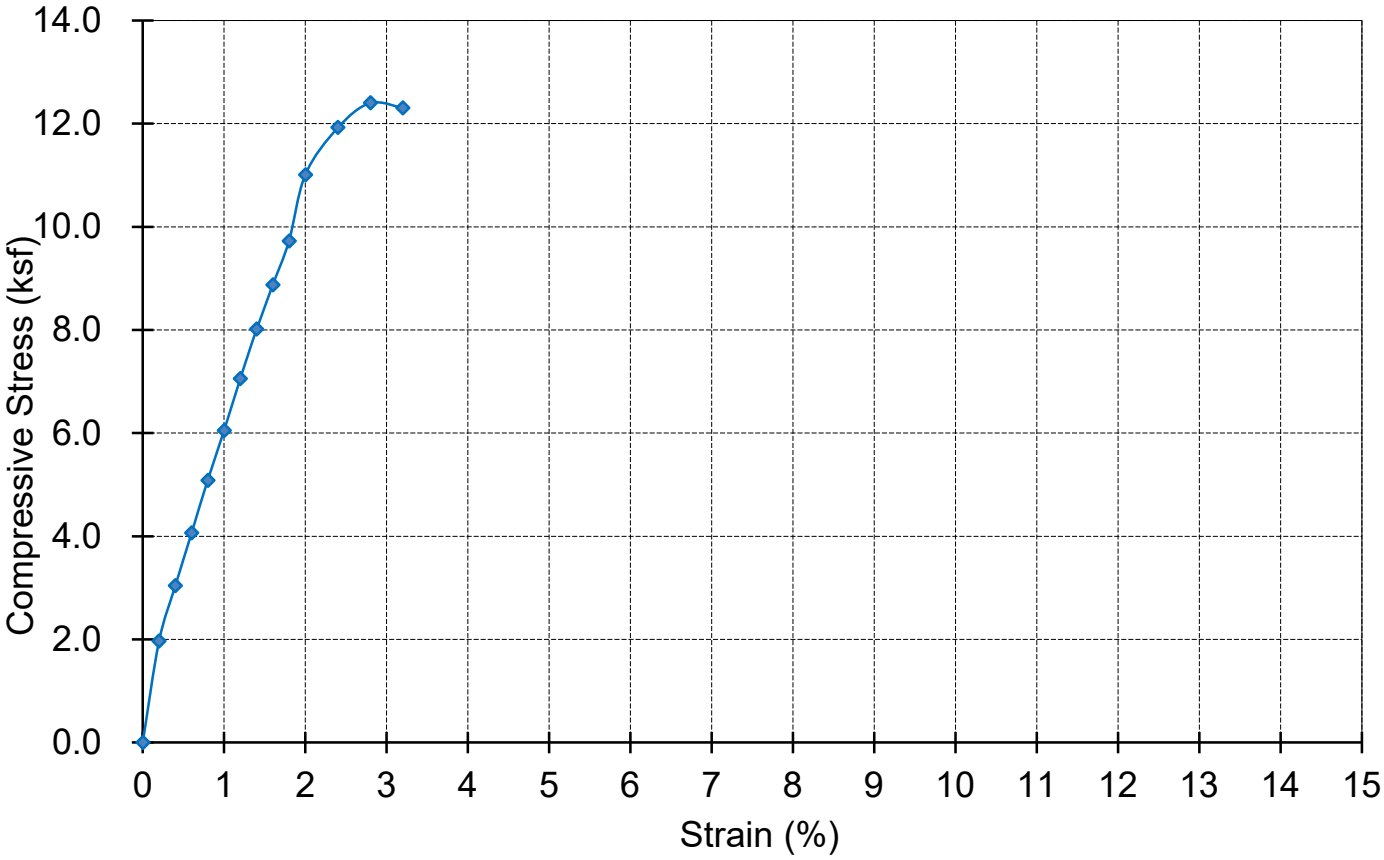
Remarks:

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Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4D

UNCONFINED COMPRESSION TEST



Boring No.:	B-2	**Unconfined Compressive Strength (ksf):**	12.4
Sample No. :	7	**Shear Strength (ksf)**	6.2
Depth (feet):	31	**Strain @ Failure (%):**	2.8
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	117
Test Method	ASTM D2166	**Water Content (%):**	13
Material Type:	CL		
Material Description:	Sandy Lean CLAY, (CL), Olive Brown		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

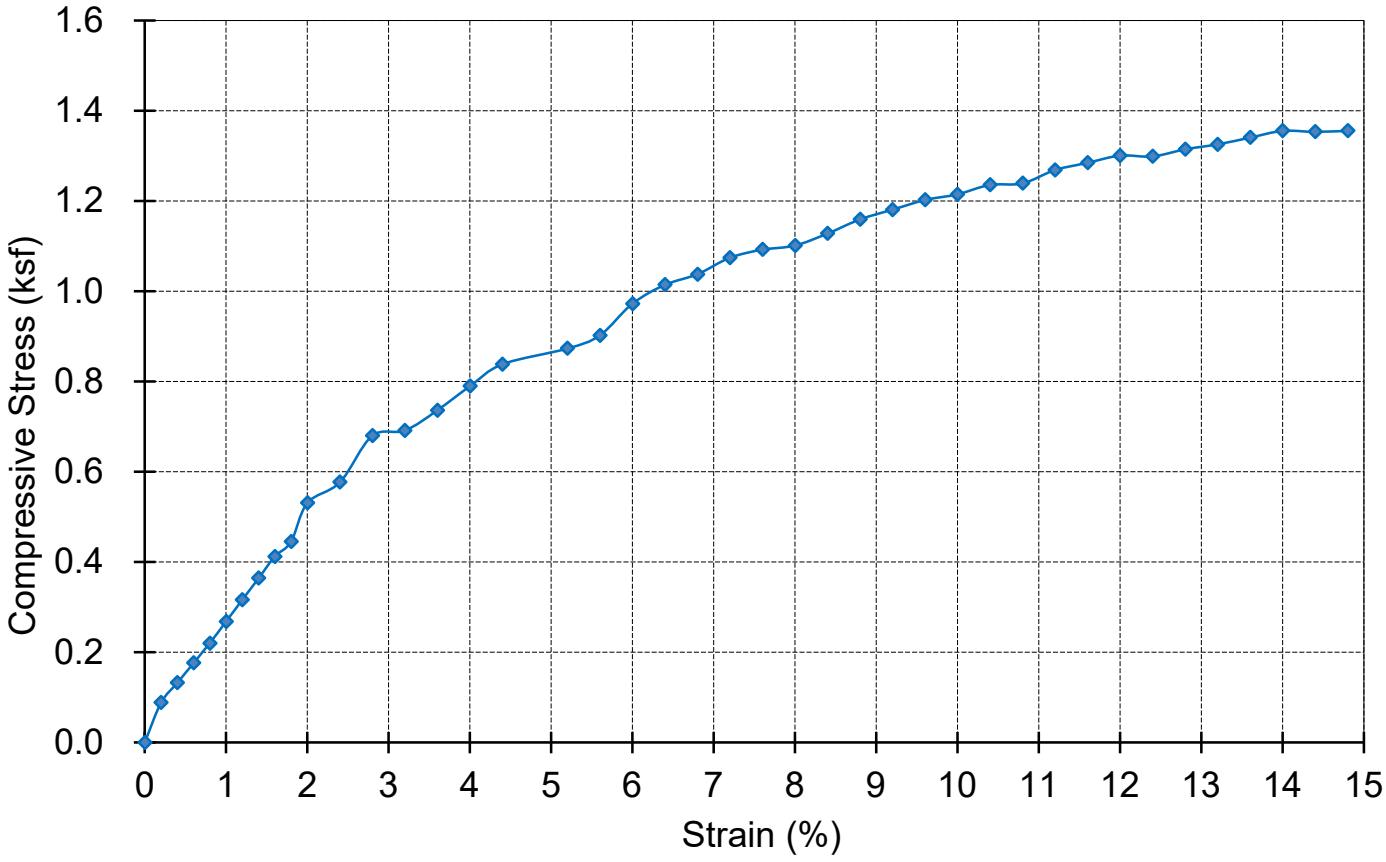
Remarks:

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Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4E

UNCONFINED COMPRESSION TEST



Boring No.:	B-2	**Unconfined Compressive Strength (ksf):**	1.4
Sample No. :	11	**Shear Strength (ksf)**	0.7
Depth (feet):	51	**Strain @ Failure (%):**	14.0
Sample Type:	MC - 2.416 inch dia.	**Initial Dry Density (pcf):**	104
Test Method	ASTM D2166	**Water Content (%):**	23
Material Type:	CL		
Material Description:	Lean CLAY, (CL), Greenish GRAY		

Initial Height (inch):	5.00
Initial Diameter (inch):	2.42
Initial Area (ft^2):	0.032
Strain Rate (inch/min)	0.1

Remarks:

PARIKH
Practicing in the Geosciences

Five Wounds Trail - Trestle Evaluation Project
San Jose, CA

JOB NO.: 2022-104-GEO | **PLATE NO.:** B-4F

CORROSION TEST

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 05/04/2022
Date Submitted 04/27/2022

To: Do Nguyen
Parikh Consultants, Inc.
1497 N.Milpitas Blvd
Milpitas, CA 95035

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

The reported analysis was requested for the following location:
Location : 2020-104-GEO FIVE W. Site ID : B-1 #7@31.
Thank you for your business.

* For future reference to this analysis please use SUN # 87234-181468.

EVALUATION FOR SOIL CORROSION

Soil pH	7.74	
Minimum Resistivity	2.17 ohm-cm (x1000)	
Chloride	15.1 ppm	00.00151 %
Sulfate	46.0 ppm	00.00460 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

PLATE NO.: B-5A

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 05/04/2022
Date Submitted 04/27/2022

To: Do Nguyen
Parikh Consultants, Inc.
1497 N.Milpitas Blvd
Milpitas, CA 95035

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

The reported analysis was requested for the following location:
Location : 2020-104-GEO FIVE W. Site ID : B-2 #9@41.
Thank you for your business.

* For future reference to this analysis please use SUN # 87234-181469.

EVALUATION FOR SOIL CORROSION

Soil pH	7.52	
Minimum Resistivity	1.37 ohm-cm (x1000)	
Chloride	13.2 ppm	00.00132 %
Sulfate	71.5 ppm	00.00715 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

PLATE NO.: B-5B

APPENDIX C

GROUND MOTION DATA SHEET

ARS Online V3.0.2

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 37.326303 **Longitude:** -121.865962 **Vs30 (m/s):** 241 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.64	0.74	1	1	1	0.64	0.74
0.10	1.14	1.25	1	1	1	1.14	1.25
0.20	1.41	1.68	1	1	1	1.41	1.68
0.30	1.41	1.89	1	1	1	1.41	1.89
0.50	1.24	1.85	1	1	1	1.24	1.85
0.75	1.03	1.53	1	1	1.1	1.13	1.68
1.0	0.85	1.28	1	1	1.2	1.02	1.53
2.0	0.48	0.7	1	1	1.2	0.57	0.84
3.0	0.3	0.46	1	1	1.2	0.37	0.55
4.0	0.22	0.32	1	1	1.2	0.26	0.38
5.0	0.17	0.24	1	1	1.2	0.21	0.28

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	6.75
mean site-source distance (km, for Sa at 1s)	13.9

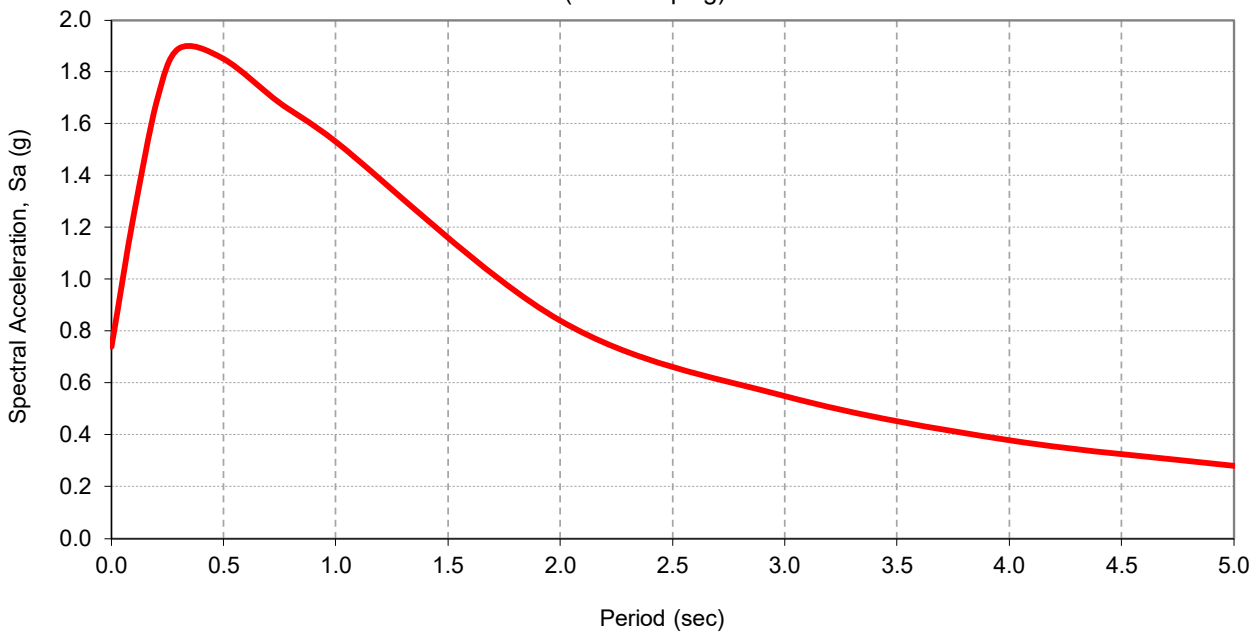
Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 13.9 Update

RECOMMENDED ARS DESIGN CURVE

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.3263
Longitude	-121.8660
V_{S30} (m/s) =	241
Mean Magnitude (for PGA)	6.75
Near Fault Factor, Derived from USGS Unified Hazard Site (km) =	13.9

Recommended Response Spectrum

Period (sec)	Spectral Acceleration (2014) (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Design Spectral Acceleration (2014) (g)
0.0	0.74	1	1	0.74
0.1	1.25	1	1	1.25
0.2	1.68	1	1	1.68
0.3	1.89	1	1	1.89
0.5	1.85	1	1	1.85
0.75	1.53	1.1	1	1.68
1.0	1.28	1.2	1	1.53
2.0	0.7	1.2	1	0.84
3.0	0.46	1.2	1	0.55
4.0	0.32	1.2	1	0.38
5.0	0.24	1.2	1	0.28

Source:

1. Caltrans ARS Online tool (V.3.0.2, https://arsonline.dot.ca.gov/)
2. USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/)
3. Caltrans SDC 2.0 was adopted September 1, 2019. Design Spectrum is based on the USGS 975 year uniform hazard spectrum only.

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Practicing in the Geosciences

FIVE WOUNDS TRAIL- TRESTLE EVALUATION PROJECT
SAN JOSE, CALIFORNIA

Project No.: 2022-104-GEO | **Appendix II**

STRENGTH PARAMETERS AND SHEAR WAVE VELOCITY CALCULATION

SOIL STRENGTH PARAMETERS & V_{s30}

Calc By: SC
Date: 4/27/22

PROJECT NAME: ***Five Wounds Trail - Trestle Evaluation Project***
PROJECT NO.: ***2022-104-GEO***
STRUCTURE:
BORING NO.: ***B-1 (Elev. 104') 2022***

SOIL GROUPS
1. SANDS & GRAVELS
2. CLAYS AND PLASTIC SILTS
3. NON TO LOW PLASTIC SILTS
4. YOUNG SEDIMENTARY ROCKS
5. LIQUEFIABLE SANDS (RESIDUAL STRENGTH)
6. LIQUEFIABLE SILTS (RESIDUAL STRENGTH)

BOREHOLE DIA (in)= 7.5
GW DEPTH (ft)= 29
HAMMER ENERGY = 54%
DRILLING RODS (Y/N)= Y

Nd 18
N_{30} 21

V_{sd} (m/s) *206*
V_{s30} (m/s) *241*
Correlation 1) Caltrans

Sample No	Layer Thickness from	to	Sample Depth (ft)	Soil Type	Field Blow Count	Sampler Type	Unit Weight (pcf)	σ_v (psf)	σ_v' (psf)	SPT-$N_{eq.}$	N_{60} CE Corr.	N_{60} CR,CB,CS Corr.	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60, CS}$	Correlated Strength Parameters ϕ (°)	c (psf)	S_r (psf)	Lab Test Results c (psf)	Vs (m/s)
1	0.0	4.0	2	2	47	MC	125	250	250	31	27.5	23.7	1.70	40.3				3437		750	127
2	4.0	8.0	5	2	45	MC	125	625	625	29	26.3	22.7	1.70	38.6				3291			201
3	8.0	11.0	10	2	22	SPT	125	1250	1250	22	19.8	25.2	1.26	31.8				2475			211
4	11.0	18.0	15	2	56	MC	125	1875	1875	36	32.8	32.0	1.03	33.1				4095			254
5	18.0	23.0	20	2	37	MC	125	2500	2500	24	21.6	23.6	0.89	21.2				2706		1,600	183
6	23.0	28.0	25	2	18	SPT	125	3125	3125	18	16.2	20.6	0.80	16.5				2025			235
7	28.0	33.0	30	2	15	MC	125	3750	3688	10	8.8	10.1	0.74	7.4				1097			209
8	33.0	37.5	35	2	48	SPT	125	4375	4001	48	43.2	64.6	0.71	45.7				5400			306
9	37.5	43.0	40	2	14	SPT	125	5000	4314	14	12.6	16.1	0.68	10.9				1575			233
10	43.0	48.0	45	2	22	MC	125	5625	4627	14	12.9	14.8	0.66	9.7				1609			237
11	48.0	52.5	50	2	48	MC	125	6250	4940	31	28.1	32.3	0.64	20.5				3510		600	115
12	52.5	60.0	55	2	18	SPT	125	6875	5253	18	16.2	21.0	0.62	13.0				2025			255
13	60.0	61.5	60	2	36	SPT	125	7500	5566	36	32.4	48.0	0.60	28.8				4050			302

Note:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner), C_N (Overburden) are per Youd 2001
2. For fine-grained materials, the correlation between blow-counts and shear is based on NAVFAC DM 7.1.
3. The phi angle was estimated based on Meyerhof (1956).
4. Residual Strength (Sr) is based on Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading", Caltrans 2011
5. The Vs were correlated based on N_{60} for Soil Types 1,3, 4; based on N_{60} or c_{lab} for Soil Type 2 and based on Sr for Soil Types 5 & 6 per Caltrans Guidelines (2012).
6. Spreadsheet Revision Date: 10/29/13

SOIL STRENGTH PARAMETERS & V_{s30}

Calc By: SC
Date: 4/27/22

PROJECT NAME: ***Five Wounds Trail - Trestle Evaluation Project***
PROJECT NO.: ***2022-104-GEO***
STRUCTURE:
BORING NO.: ***B-2 (Elev. 106') 2022***

SOIL GROUPS
1. SANDS & GRAVELS
2. CLAYS AND PLASTIC SILTS
3. NON TO LOW PLASTIC SILTS
4. YOUNG SEDIMENTARY ROCKS
5. LIQUEFIABLE SANDS (RESIDUAL STRENGTH)
6. LIQUEFIABLE SILTS (RESIDUAL STRENGTH)

BOREHOLE DIA (in)= 7.5
GW DEPTH (ft)= 31
HAMMER ENERGY = 61%
DRILLING RODS (Y/N)= Y

Nd 13
N_{30} 15
V_{sd} (m/s) *178*
V_{s30} (m/s) *208*
Correlation 1) Caltrans

Sample No	Layer Thickness from	to	Sample Depth (ft)	Soil Type	Field Blow Count	Sampler Type	Unit Weight (pcf)	σ_v (psf)	σ_v' (psf)	SPT-$N_{eq.}$	N_{60} CE Corr.	N_{60} CR,CB,CS Corr.	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60, CS}$	Correlated Strength Parameters ϕ (°)	c (psf)	S_r (psf)	Lab Test Results c (psf)	Vs (m/s)
1	0.0	4.0	2	1	4	SPT	125	250	250	4	4.1	3.9	1.70	6.6		6.6	34				117
2	4.0	8.0	5	1	7	SPT	125	625	625	7	7.1	6.9	1.70	11.7	5%	11.7	36				154
3	8.0	12.0	10	1	17	SPT	125	1250	1250	17	17.3	21.5	1.26	27.2		27.2	39				197
4	12.0	18.0	15	1	13	MC	125	1875	1875	8	8.6	8.4	1.03	8.7	21%	13.2	34				203
5	18.0	23.0	20	2	25	MC	125	2500	2500	16	16.5	18.0	0.89	16.1				2065		300	100
6	23.0	27.5	25	2	61	MC	125	3125	3125	40	40.3	44.0	0.80	35.2				5039			289
7	27.5	33.5	30	2	48	MC	125	3750	3750	31	31.7	36.5	0.73	26.6				3965		3,100	250
8	33.5	38.0	35	3	43	MC	125	4375	4125	28	28.4	32.7	0.70	22.8	55%			2842			270
9	38.0	43.0	40	2	22	MC	125	5000	4438	14	14.5	16.7	0.67	11.2				1817			242
10	43.0	48.0	45	2	21	MC	125	5625	4751	14	13.9	16.0	0.65	10.4				1735			243
11	48.0	53.0	50	2	20	MC	125	6250	5064	13	13.2	15.2	0.63	9.6				1652		350	100
12	53.0	57.5	55	2	24	MC	125	6875	5377	16	15.9	18.2	0.61	11.1				1983			255
13	57.5	61.5	60	6	47	MC	125	7500	5690	31	31.1	35.7	0.59	21.2		21.2			1737		190

Note:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner), C_N (Overburden) are per Youd 2001
2. For fine-grained materials, the correlation between blow-counts and shear is based on NAVFAC DM 7.1.
3. The phi angle was estimated based on Meyerhof (1956).
4. Residual Strength (Sr) is based on Caltrans "Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading", Caltrans 2011
5. The Vs were correlated based on N_{60} for Soil Types 1,3, 4; based on N_{60} or c_{lab} for Soil Type 2 and based on Sr for Soil Types 5 & 6 per Caltrans Guidelines (2012).
6. Spreadsheet Revision Date: 10/29/13

LIQUEFACTION ANALYSES

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Five Wounds Trail - Trestle Evaluation Project***
PROJECT NO. ***2022-104-GEO***
BORING NO. ***B-1 (Elev. 104') 2022***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
a_{max} (g)= 0.74
FAULT M_w = 6.75

GW DEPTH (ft)= 29

BOREHOLE DIA (in)= 7.5
HAMMER ENERGY = 54%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 29 *(below OG)*

MSF = 1.31

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*											*CYCLIC STRESS RATIO (CSR)*				*$F.S.=(CRR_{7.5}/CSR)*MSF*Ks*Ka$*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	$SPT\text{-}N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	2	2	47	MC	30.6	0.9	0.75	1.0	1.15	23.7	250.0	1.7	40.3											
4.0	8.0	2	5	2	45	MC	29.3	0.9	0.80	1.0	1.15	24.2	625.0	1.7	41.2											
8.0	11.0	3	10	2	22	SPT	22.0	0.9	0.85	1.2	1.15	23.2	1250.0	1.3	29.4											
11.0	18.0	4	15	2	56	MC	36.4	0.9	0.95	1.0	1.15	35.8	1875.0	1.0	37.0											
18.0	23.0	5	20	2	37	MC	24.1	0.9	0.95	1.0	1.15	23.6	2500.0	0.9	21.2											
23.0	28.0	6	25	2	18	SPT	18.0	0.9	1.00	1.2	1.15	22.4	3125.0	0.8	17.9											
28.0	33.0	7	30	2	15	MC	9.8	0.9	1.00	1.0	1.15	10.1	3687.6	0.7	7.4											
33.0	37.5	8	35	2	48	SPT	48.0	0.9	1.00	1.2	1.15	59.6	4000.6	0.7	42.2											
37.5	43.0	9	40	2	14	SPT	14.0	0.9	1.00	1.2	1.15	17.4	4313.6	0.7	11.8											
43.0	48.0	10	45	2	22	MC	14.3	0.9	1.00	1.0	1.15	14.8	4626.6	0.7	9.7											
48.0	52.5	11	50	2	48	MC	31.2	0.9	1.00	1.0	1.15	32.3	4939.6	0.6	20.5											
52.5	60.0	12	55	2	18	SPT	18.0	0.9	1.00	1.2	1.15	22.4	5252.6	0.6	13.8											
60.0	61.5	13	60	2	36	SPT	36.0	0.9	1.00	1.2	1.15	44.7	5565.6	0.6	26.8											

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\ (N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC $\leq$ 5% a = 0, b = 1.0
   for 5% < FC < 35% a = exp(1.76-(190/FC^2)), b = (0.99+($FC^{1.5}$/1000))
   for FC $\geq$ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:
Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

LIQUEFACTION POTENTIAL ANALYSIS (SPT procedures per Youd et al, 2001)

PROJECT NAME ***Five Wounds Trail - Trestle Evaluation Project***
PROJECT NO. ***2022-104-GEO***
BORING NO. ***B-2 (Elev. 106') 2022***

SOIL GROUPS
1. GRAVELS, SANDS AND NONPLASTIC SILTS
2. CLAYS AND PLASTIC SILTS

FAULT INFO
a_{max} (g)= 0.74
FAULT M_w = 6.75

GW DEPTH (ft)= 31

BOREHOLE DIA (in)= 7.5
HAMMER ENERGY = 61%

CUT(-)/FILL(+) (ft) = 0
DESIGN GW DEPTH (ft)= 31 *(below OG)*

MSF = 1.31

SOIL STRATA							*LIQUEFACTION RESISTANCE ($CRR_{7.5}$)*												*CYCLIC STRESS RATIO (CSR)*				*F.S.=($CRR_{7.5}$/CSR)*MSF*Ks*Ka*			*POST-LIQ. SETTLEMENT*	
Layer Thickness from	to	Sample No	Depth (ft)	Soil Type	Blow Count	Sampler Type	$SPT\text{-}N_{eq.}$	C_E	C_R	C_S	C_B	N_{60}	σ_v' (psf)	C_N	$(N_1)_{60}$	F.C.	$(N_1)_{60,CS}$	$CRR_{7.5}$	σ_v (psf)	σ_v' (psf)	r_d	CSR	Ks	Ka	F.S.	Vol. Strain (%)	ΔD (in)
0.0	4.0	1	2	1	4	SPT	4.0	1.0	0.75	1.2	1.15	4.2	250.0	1.7	7.2		7.2		250.0	250.0	1.0	0.5	1.0	1.0			
4.0	8.0	2	5	1	7	SPT	7.0	1.0	0.80	1.2	1.15	7.9	625.0	1.7	13.4	5%	13.4		625.0	625.0	1.0	0.5	1.0	1.0			
8.0	12.0	3	10	1	17	SPT	17.0	1.0	0.85	1.2	1.15	20.3	1250.0	1.3	25.6		25.6		1250.0	1250.0	1.0	0.5	1.0	1.0			
12.0	18.0	4	15	1	13	MC	8.5	1.0	0.95	1.0	1.15	9.4	1875.0	1.0	9.7	21%	14.3		1875.0	1875.0	1.0	0.5	1.0	1.0			
18.0	23.0	5	20	2	25	MC	16.3	1.0	0.95	1.0	1.15	18.0	2500.0	0.9	16.1												
23.0	27.5	6	25	2	61	MC	39.7	1.0	1.00	1.0	1.15	46.4	3125.0	0.8	37.1												
27.5	33.5	7	30	2	48	MC	31.2	1.0	1.00	1.0	1.15	36.5	3750.0	0.7	26.6												
33.5	38.0	8	35	1	43	MC	28.0	1.0	1.00	1.0	1.15	32.7	4125.4	0.7	22.8	55%	32.3		4375.0	4125.4	0.9	0.5	0.7	1.0	NON-LIQ.		
38	43.0	9	40	2	22	MC	14.3	1.0	1.00	1.0	1.15	16.7	4438.4	0.7	11.2												
43.0	48.0	10	45	2	21	MC	13.7	1.0	1.00	1.0	1.15	16.0	4751.4	0.6	10.4												
48.0	53.0	11	50	2	20	MC	13.0	1.0	1.00	1.0	1.15	15.2	5064.4	0.6	9.6												
53.0	57.5	12	55	2	24	MC	15.6	1.0	1.00	1.0	1.15	18.2	5377.4	0.6	11.1												
57.5	61.5	13	60	1	47	MC	30.6	1.0	1.00	1.0	1.15	35.7	5690.4	0.6	21.2		21.2	0.2	7500.0	5690.4	0.7	0.4	0.7	1.0	(0.48)	1.34%	0.64

Notes:
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner) are per Youd et al. (2001).
2. For correction of overburden, $C_N = (1/\sigma_v')^{0.5}$ with a maximum value of 1.7.
3. The influence of Fines Contents are expressed by the following correction: $(N_1)_{60cs} = a + b\ (N_1)_{60}$
   where a and b = coefficients determined from the following relationships
   for FC ≤ 5% a = 0, b = 1.0
   for 5% < FC < 35% $a = \exp(1.76-(190/FC^2))$, $b = (0.99+(FC^{1.5}/1000))$
   for FC ≥ 35% a = 5.0, b = 1.2
4. For $(N_1)_{60,cs}$ greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Reference:
Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Evaluation of Liquefaction Resistance of Soils, Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10

ABUTMENT WALL LATERAL EARTH PRESSURES

Rankine Active Lateral Pressure Coefficient (K_a)

Project Name/Number: Five Wounds Trail Trestle Evaluation **By:** AL

Structure Name/Number: Abutment Wall Native Soil (Φ'=30-degree) **Date:** 10/13/2022

Parameters	Angle in degrees	Angle in radians	
ϕ	30	0.524	(Friction Angle of Soil)
β	0	0.000	(Backfill angle with horizontal)

K_a	0.333

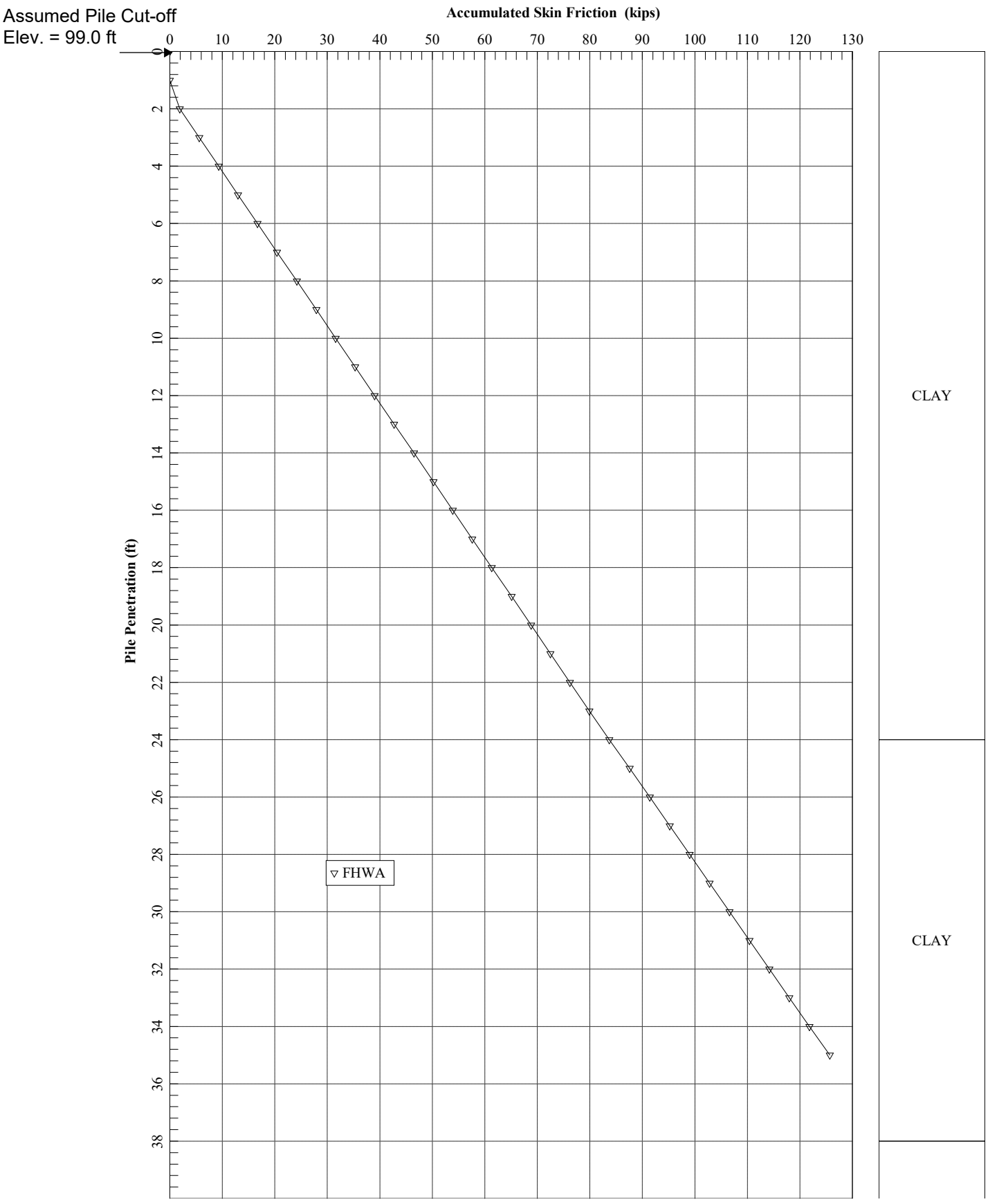
$$K_a = \frac{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

AXIAL PILE CAPACITY ANALYSES



Five Wounds Trail - Trestle Evaluation

Boring B-1
(Elev. = 104.0 ft)

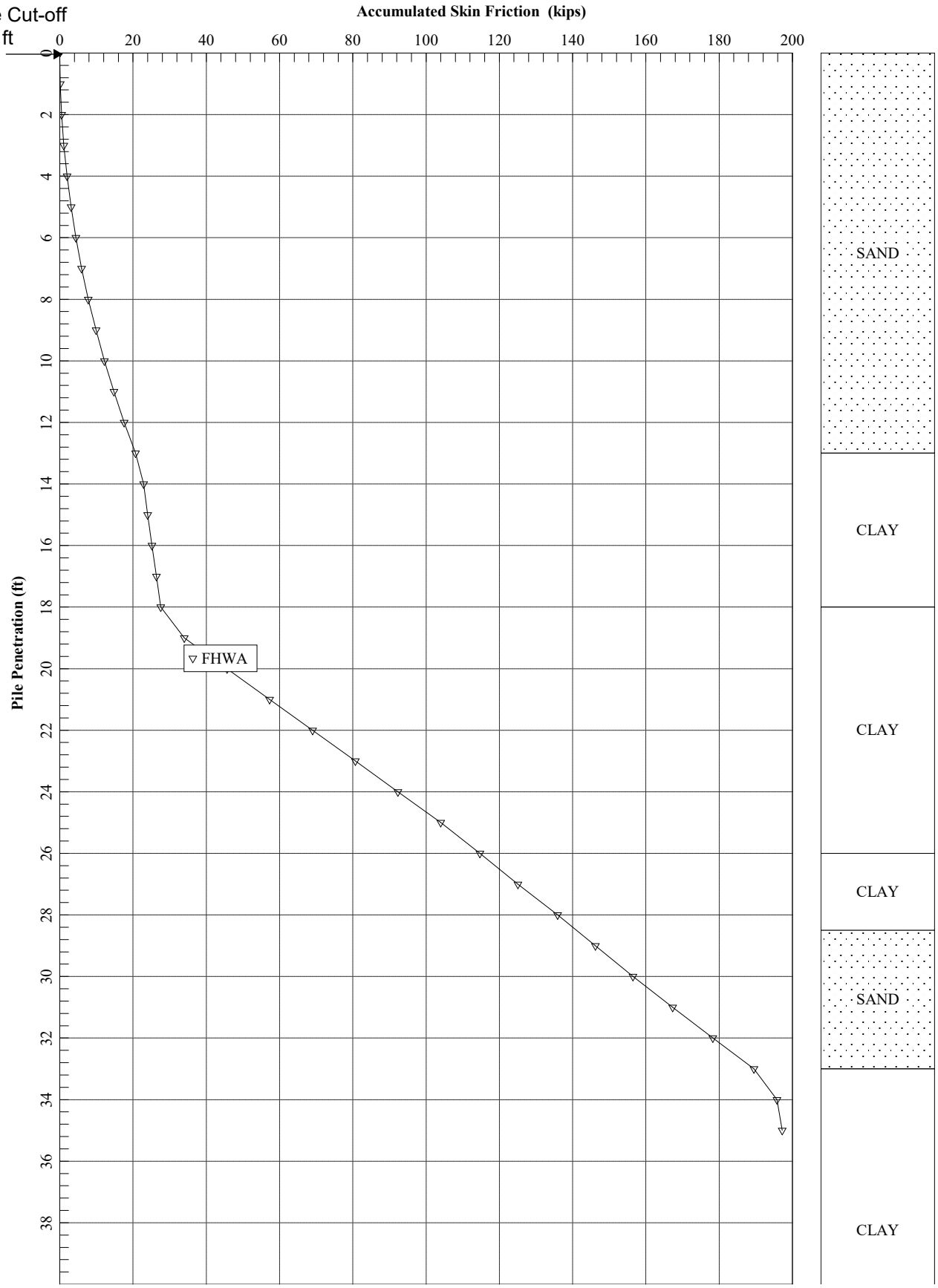


South Abutment - 15-Inch Timber Pile

Five Wounds Trail - Trestle Evaluation

Boring B-2
(Elev. = 106.0 ft)

Assumed Pile Cut-off
Elev. = 101.0 ft



North Abutment - 15-Inch Timber Pile

Appendix C

Draft Hydraulic Analysis Technical Memorandum

DRAFT Technical Memorandum

November 22, 2022

To	City of San Jose	**Contact No.**	805-858-3133
Copy to	Steve Grupico, PE	**Email**	seth.stevens@ghd.com
From	Seth Stevens, PE, CFM	**Project No.**	12575434
Project Name	PW 9966 - TRAIL: Five Wounds Trail - Trestle Evaluation		
Subject	Draft Hydraulic Analysis		

1. Introduction

The City of San Jose (City) is evaluating the structural integrity of an existing railroad trestle bridge over Coyote Creek that falls within the preliminary alignment of the Five Wounds Trail located in central San Jose, north of the intersection of Story Road and Senter Road (see Attachment 1).

1.1 Purpose of this Memorandum

This Technical Memorandum (TM) summarizes the methods and results of a hydraulic analysis of Coyote Creek that was conducted as part of the evaluation of the existing bridge. The key objectives of the analysis are:

- Provide a hydraulic assessment to estimate the water surface profile, flow depth, and flow velocity in Coyote Creek in the vicinity of the existing bridge
- Provide an evaluation of the scour at the existing bridge

1.2 Scope and limitations

This technical memorandum has been prepared by GHD for the City of San Jose. It is not intended for circulation or incorporation into other documents. The matters discussed in this memorandum are limited to those specifically detailed in the memorandum and are subject to any limitations or assumptions specially set out.

Accessibility of documents

If this Technical Memorandum is required to be accessible in any other format this can be provided by GHD upon request and at an additional cost if necessary.

This Technical Memorandum is provided as an interim output under our agreement with City of San Jose. It is provided to foster discussion in relation to technical matters associated with the project and should not be relied upon in any way.

2. Sources of Data

2.1 Vertical Datum

This study references the North American Vertical Datum of 1988 (NAVD88) in units of feet and all elevations presented herein are based on that datum.

2.2 Topographic Data

Two sources of topographic data were used for the hydraulic modeling. The first source was ground survey data collected by GHD's survey crew. Detailed bridge geometry information was collected including three-dimensional scans which were used to develop cross-sections of the bridge with pier elevations, pier widths, deck elevation, deck width, deck thickness, and abutment elevations. This data was used to update the existing bridge geometry within the hydraulic model.

The second source of topographic data was a digital elevation model (DEM) downloaded from NOAA Digital Coast: Data Access Viewer. The DEM was developed for Santa Clara County using LiDAR data collected in 2020. This data was used to develop additional cross-sections in the hydraulic model within the project study area.

2.3 Hydraulic Model

The Santa Clara Valley Water District's (SCVWD) existing steady-flow one-dimensional HEC-RAS model for Coyote Creek was used as the basis for the hydraulic modeling. The model extends from Anderson Dam to the San Francisco Bay and was calibrated to a 2017 flood event. SCVWD periodically updates the model and the model used for this study was provided to GHD in September 2022.

2.4 Flow Data

The flow rates used for the analysis were the 100-year design peak flow rates for Coyote Creek provided by SCVWD in September 2022. These flows are based on the 2020 SCVWD Design Flows Manual and vary along the channel. The flow rate at the study area is 13,430 cubic feet per second (cfs).

2.5 Geotechnical Data

Geotechnical data for the scour analysis was provided by Parikh Consultants, Inc. and consisted of a grain size distribution for a bulk sample that was collected in the creek bed (Parikh Consultants, Inc., 2022). The values from the grain size distribution used for the scour analysis are shown in Table 1.

Table 1 Creek Bed Material Diameters used for Scour Analysis

D_{50}	D_{95}
6.1 mm	50 mm

3. Hydraulic Assessment

3.1 Hydraulic Modeling Overview

Hydraulic modeling for Coyote Creek was completed as part of this study to estimate the water surface profile, flow depth, and flow velocity in Coyote Creek in the vicinity of the existing bridge. The modeling was conducted

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using the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS) version 6.2 software.
Hydraulic Model Setup

3.1.1 Base Model Geometry

The base model used for this study is SCVWD's Coyote Creek model as discussed in Section 2.3. The cross-sections are georeferenced using the California State Plane Coordinate System Zone 3. The vertical datum is NAVD88. The model is run with a subcritical flow regime and a downstream boundary condition of known water surface elevation of 7.64 feet (NAVD88), which is the recorded mean higher-high water at the Coyote Creek tide gauge (Station ID 9414575). The geometry file "Anderson to Bay – Perc Pond Gates Open; Coyote-AndersontoBay.g05" was used as the basis for the model development.

3.1.2 Existing Conditions Geometry

A new geometry file was created for the existing conditions by making the following changes to the Base Model geometry:

1. Four cross-sections were added to the model. These include 150394.6 and 105512 downstream of the bridge and 105693.4 and 105778 upstream of the bridge. The geometries for these cross-sections were obtained from the 2020 LiDAR DEM described in Section 2.2. These cross-sections are shown in Attachment 2.
2. The geometries for the cross-sections immediately downstream and upstream of the bridge (105590 and 105612, respectively) were updated using the 2020 LiDAR DEM described in Section 2.2.
3. The railroad trestle bridge geometry was revised to reflect the survey data collected in the field.

The resulting geometry file developed for the existing conditions is "Existing; Coyote-AndersontoBay.g01".

3.2 Model Results

The model was run with the existing conditions geometry using the full model domain. The run was conducted with a subcritical flow regime and the downstream boundary condition described in Section 3.1.1. A summary of the modeling results for existing conditions are shown in Table 1 and the water surface profile is shown in Figure 1. The average flow velocity and maximum depth of flow at the upstream bridge cross-section were estimated to be 2.2 feet per second and 33.9 feet, respectively.

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Table 2 Summary of Hydraulic Model Results for Flow Rate of 13,430 cfs

Channel Station	Water Surface Elevation (ft)	Max. Depth of Flow (ft)	Avg. Velocity of Flow (ft/s)
106500	106.02	34.0	1.4
106200	106.02	34.9	1.4
105900	105.98	35.2	1.7
105778	105.91	33.8	2.2
105693.4	105.84	33.8	3.0
105612	105.87	33.9	2.2
105601	Trestle Bridge		
105590	105.84	34.3	2.3
105512	105.83	34.6	2.2
105394.6	105.78	34.9	2.4
105300	105.77	37.0	2.5
105000	105.78	37.2	1.7

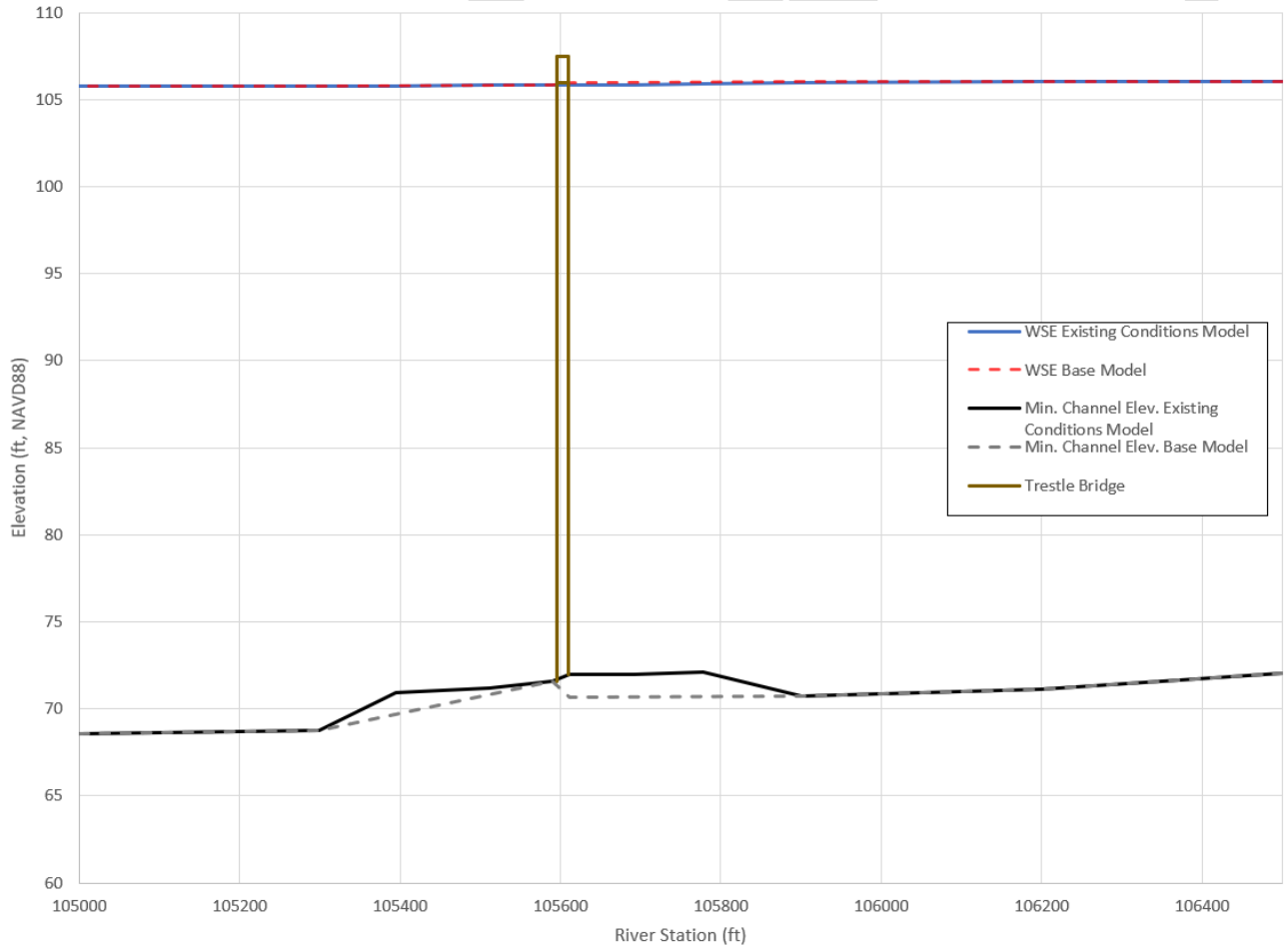


Figure 1 Coyote Creek Water Surface Profile at Trestle Bridge

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4. Bridge Scour Analysis

4.1 Basis of Scour Analysis

Preliminary bridge scour analyses were conducted as part of this study and included two primary components: general scour due to contraction and bends in the channel, and local scour at the piers or abutments. The sum of these components represents the total scour at the bridge. The analysis was performed using the results of the hydraulic modeling discussed in Section 3. Different flow scenarios and downstream boundary conditions in the hydraulic model could result in more or less scour at the bridge and the sensitivity of scour to these variables was not evaluated as part of this study.

The bridge scour analysis was conducted for the existing condition based on methods presented in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 *Evaluating Scour at Bridges* (FHWA, 2012), commonly referred to as HEC-18, and methods presented in NRCS Technical Supplement 14B *Scour Calculations* (NRCS, 2007).

4.2 Long-Term Bed Elevation Change

Long-term bed elevation change, as it relates to scour, is due to degradation of the channel bed as it tends toward an equilibrium slope. Historical channel elevation data at the project site could inform the potential for long-term degradation in the area. However, long-term bed elevation change was not evaluated as part of this study.

4.3 Clear-water versus Live-Bed Scour

Clear-water scour and live-bed scour are two methods by which contraction and pier scour occur. Clear-water scour occurs where there is no transport of bed material from upstream of the bridge while live-bed scour occurs where there is transport of bed material from upstream. The type of scour which occurs is dependent on the bed material grain size, upstream average velocity, and upstream average depth of flow. The critical velocity, V_C, necessary for transport of the bed material median diameter, D_{50}, is used as an indicator for clear-water or live-bed scour conditions. Clear-water scour is assumed to occur when the average velocity, V, upstream of the bridge is less than or equal to V_C for the D_{50} of the bed material. Live-bed scour is assumed to occur if V is greater than V_C. The critical velocity was calculated using equation 6.1 from HEC-18:

$$V_C = K_u y^{1/6} D^{1/3}$$

Where:

$V_C = Critical\ velocity\ above\ which\ bed\ material\ of\ size\ D\ and\ smaller\ will\ be\ transported\ \left(\frac{ft}{s}\right)$

$y = Average\ depth\ of\ flow\ upstream\ of\ the\ bridge\ (ft)$

$D = Particle\ size\ for\ V_C\ (ft)$

$D_{50} = Particle\ size\ in\ a\ mixture\ of\ which\ 50\ percent\ are\ smaller\ (ft)$

$K_u = 11.17\ English\ units$

The critical velocity was calculated for the channel bed material (D_{50}=6.1 mm) just upstream of the bridge and it was found that the channel velocity did not exceed the critical velocity for particles of that size, so clear-water scour was used for the analysis. The clear-water versus live-bed scour calculation is included in Attachment 4.

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4.4 General Scour

General scour refers to all types of scour that are not local, and includes contraction and bend scour. General scour was calculated using the Lacey and Blench equations presented NRCS, 2007:

$$z_t = KQ_d^a W_f^b D_{50}^C$$

Where:

$$z_t = \ maximum\ scour\ depth\ at\ cross-section\ (ft)$$
$$K = coefficient\ from\ table$$
$$Q_d = design\ discharg\ ((ft^3/s)$$
$$W_f = \ flow\ width\ at\ design\ discharge\ (ft)$$
$$D_{50} = \ median\ size\ of\ bed\ material\ (mm)$$

$$a, b, c = exponents\ from\ table$$
$$K_u = 0.0077\ (English\ Units)$$

General scour calculations were performed at the cross-sections immediately upstream and downstream of the bridge. The calculations are included in Attachment 4 and show a maximum scour depth of 5.8 feet.

The bridge is located on a sharp bend in the channel, so additional analysis of bend scour was conducted as part of this study. Bend scour was calculated in the vicinity of the bridge using the standard bend scour equation in NRCS, 2007. That equation gave calculated bend scour depths ranging from approximately 45 to 65 feet, which did not seem reasonable and thus were not included in this TM. Additional investigation into bend scour is planned to be conducted prior to finalizing this TM.

4.5 Pier Scour

Local scour at bridge piers occurs due to the formation of vortices at the base of the piers which causes the flow to accelerate in that area, resulting in increased sediment transport. The magnitude of pier scour is dependent on the flow velocity, flow depth, pier width, size and gradation of bed material, pier shape, and other factors. Pier scour increases with increased flow velocity, flow depth, and pier width.

Pier scour depth was calculated at each bent using the built-in scour module in HEC-RAS using the HEC-18 Pier Scour Equation (equation 7.1), which is based on the Colorado State University (CSU) equation, for both clear-water and live-bed scour:

$$\frac{y_s}{y_1} = 2\ K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

Where:

$$y_s = Scour\ depth\ (ft)$$
$$y_1 = Flow\ depth\ depth\ directly\ upstream\ of\ the\ pier\ (ft)$$
$$K_1 = Correlation\ factor\ of\ pier\ nose\ shape$$
$$K_2 = Correlation\ factor\ of\ angle\ of\ attack\ of\ flow$$
$$K_3 = Correlation\ factor\ for\ bed\ condition$$
$$a = Pier\ width\ (ft)$$
$$Fr_1 = Froude\ Number\ directly\ upstream\ of\ the\ pier = \ V_1/(gy_1)^{1/2}$$
$$V_1 = Mean\ velocity\ of\ flow\ directly\ upstream\ of\ the\ pier\ (ft/s)$$
$$g = Acceleration\ due\ to\ gravity = 32.2\ (ft/s^2)$$

The pier scour calculations are included in Attachment 4 and the calculated scour depths are shown in Table 3.

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Table 3 Calculated Pier Scour Depths

Support Location	Pier Scour Depth (ft)
Bent 2	2.4
Bent 3	2.7
Bent 4	2.7
Bent 5	2.2
Bent 6	2.2
Bent 7	3.0
Bent 8	3.0
Bent 9	3.2
Bent 10	2.1
Bent 11	2.1
Bent 12	2.5
Bent 13	1.9

4.6 Abutment Scour

Scour occurs at bridge abutments when they obstruct the flow of the channel, causing the flow to accelerate locally. Abutment scour depth depends on the shape of the abutment, the flow rate in the main channel at the abutment, the amount of flow obstructed, bed material characteristics, the bathymetry of the channel, and the alignment of the abutment in relation to the channel. Abutment scour was calculated using the built-in scour module in HEC-RAS using Froehlich's Abutment Scour Equation given in the HEC-18 manual (equation 8.1):

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

Where:

$K_1 = Coefficient\ for\ abutment\ shape$
$K_2 = Coefficient\ for\ angle\ of\ embankment\ to\ flow$
$K_2 = (\theta/90)^{0.13}$
$\theta < 90°\ if\ embankment\ points\ downstream$
$\theta > 90°\ if\ embankment\ points\ upstream$
$L' = Length\ of\ active\ flow\ obstructed\ by\ the\ embankment\ (ft)$
$A_e = Flow\ area\ of\ the\ approach\ cross\ section\ obstructed\ by\ the\ embankment\ (ft^2)$
$Fr = Froude\ Number\ of\ approach\ flow\ upstream = V_e/(gy_a)^{0.5}$
$V_e = Q_e/A_e\ (ft/s)$
$Q_e = Flow\ obstructed\ by\ the\ abutment\ and\ approach\ embankment\ (ft^3/s)$
$y_a = Average\ depth\ of\ flow\ on\ the\ floodplain\ (A_e/L)\ (ft)$
$L = Length\ of\ embankment\ projected\ normal\ to\ the\ flow\ (ft)$
$y_s = Scour\ depth\ (ft)$

The abutment scour calculations are included in Attachment 4 and show a scour depth of 2.6 feet at Abutment 1 on the left bank (looking downstream) and 9.1 feet of scour at Abutment 14 on the right bank (looking downstream).

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4.7 Total Scour

Total scour was calculated as part of this study as the sum of the general, pier, and abutment scour. The results are shown in Table 4.

Table 4 Total Calculated Scour Depths

Support Location	General Scour Depth (ft)	Pier Scour Depth (ft)	Abutment Scour Depth (ft)	Total Scour Depth (ft)
Abutment 1 (Left Bank)	5.8	0.0	2.6	8.4
Bent 2	5.8	2.4	0.0	8.1
Bent 3	5.8	2.7	0.0	8.4
Bent 4	5.8	2.7	0.0	8.5
Bent 5	5.8	2.2	0.0	7.9
Bent 6	5.8	2.2	0.0	8.0
Bent 7	5.8	3.0	0.0	8.8
Bent 8	5.8	3.0	0.0	8.8
Bent 9	5.8	3.2	0.0	9.0
Bent 10	5.8	2.1	0.0	7.9
Bent 11	5.8	2.1	0.0	7.8
Bent 12	5.8	2.5	0.0	8.2
Bent 13	5.8	1.9	0.0	7.6
Abutment 14 (Right Bank)	5.8	0.0	9.1	14.9

5. References

FHWA. (2012). *Hydraulic Engineering Circular No. 18 Evaluating Scour at Bridges, Fifth Edition.*

NRCS. (2007). National Engineering Handbook Technical Supplement 14B Scour Calculations.

Parikh Consultants, Inc. (2022). *Geotechnical Investigation Report Five Wounds Trail - Trestle Evaluation San Jose, CA.*

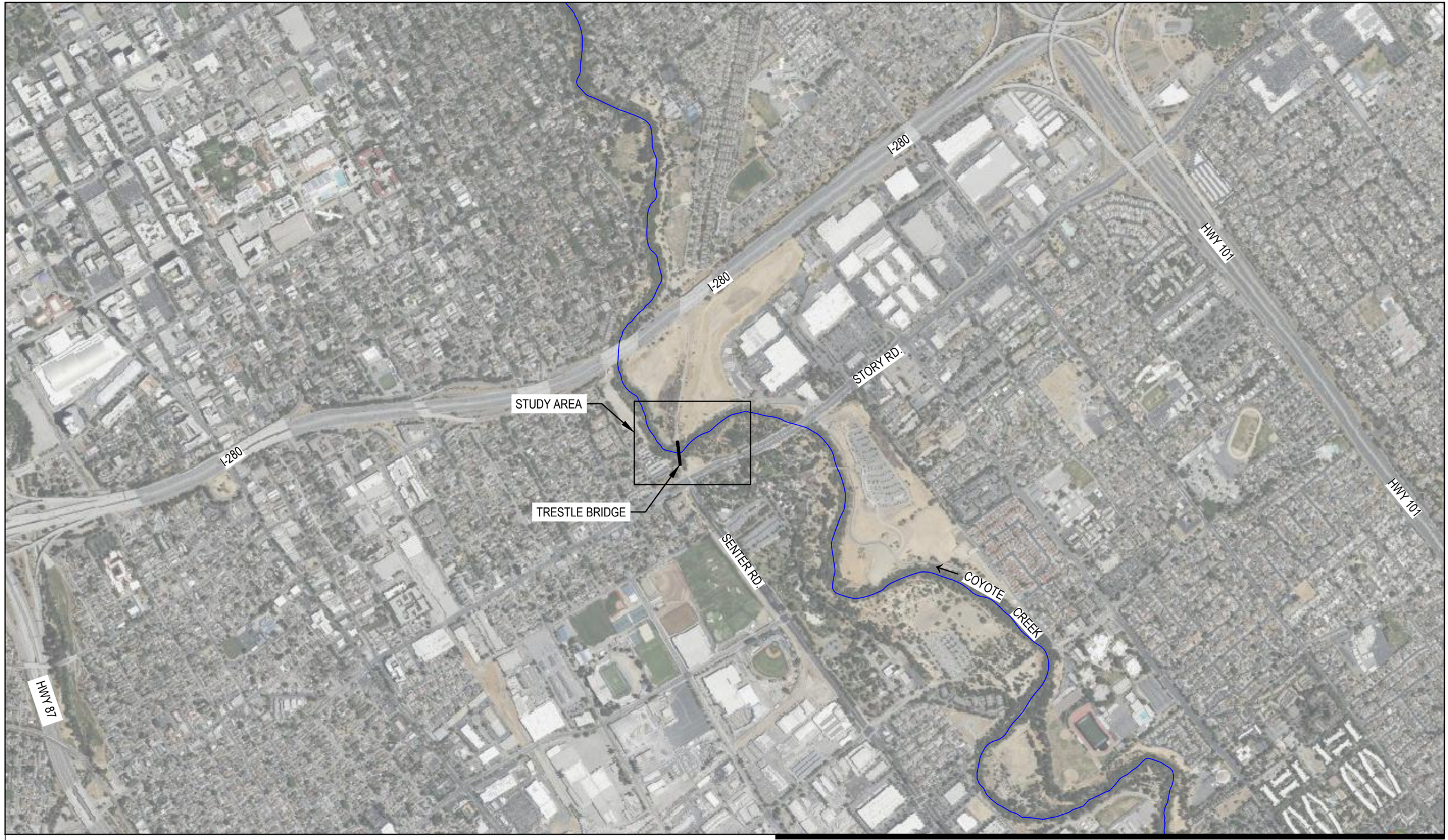
DRAFT

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Attachments

Attachment 1

Vicinity Map



N

0 1000' 2000'

GHD

CITY OF SAN JOSE
FIVE WOUNDS TRAIL - TRESTLE EVALUATION

VICINITY MAP

Project No. **12575434**
Date **November 22**

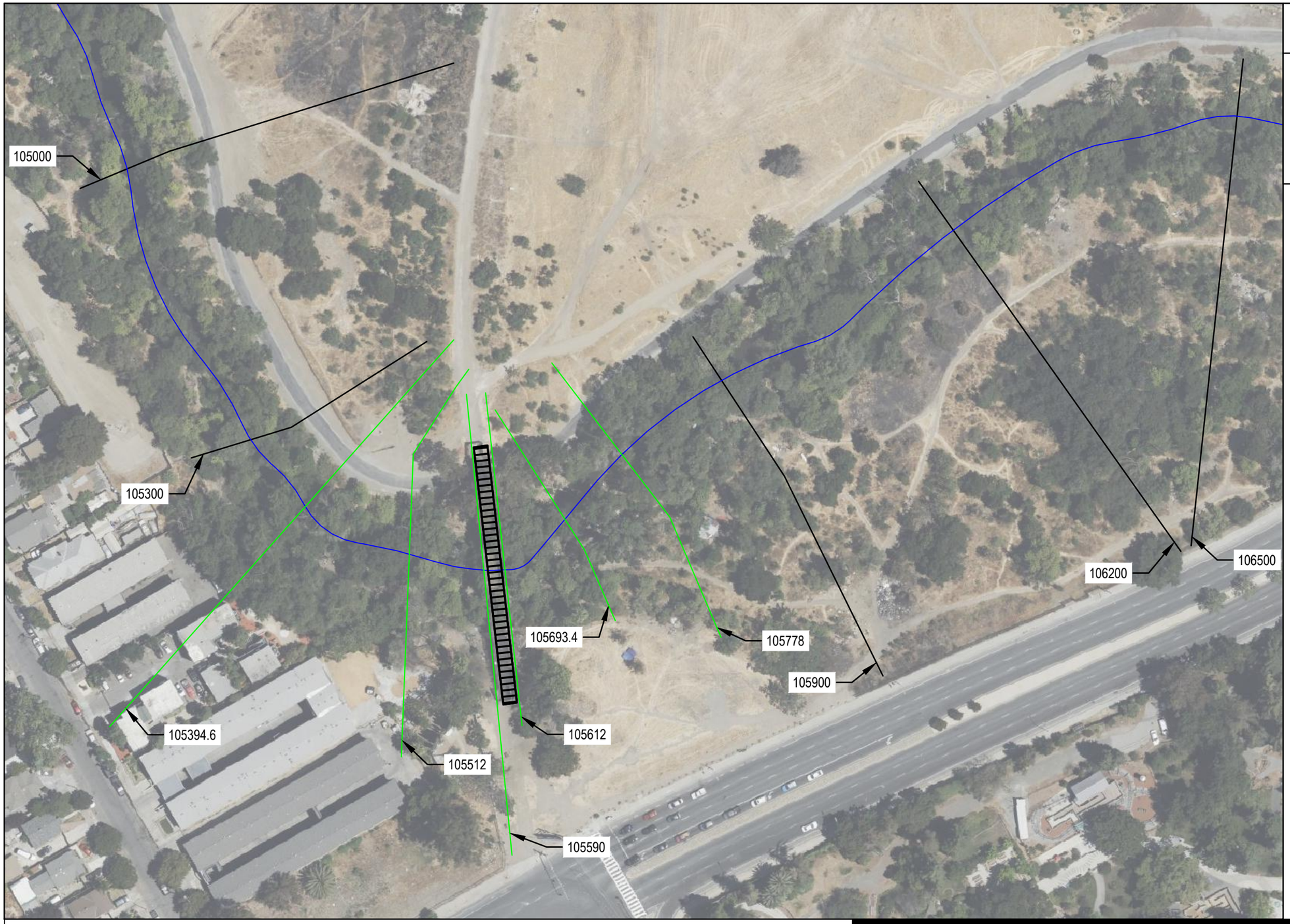
FIGURE 1

Data Source

Filename: \\ghdnet\ghd\US\San Francisco\Projects\561\12575434\Digital_Design\ACAD\Figures\12575434-FIGURE 1-HYDRAULIC WORK MAP.dwg
Plot Date: 18 November 2022 8:21 AM

Attachment 2

Hydraulic Work Map



LEGEND

- COYOTE CREEK
- NEW CROSS SECTIONS
- VALLEY WATER CROSS SECTIONS
- TRESTLE BRIDGE

N

0 100' 200'

GHD

CITY OF SAN JOSE
FIVE WOUNDS TRAIL - TRESTLE EVALUATION

HYDRAULIC WORK MAP

Project No. **12575434**
Date **November 22**

FIGURE 2

Data Source

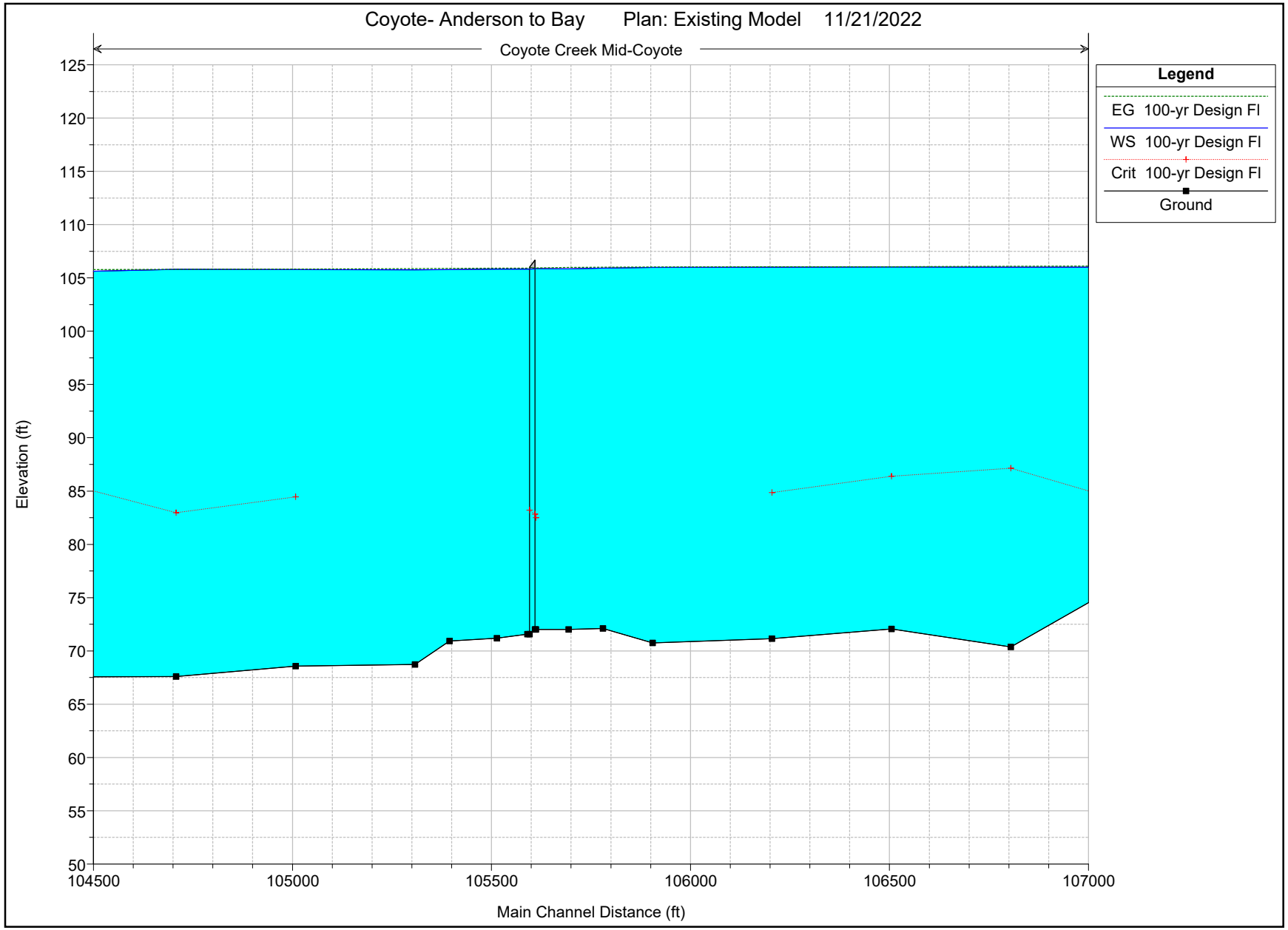
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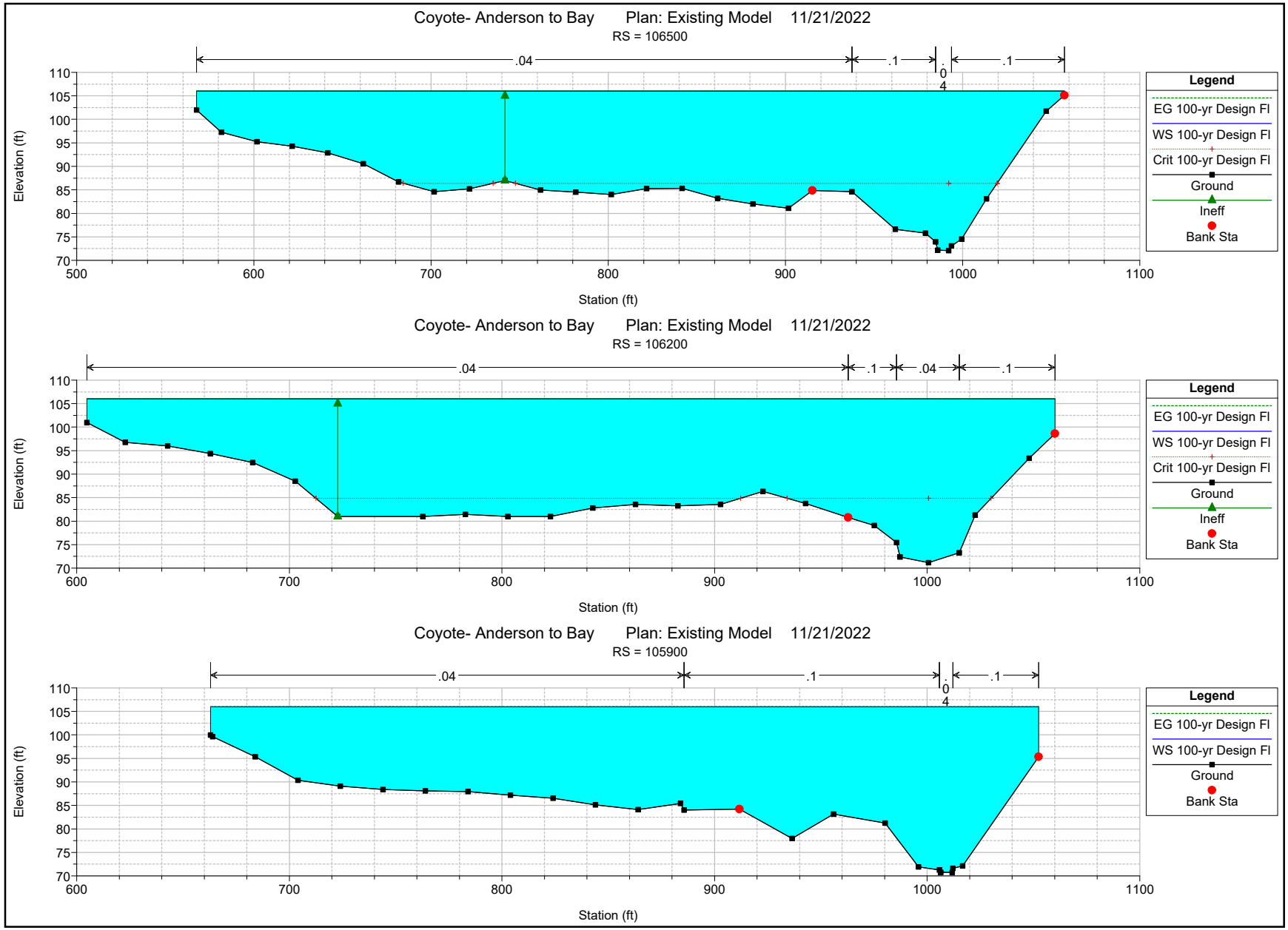
Attachment 3

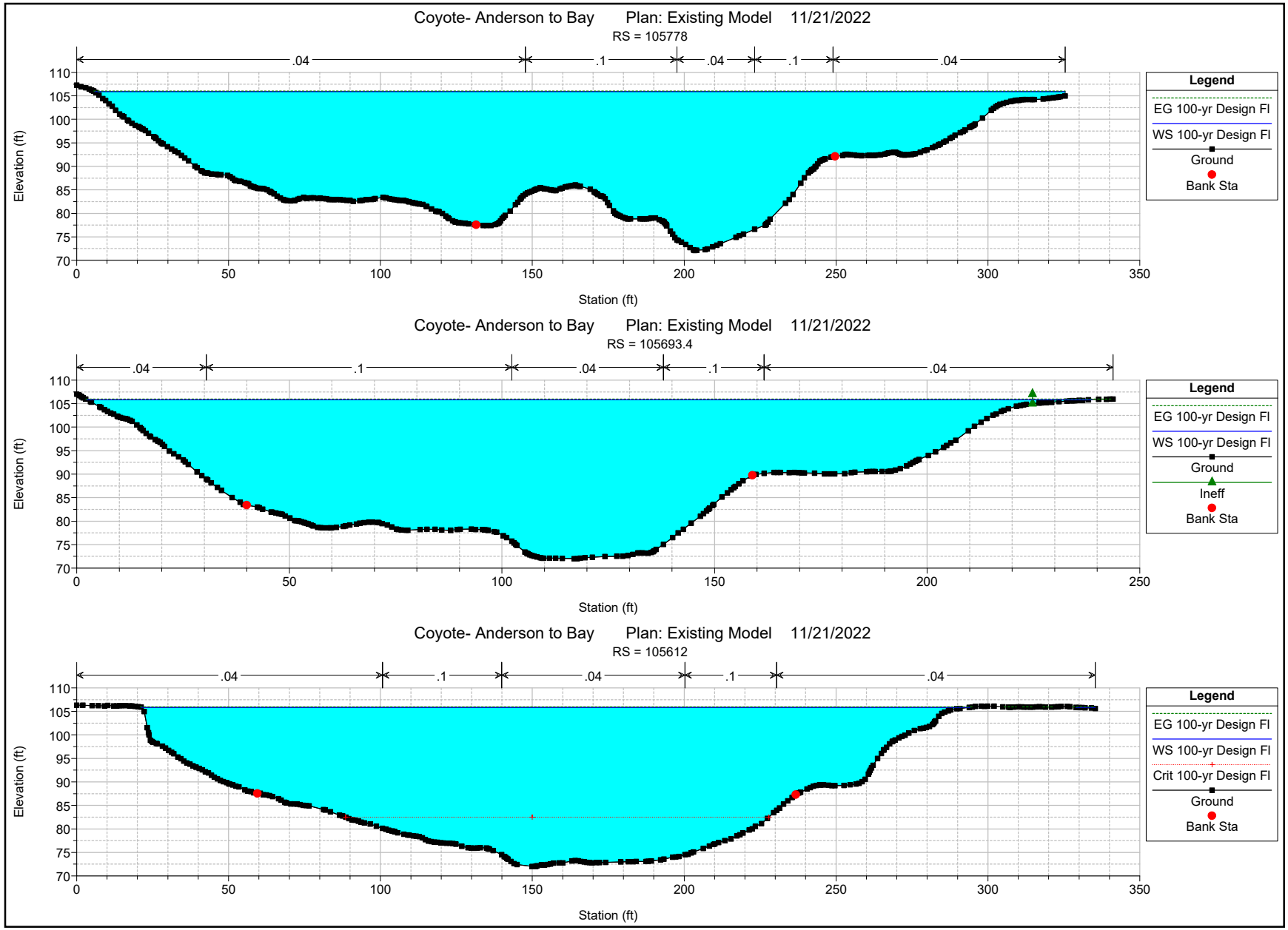
Hydraulic Model Results

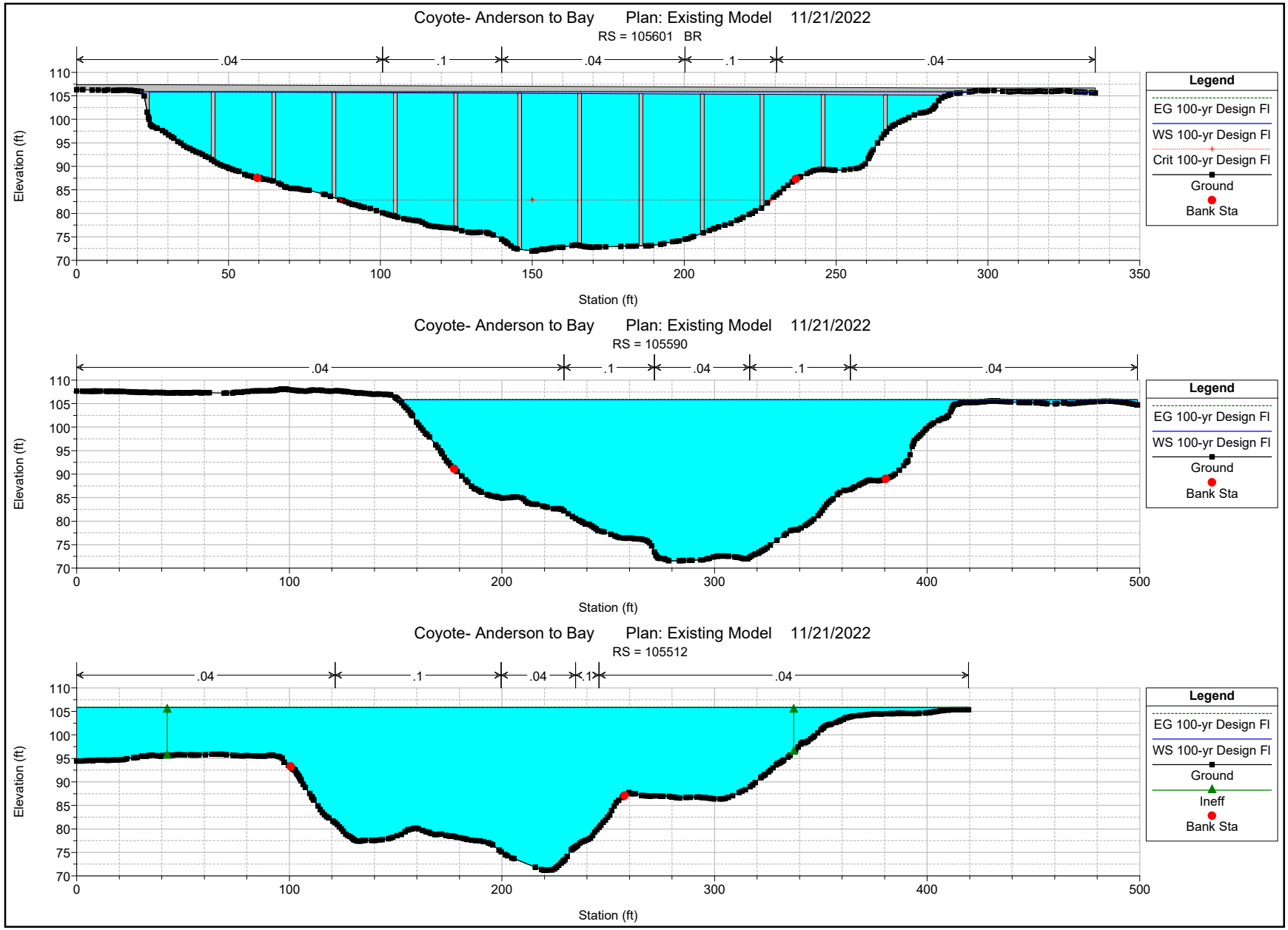
HEC-RAS Plan: Existing Locations: User Defined Profile: 100-yr Design Fl

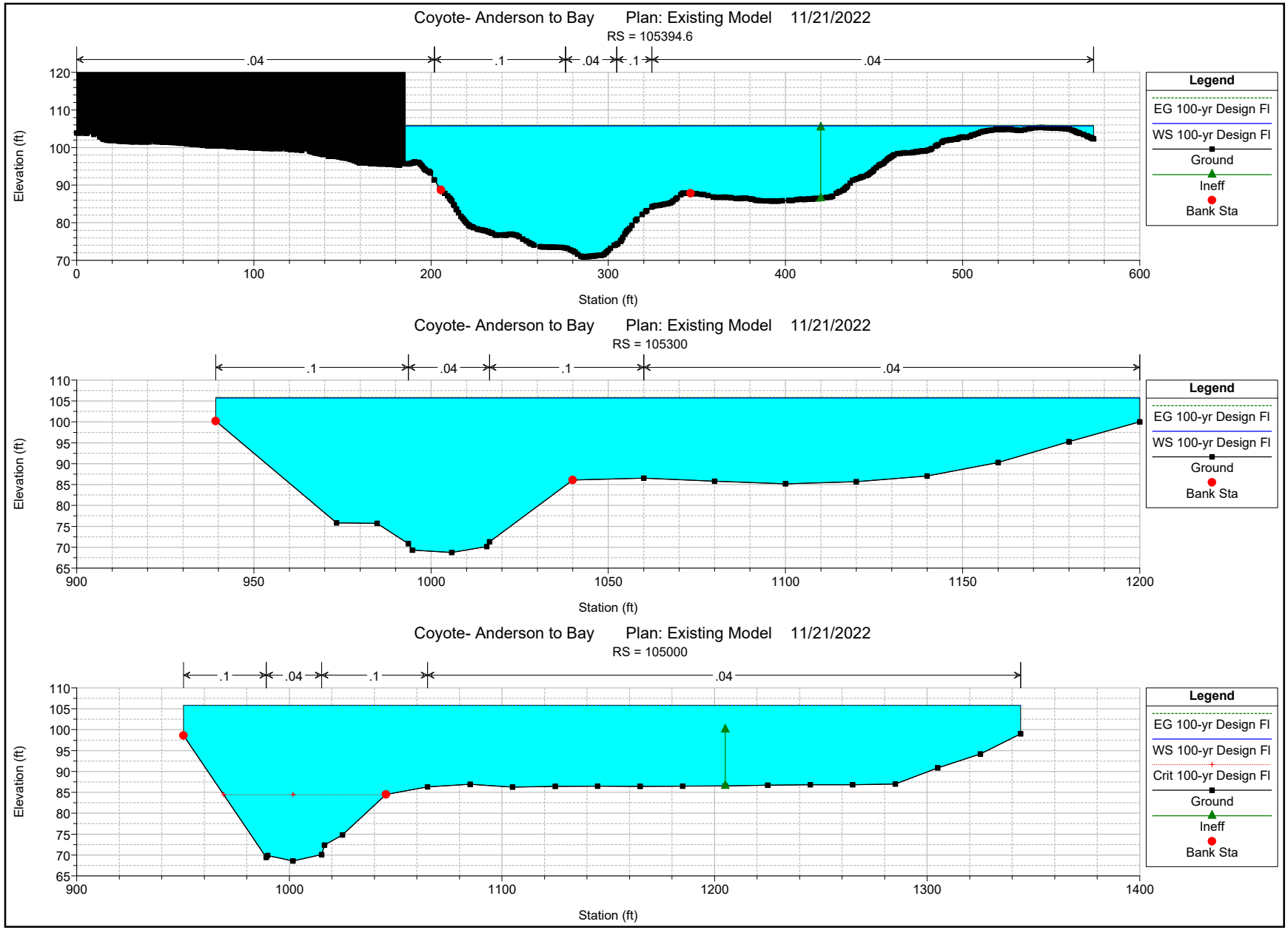
River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Coyote Creek	Mid-Coyote	106500	100-yr Design Fl	13430.00	72.06	106.02	86.40	106.06	0.000043	0.82	9569.15	489.84	0.03
Coyote Creek	Mid-Coyote	106200	100-yr Design Fl	13430.00	71.15	106.02	84.86	106.05	0.000035	0.79	9612.61	455.34	0.03
Coyote Creek	Mid-Coyote	105900	100-yr Design Fl	13430.00	70.74	105.98		106.03	0.000086	1.12	8096.75	389.40	0.04
Coyote Creek	Mid-Coyote	105778	100-yr Design Fl	13430.00	72.10	105.91		106.01	0.000140	1.74	5991.27	319.58	0.06
Coyote Creek	Mid-Coyote	105693.4	100-yr Design Fl	13430.00	72.01	105.84		105.98	0.000372	2.99	4494.42	236.35	0.10
Coyote Creek	Mid-Coyote	105612	100-yr Design Fl	13430.00	72.01	105.87	82.50	105.95	0.000131	2.30	6003.09	279.25	0.08
Coyote Creek	Mid-Coyote	105601		Bridge									
Coyote Creek	Mid-Coyote	105590	100-yr Design Fl	13430.00	71.57	105.84		105.92	0.000173	2.40	5850.30	347.56	0.08
Coyote Creek	Mid-Coyote	105512	100-yr Design Fl	13430.00	71.19	105.83		105.91	0.000132	1.95	6248.86	419.42	0.07
Coyote Creek	Mid-Coyote	105394.6	100-yr Design Fl	13430.00	70.92	105.78		105.89	0.000168	2.09	5559.68	388.58	0.07
Coyote Creek	Mid-Coyote	105300	100-yr Design Fl	13430.00	68.73	105.77		105.88	0.000198	1.85	5452.25	260.77	0.06
Coyote Creek	Mid-Coyote	105000	100-yr Design Fl	13430.00	68.58	105.78	84.46	105.83	0.000066	1.10	8044.14	393.75	0.04


Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
Coyote Creek Mid-Coyote
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Elevation (ft)
125
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
104500
105000
105500
106000
106500
107000
Main Channel Distance (ft)


Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 106500
.04
.1
.1
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Ineff
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 106200
.04
.1
.04
.1
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Ineff
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105900
.04
.1
.1
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Bank Sta


Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105778
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
50
100
150
200
250
300
350
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105693.4
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
50
100
150
200
250
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Ineff
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105612
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
50
100
150
200
250
300
350
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Bank Sta


Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105601 BR
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
50
100
150
200
250
300
350
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105590
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
100
200
300
400
500
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105512
.04
.1
.04
.1
.04
Elevation (ft)
110
105
100
95
90
85
80
75
70
0
100
200
300
400
500
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Ineff
Bank Sta


Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105394.6
.04
.1
.04
.1
.04
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Ineff
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105300
.1
.04
.1
.04
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Ground
Bank Sta
Coyote- Anderson to Bay Plan: Existing Model 11/21/2022
RS = 105000
.1
.04
.1
.04
Elevation (ft)
Station (ft)
Legend
EG 100-yr Design Fl
WS 100-yr Design Fl
Crit 100-yr Design Fl
Ground
Ineff
Bank Sta

Attachment 4

Bridge Scour Calculations

Live-Bed vs Clear-Water Scour Determination

Project Name: Five Wounds Trail - Trestle Evaluation
Project No.: 12575434
Updated: 11/18/2022
Calc By: STS

The following calculations are based on the methods presented in FHWA HEC No. 18, Fifth Edition for calculating critical velocity.

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in the Appendix C. This equation is:

$$V_c = K_u y^{1/6} D^{1/3} \quad (6.1)$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y = Average depth of flow upstream of the bridge, ft (m)
D = Particle size for V_c, ft (m)
D_{50} = Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u = 6.19 SI units
K_u = 11.17 English units

Critical Velocity Calculation

Input Parameters

Ku: 11.17
Channel D_{50}: 6.1 mm
0.0200 ft
Overbank D_{50}: 6.1 mm
0.0200 ft

Flow Scenario	Flow (cfs)	Flow Area* (sf)	Top Width* (ft)	Avg. Flow Depth*, y (ft)	Critical Velocity, V_c (ft/s)	Channel Velocity*, V (ft/s)	Contraction Scour Type
FEMA FIS	13,430	6,003	279	21.5	5.06	2.24	Clear-Water

*Channel Velocity, Flow Area, and Top Width are from HEC-RAS output at x-section 105612 just upstream of bridge.

Project: Five Wounds Trail Trestle Bridge
Prj No: 12575434
Coyote Creek Scour Analysis
Date: 11/22/2022
Reference: National Engineering Handbook - Technical Supplement 14B

General Scour

General scour refers to all types of scour that are not local (fig. TS14B–3). General scour commonly, but not necessarily, occurs over the entire cross section, and may involve reaches of varying length depending on the type of scour and site-specific conditions. General scour includes contraction scour and bend scour.

Pemberton and Lara (1984) suggested that regime equations provided by Blench (1970) and Lacey (1931) could be used to predict general scour in natural channels. A designer may compute scour depth using both formulas, and average the outcome or take the largest value.

$$z_t = KQ_d^a W_f^b D_{50}^c \qquad \text{(eq. TS14B–23)}$$

where:

z_t = maximum scour depth at the cross section or reach in question, ft (m)
K = coefficient (table TS14B–8)
Q_d = design discharge, ft^3/s (m^3/s)
W_f = flow width at design discharge, ft (m)
D_{50} = median size of bed material (mm)
a, b, c = exponents (table TS14B–8)

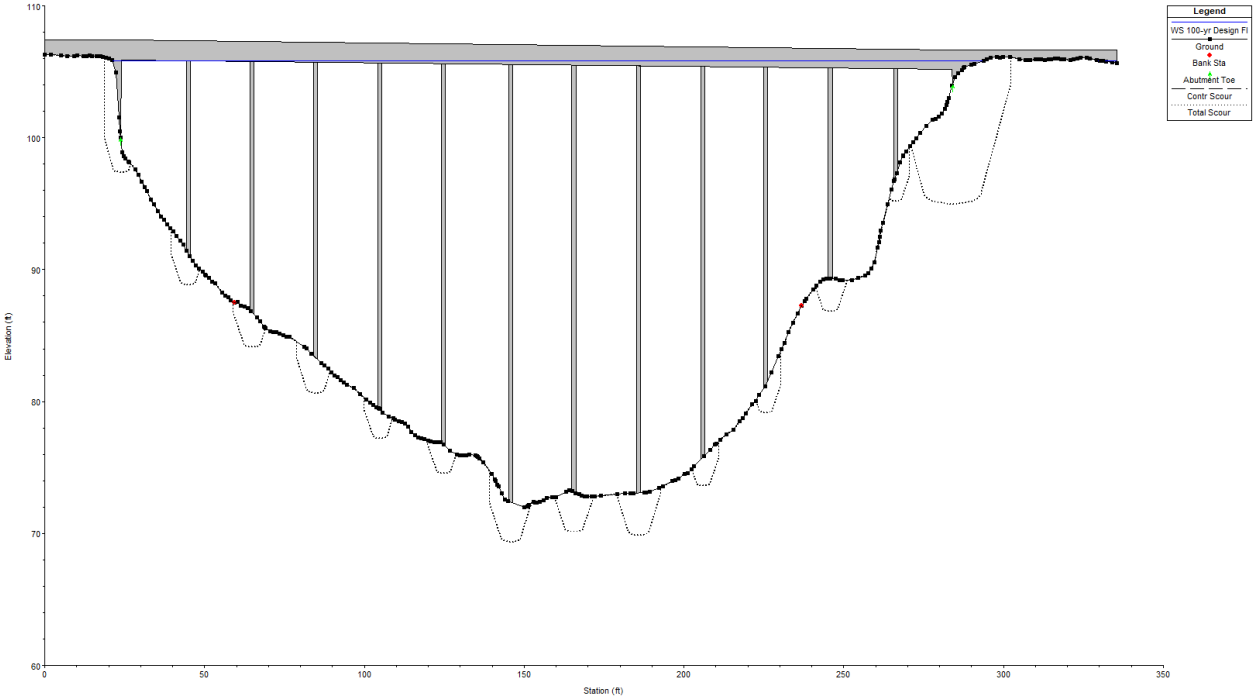
Table TS14B–8 Constants for Lacey and Blench relations, U.S. units (D_{50} in mm)

Condition	Lacey				Blench			
	K	a	b	c	K	a	b	c
Straight reach	0.097	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Moderate bend	0.195	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Severe bend	0.292	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Right angle bend	0.389	1/3	0	–1/6	1.105	2/3	–2/3	–0.1092
Vertical rock wall	0.487	1/3	0	–1/6				

Cross-Section	Q_d (cfs)	W_f (ft)	D_{50} (mm)	Lacey					Blench					Avg Z_t (ft)
				K	a	b	c	z_t (ft)	K	a	b	c	z_t (ft)	
105612	13,430	279.23	6.1	0.292	0.333	0	-0.167	**5.13**	0.530	0.667	-0.667	-0.109	**5.75**	**5.44**
105590	13,430	347.56	6.1	0.292	0.333	0	-0.167	**5.13**	0.530	0.667	-0.667	-0.109	**4.97**	**5.05**

Contraction, Pier, and Abutment Scour Calculation Output from HEC-RAS Model

Flow: 13,430 cfs



Hydraulic Design Data

Contraction Scour

	Left	Channel	Right
Input Data			
Average Depth (ft):	10.75	28.00	11.56
Approach Velocity (ft/s):	2.22	2.99	3.40
Br Average Depth (ft):	30.19		
BR Opening Flow (cfs):	1624.91	10314.63	1490.46
BR Top WD (ft):	14.95		
Grain Size D50 (mm):	6.10	6.10	6.10
Approach Flow (cfs):	897.44	9940.44	2592.12
Approach Top WD (ft):	37.61	118.90	65.86
K1 Coefficient:	0.590	0.590	0.590
Results			
Scour Depth Ys (ft):	0.00		
Critical Velocity (ft/s):	4.51		
Equation:	Clear		

Pier Scour

Pier: #2 (CL = 44.89)

Input Data

Pier Shape:	Circular cylinder
Pier Width (ft):	1.25
Grain Size D50 (mm):	6.10000
Depth Upstream (ft):	14.67
Velocity Upstream (ft/s):	2.04
K1 Nose Shape:	1.00
Pier Angle:	
Pier Length (ft):	20.00

K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.35
Froude #: 0.09
Equation: CSU equation

Pier: #3 (CL = 64.89)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 19.10
Velocity Upstream (ft/s): 2.47
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.65
Froude #: 0.10
Equation: CSU equation

Pier: #4 (CL = 84.72)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 22.52
Velocity Upstream (ft/s): 2.47
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.71
Froude #: 0.09
Equation: CSU equation

Pier: #5 (CL = 104.89)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 26.49
Velocity Upstream (ft/s): 1.42
K1 Nose Shape: 1.00

Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.18
Froude #: 0.05
Equation: CSU equation

Pier: #6 (CL = 124.8)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 29.12
Velocity Upstream (ft/s): 1.42
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.21
Froude #: 0.05
Equation: CSU equation

Pier: #7 (CL = 145.8)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 33.50
Velocity Upstream (ft/s): 2.79
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 3.01
Froude #: 0.08
Equation: CSU equation

Pier: #8 (CL = 165.55)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 32.73

Velocity Upstream (ft/s): 2.79
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 3.00
Froude #: 0.09
Equation: CSU equation

Pier: #9 (CL = 185.76)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 32.80
Velocity Upstream (ft/s): 3.26
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 3.21
Froude #: 0.10
Equation: CSU equation

Pier: #10 (CL = 205.97)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 30.11
Velocity Upstream (ft/s): 1.29
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.13
Froude #: 0.04
Equation: CSU equation

Pier: #11 (CL = 225.6)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25

Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 24.64
Velocity Upstream (ft/s): 1.29
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.07
Froude #: 0.05
Equation: CSU equation

Pier: #12 (CL = 245.76)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 16.53
Velocity Upstream (ft/s): 2.23
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 2.49
Froude #: 0.10
Equation: CSU equation

Pier: #13 (CL = 266.21)

Input Data

Pier Shape: Circular cylinder
Pier Width (ft): 1.25
Grain Size D50 (mm): 6.10000
Depth Upstream (ft): 8.80
Velocity Upstream (ft/s): 1.39
K1 Nose Shape: 1.00
Pier Angle:
Pier Length (ft): 20.00
K2 Angle Coef: 1.00
K3 Bed Cond Coef: 1.10
Grain Size D90 (mm):
K4 Armouring Coef: 1.00
Set K1 value to 1.0 because angle > 5 degrees

Results

Scour Depth Ys (ft): 1.86
Froude #: 0.08
Equation: CSU equation

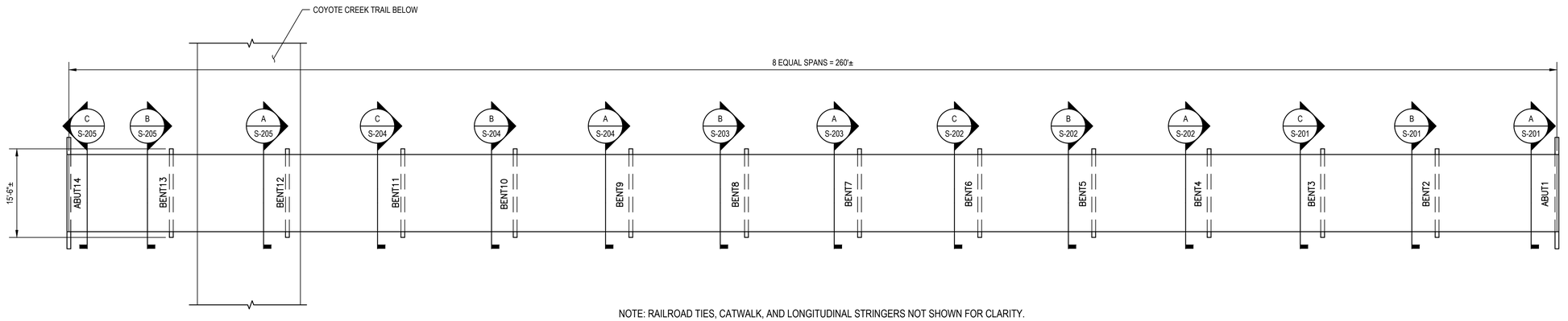
Abutment Scour

	Left	Right
Input Data		
Station at Toe (ft):	23.82	284.00
Toe Sta at appr (ft):	4.32	206.10
Abutment Length (ft):	3.03	32.64
Depth at Toe (ft):	5.88	1.79
K1 Shape Coef:	1.00 - Vertical abutment	
Degree of Skew (degrees):	90.00	90.00
K2 Skew Coef:	1.00	1.00
Projected Length L' (ft):	3.03	32.64
Avg Depth Obstructed Ya (ft):	1.43	2.64
Flow Obstructed Qe (cfs):	3.28	152.42
Area Obstructed Ae (sq ft):	4.33	86.05
Results		
Scour Depth Ys (ft):	2.61	9.10
Qe/Ae = Ve:	0.76	1.77
Froude #:	0.11	0.19
Equation:	Froehlich	Froehlich

Appendix D

Conceptual Sketches of Analysis Options 2 & 3

ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



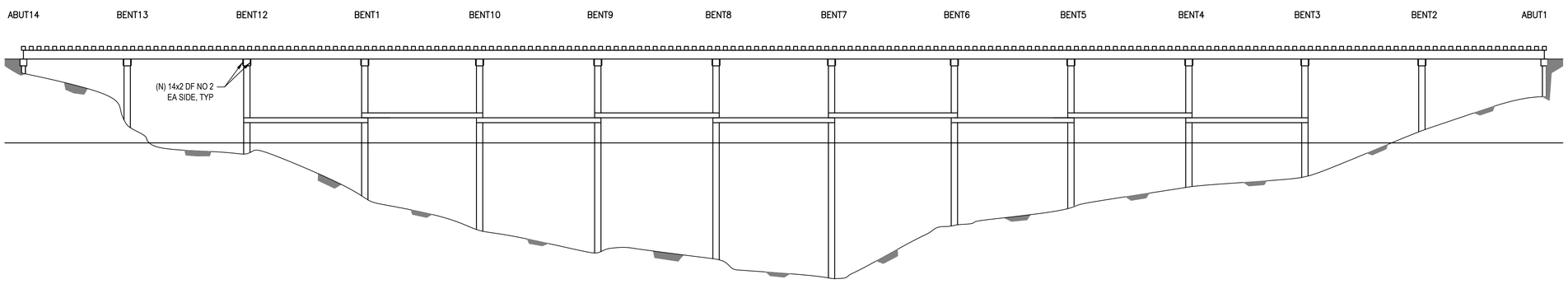
NOTE: RAILROAD TIES, CATWALK, AND LONGITUDINAL STRINGERS NOT SHOWN FOR CLARITY.

FIVE WOUNDS TRESTLE PLAN

SCALE 3/32" = 1'-0" (APPROXIMATE)

0 8' 16' 24'

N



FIVE WOUNDS TRESTLE ELEVATION

SCALE 3/32" = 1'-0" (APPROXIMATE)

0 8' 16' 24'

S-101

GHD Inc.
2235 Mercury Way Suite 150
Santa Rosa California 95407 USA
T 1 707 523 1010
www.ghd.com

9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

1 - OVERALL PLAN VIEW AND ELEVATION - ALT 1

	REVISIONS	INITIALS	DATE
6			
5			
4			
3			
2			
1			

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SANITARY
STORM
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SURVEY

CITY OF SAN JOSE CAPITAL OF SILICON VALLEY

DEPARTMENT OF PUBLIC WORKS
SAN JOSE, CALIFORNIA

DESIGNED BY S. BURNS
CHECKED BY B. CROWELL
PROJ MGR S. GRUPICO
DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 3 OF 8 SHEETS

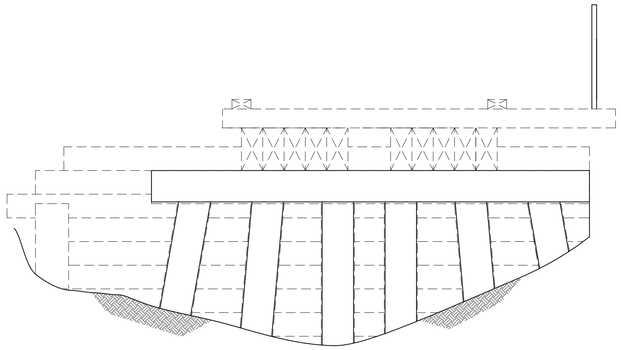
MATT CANO
DIRECTOR OF PUBLIC WORKS

9966

PRELIMINARY

Plot Date: 24 March 2023 - 3:39 PM Plotted By: Aly Pratt Cad File No. N:\US\San Francisco\Projects\56\11225754341Digital_Design\ACAD\Sheets\Report Alt 1 for report_Added Timber\11257543 4-GHD-0001-BRG-ST-0101.dwg

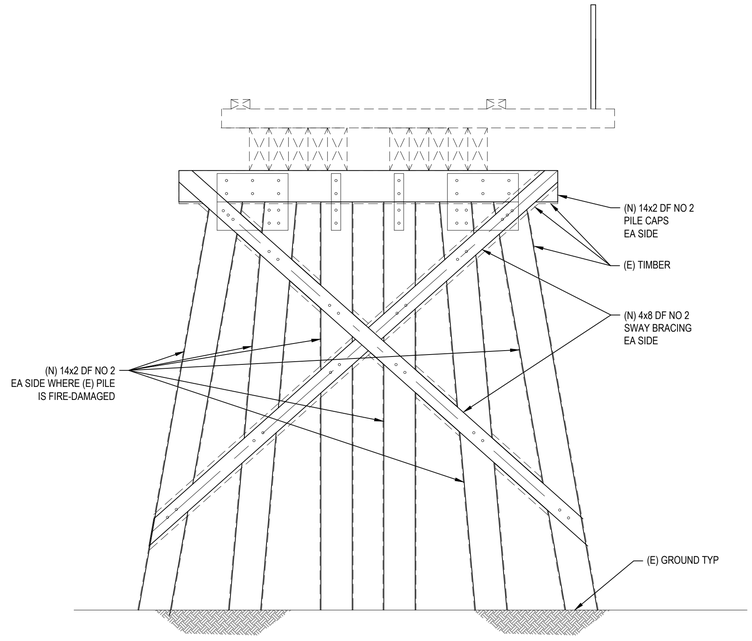
ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



1 ABUTMENT 1 ELEVATION - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

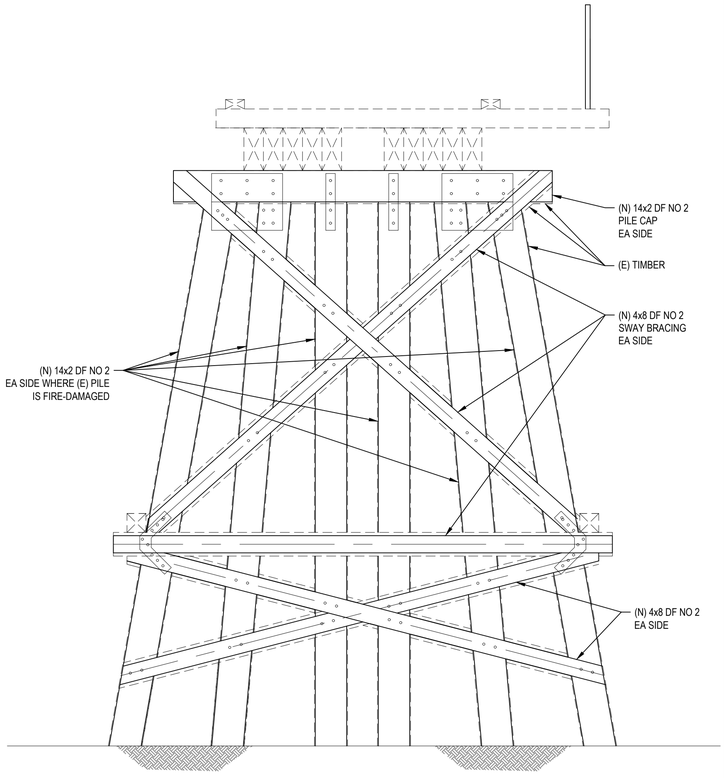
0 2' 4' 6'



A BENT 2 ELEVATION - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



B BENT 3 ELEVATION - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

S-201

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

BENT ELEVATIONS - ALT 1

	REVISIONS	INITIALS	DATE
6			
5			
4			
3			
2			
1			

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SURVEY

CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

DEPARTMENT OF PUBLIC WORKS
SAN JOSE, CALIFORNIA

DESIGNED BY S. BURNS
CHECKED BY B. CROWELL
PROJ MGR S. GRUPICO
DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 4 OF 8 SHEETS

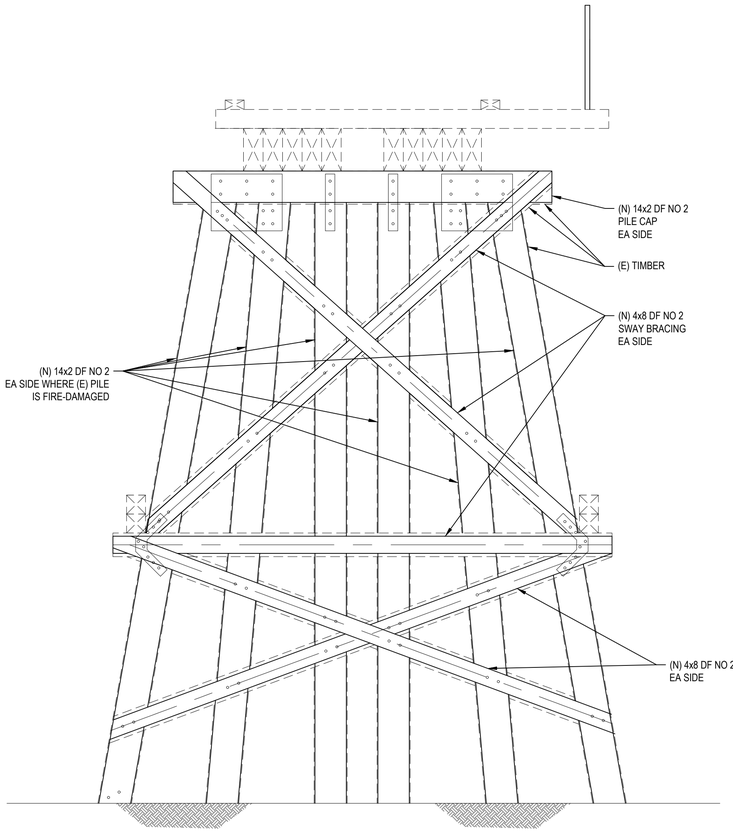
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DIRECTOR OF PUBLIC WORKS

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Plot Date: 24 March 2023 - 3:39 PM Plotted By: Aly Pratt Cad File No: N:\US\San Francisco\Projects\56111\25754341\Digital_Design\ACAD\Sheets\Report Alt 1 for report_Added Timber\12575434-GHD-0001-BRG-ST-0201.dwg

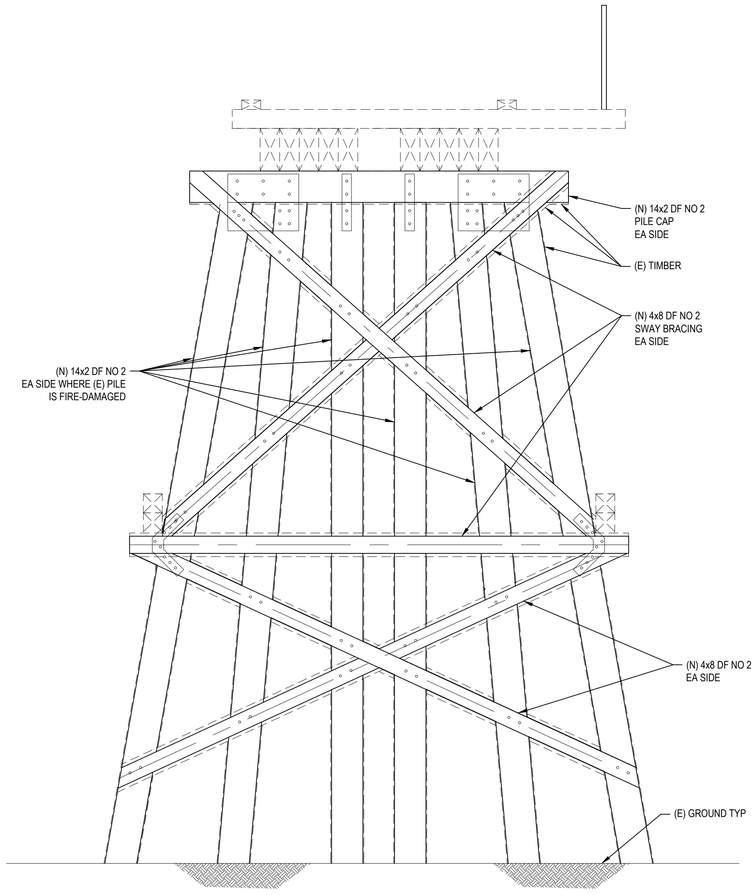
ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



A **BENT 4 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

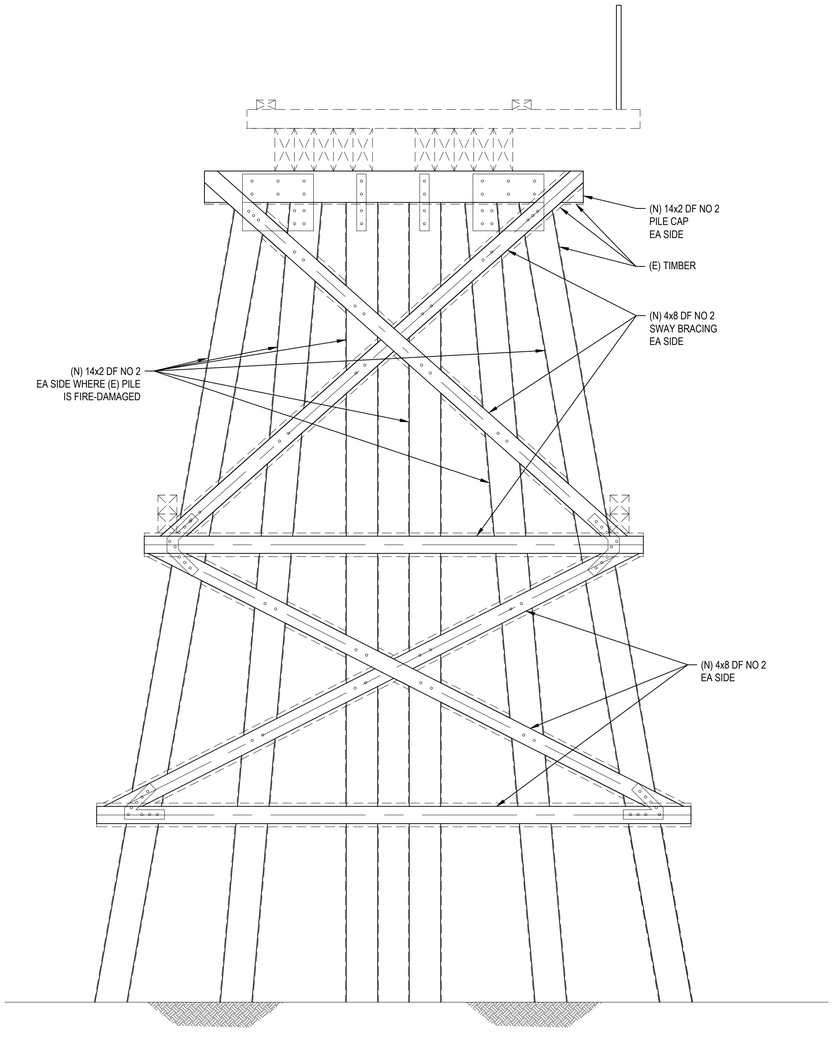
0 2' 4' 6'



B **BENT 5 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



C **BENT 6 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

BENT ELEVATIONS - ALT 1

	REVISIONS	INITIALS	DATE
6			
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TRAFFIC SIGNALS/SIGNAL OPS
SURVEY

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DEPARTMENT OF PUBLIC WORKS
SAN JOSE, CALIFORNIA

DESIGNED BY S. BURNS
CHECKED BY B. CROWELL
PROJ MGR S. GRUPICO
DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 5 OF 8 SHEETS

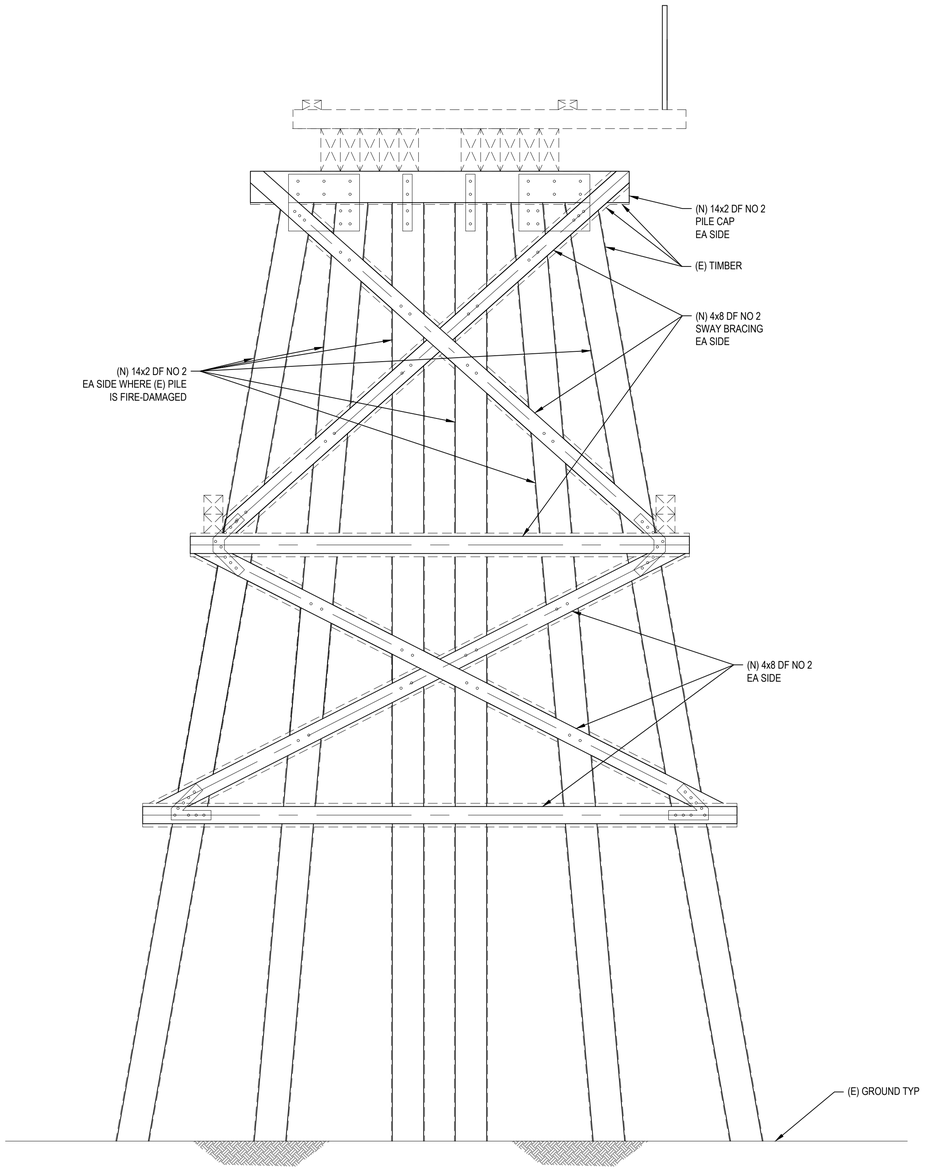
MATT CANO
DIRECTOR OF PUBLIC WORKS

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Plot Date: 24 March 2023 - 3:39 PM Plotted By: Aly Pratt Cad File No. N:\US\San Francisco\Projects\55\11225754\34\Digital_Design\ACAD\Sheets\Report Alt 1 for report_Added Timber\11257543 4-GHD-0001-BRG-ST-0201.dwg

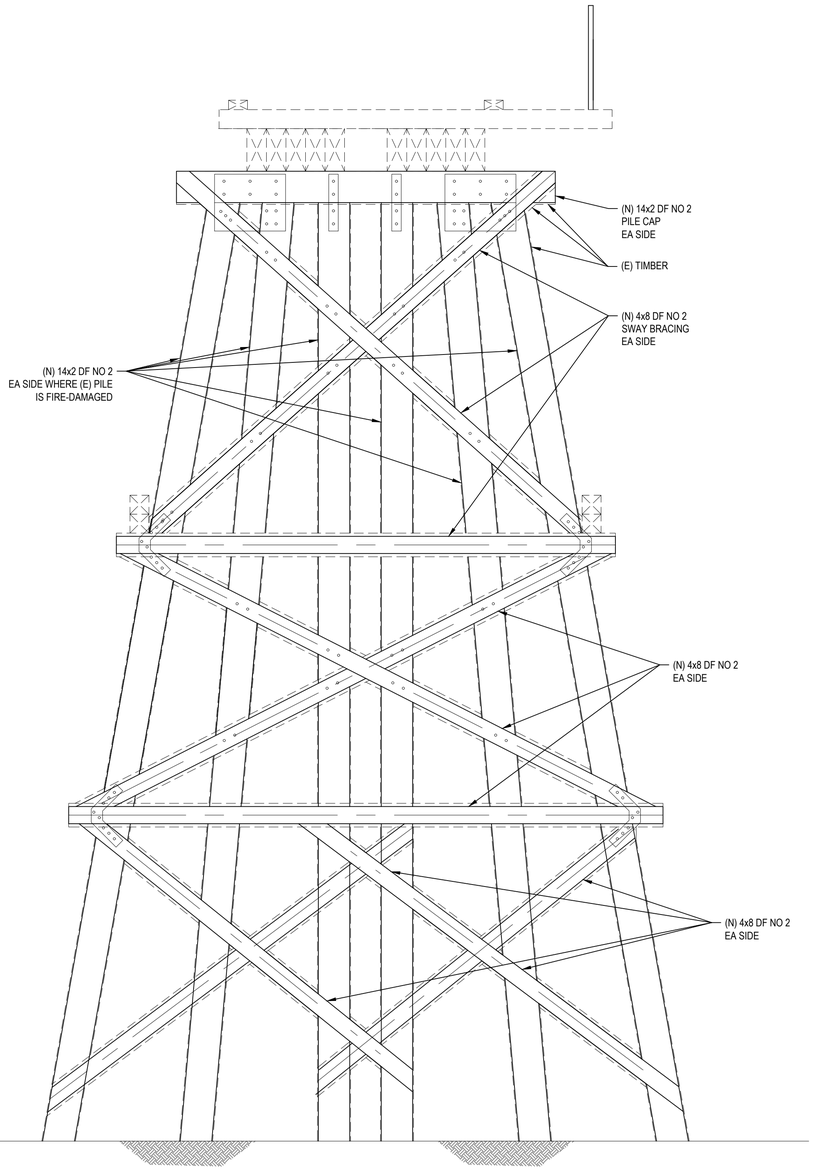
ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



A **BENT 7 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



B **BENT 8 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

BENT ELEVATIONS - ALT 1

	REVISIONS	INITIALS	DATE
6			
5			
4			
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PROJ MGR S. GRUPICO
DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 6 OF 8 SHEETS

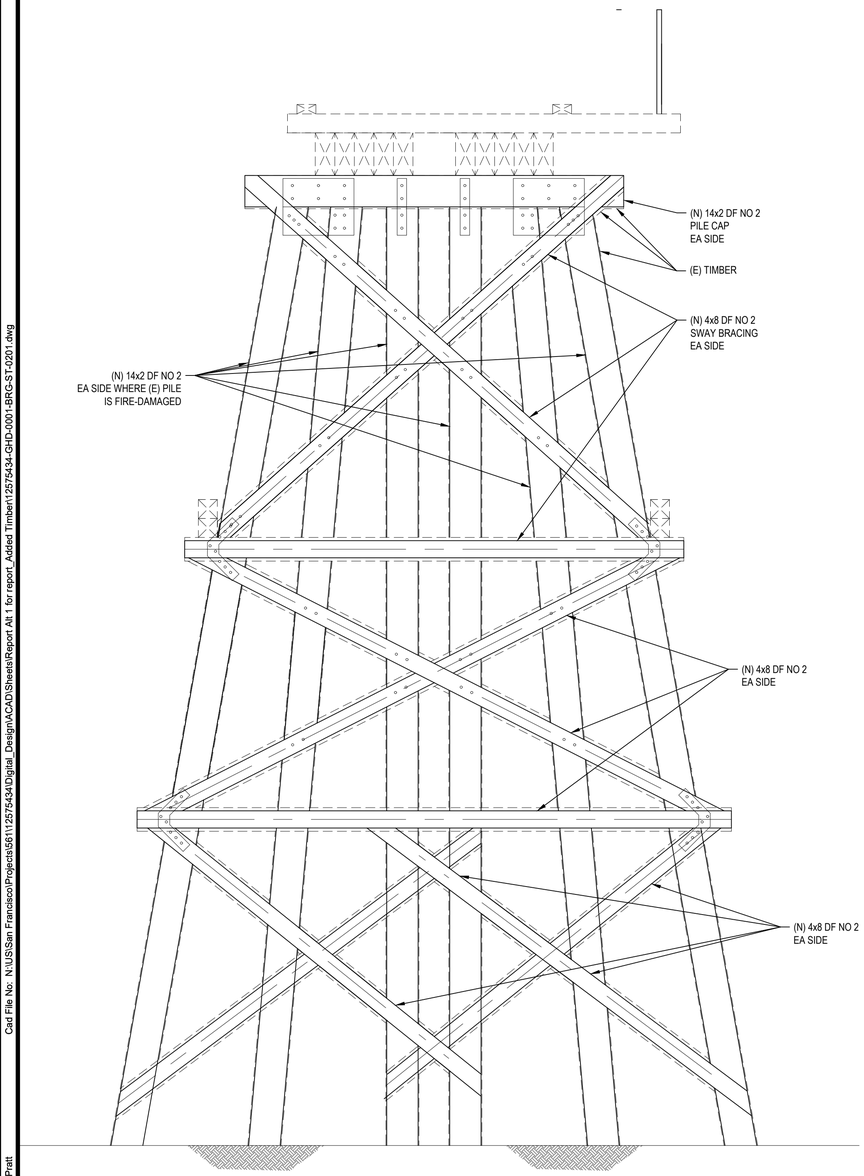
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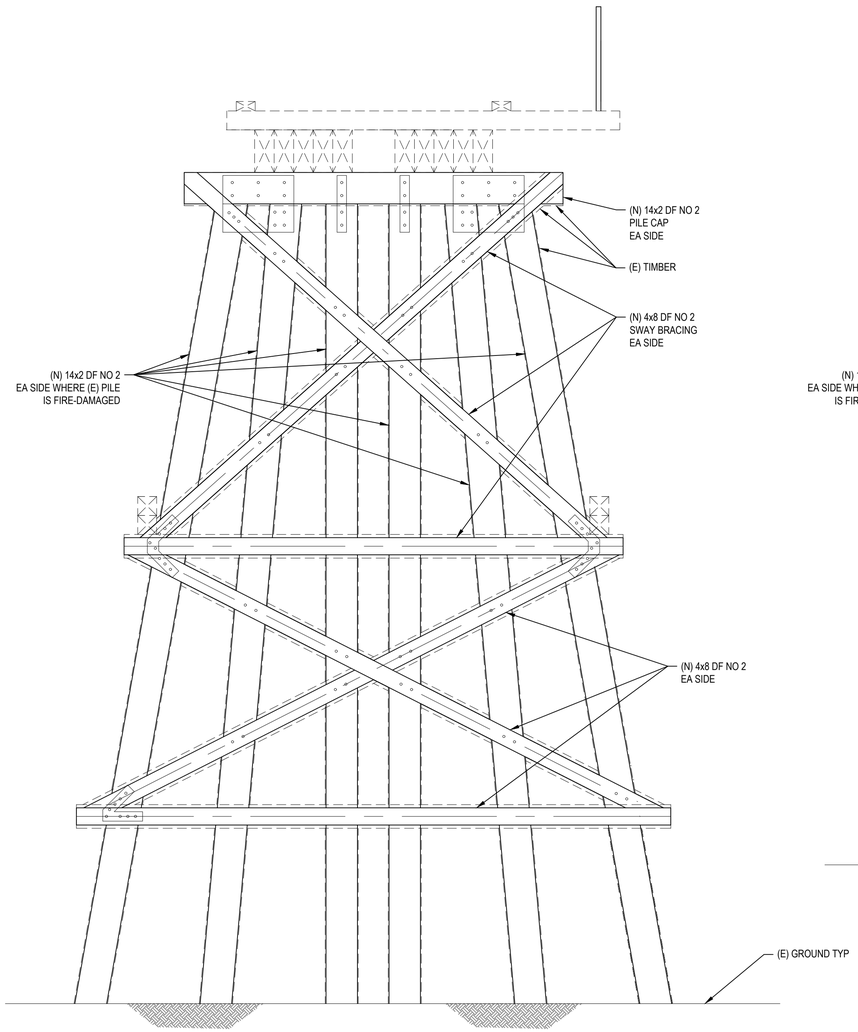
ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



A **BENT 9 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

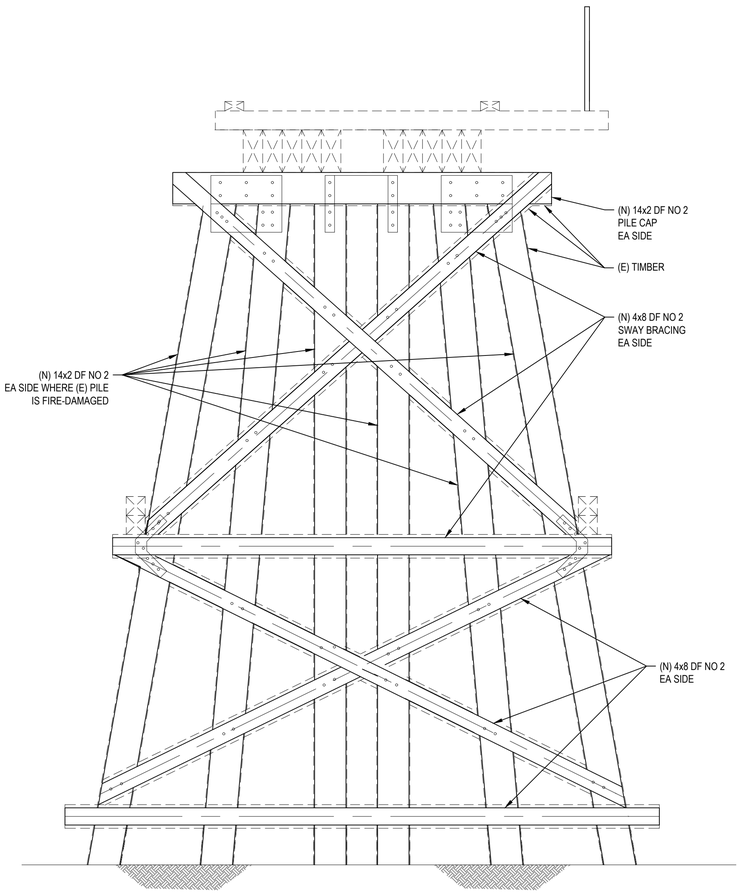
0 2' 4' 6'



B **BENT 10 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



C **BENT 11 ELEVATION - LOOKING SOUTH**

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

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BENT ELEVATIONS - ALT 1

	REVISIONS	INITIALS	DATE
6			
5			
4			
3			
2			
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SAN JOSE, CALIFORNIA

DESIGNED BY S. BURNS
CHECKED BY B. CROWELL
PROJ MGR S. GRUPICO
DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 7 OF 8 SHEETS

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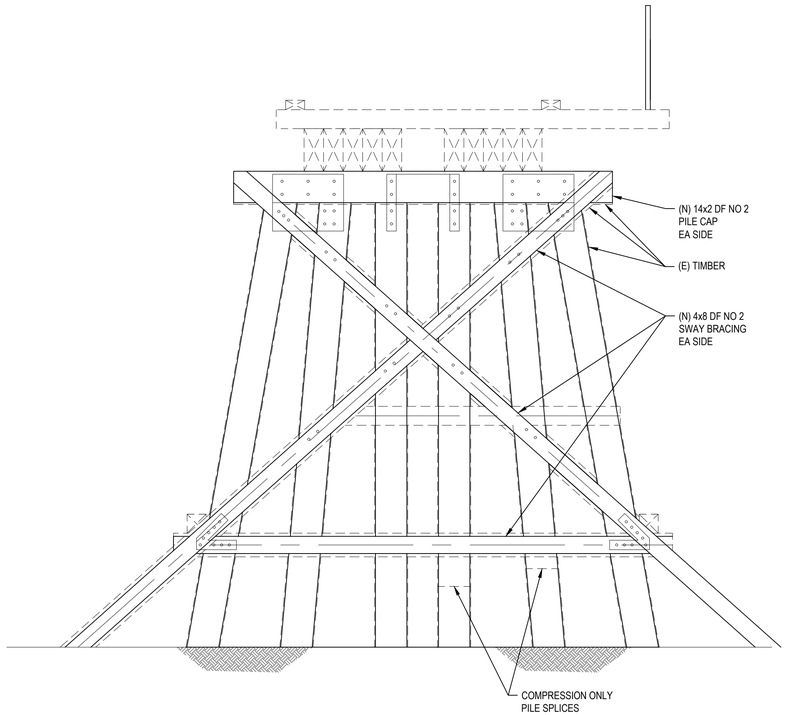
9966

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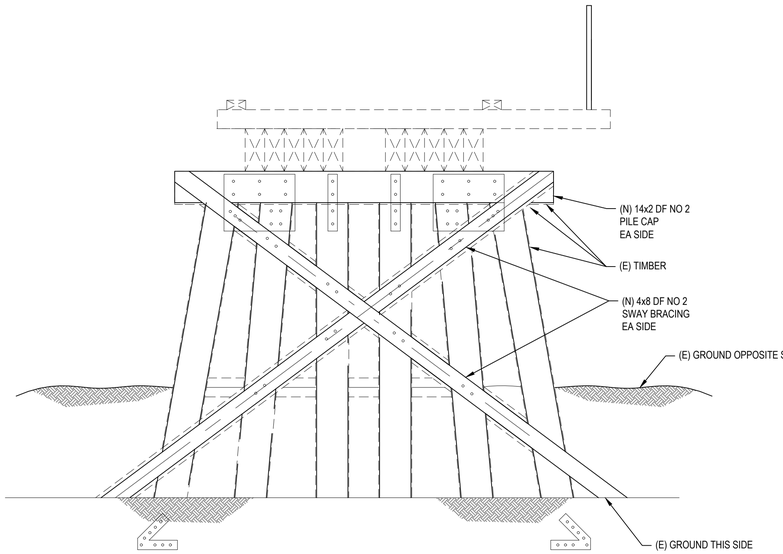
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Plotted By: Aly Pratt
Plot Date: 24 March 2023 - 3:39 PM

Plot Date: 24 March 2023 - 3:39 PM Plotted By: Aly Pratt Cad File No: N:\US\San Francisco\Projects\56\11225754\34\Digital_Design\ACAD\Sheets\Report Alt 1 for report_Added Timber\11257543 4-GHD-0001-BRG-ST-0201.dwg

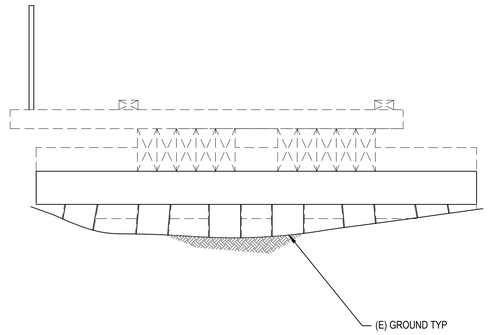
ANALYSIS OPTION 2 - RETROFIT FOR COLLAPSE PREVENTION



A **BENT 12 ELEVATION - LOOKING SOUTH**
SCALE 3/8" = 1'-0" (APPROXIMATE)
0 2' 4' 6'



B **BENT 13 ELEVATION - LOOKING SOUTH**
SCALE 3/8" = 1'-0" (APPROXIMATE)
0 2' 4' 6'



C **ABUT 14 ELEVATION - LOOKING NORTH**
SCALE 3/8" = 1'-0" (APPROXIMATE)
0 2' 4' 6'

S-205

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

BENT ELEVATIONS - ALT 1

	REVISIONS	INITIALS	DATE
6			
5			
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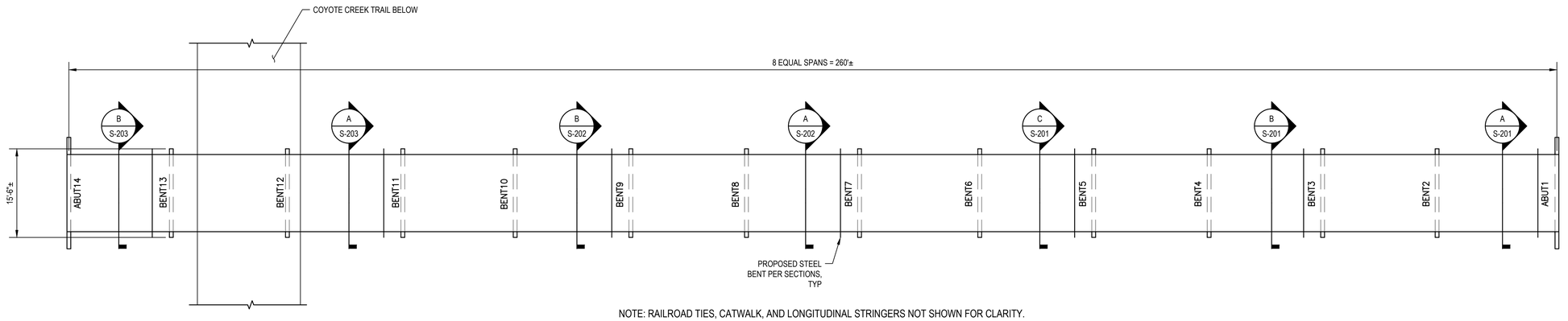
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PROJ MGR S. GRUPICO
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SCALE AS SHOWN
SHEET NO. 8 OF 8 SHEETS

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ANALYSIS OPTION 3 - RETROFIT FOR MULTI-USE PATH



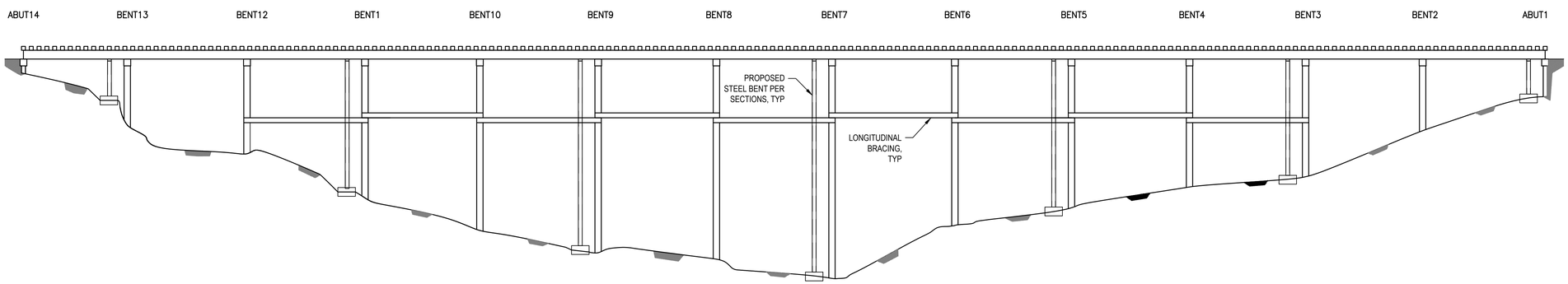
NOTE: RAILROAD TIES, CATWALK, AND LONGITUDINAL STRINGERS NOT SHOWN FOR CLARITY.

FIVE WOUNDS TRESTLE PLAN

SCALE 3/32" = 1'-0" (APPROXIMATE)

0 8' 16' 24'

N



FIVE WOUNDS TRESTLE ELEVATION

SCALE 3/32" = 1'-0" (APPROXIMATE)

0 8' 16' 24'

S-101

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

OVERALL PLAN VIEW AND ELEVATION - ALT 2

	REVISIONS	INITIALS	DATE
6			
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DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 3 OF 8 SHEETS

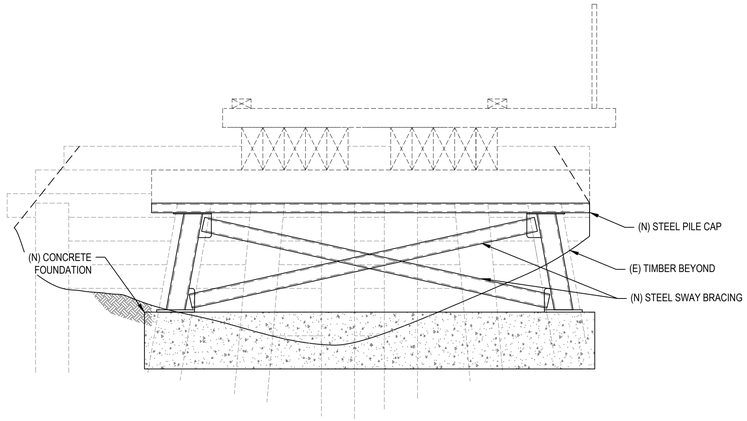
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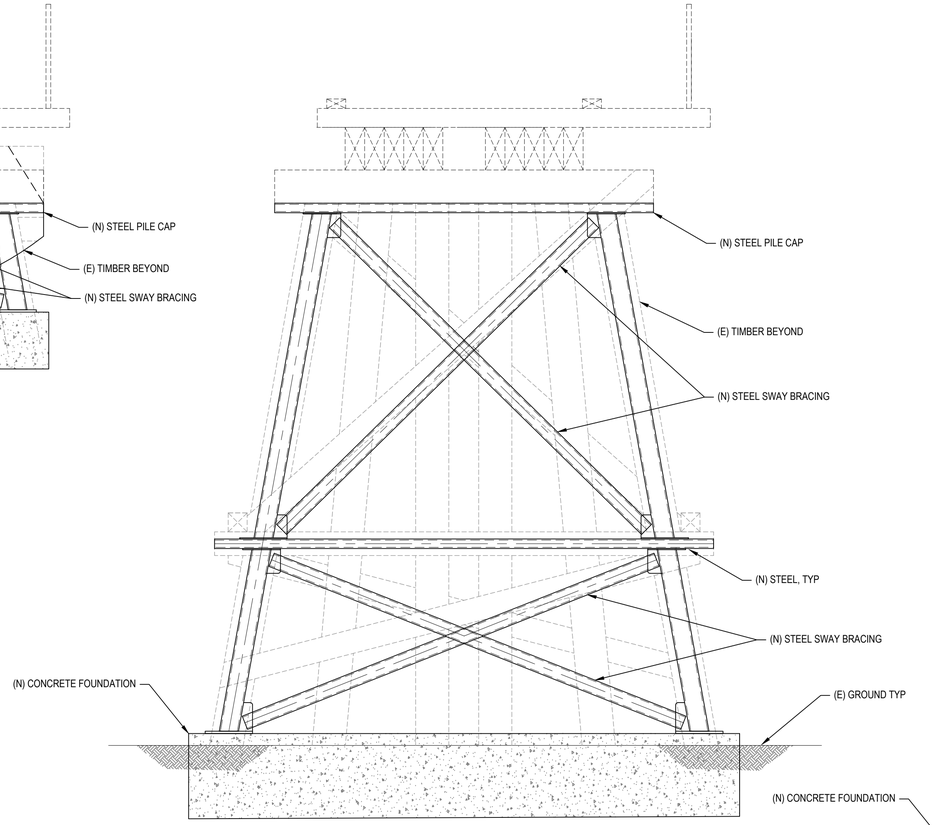
ANALYSIS OPTION 3 - RETROFIT FOR MULTI-USE PATH



1 SECTION A - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

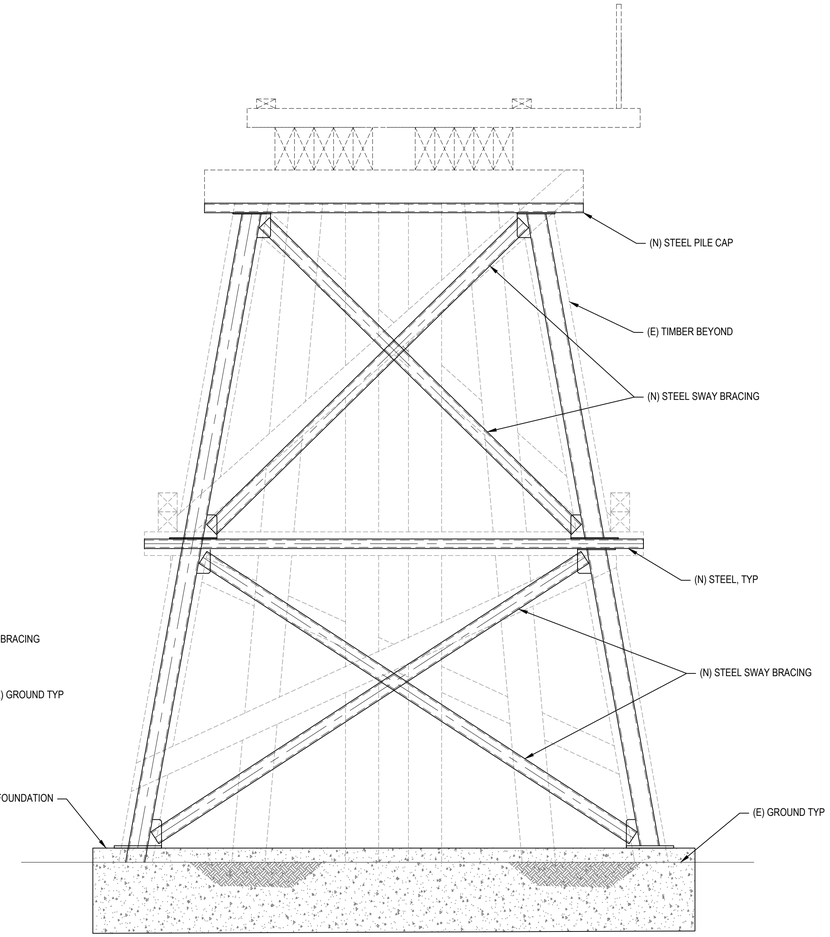
0 2' 4' 6'



A SECTION B - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



B SECTION C - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

ELEVATIONS - ALT 2

	REVISIONS	INITIALS	DATE
6			
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DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 4 OF 8 SHEETS

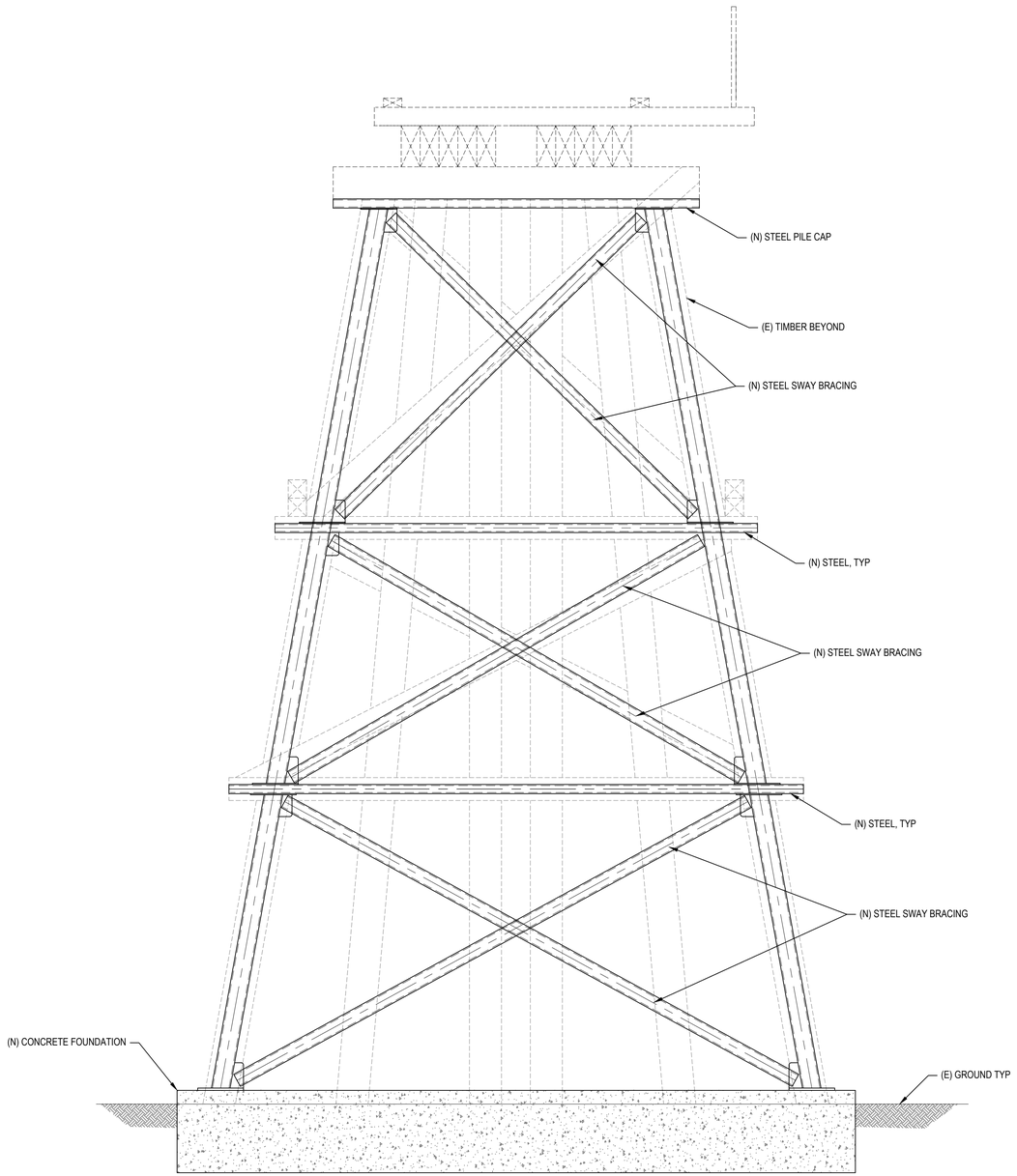
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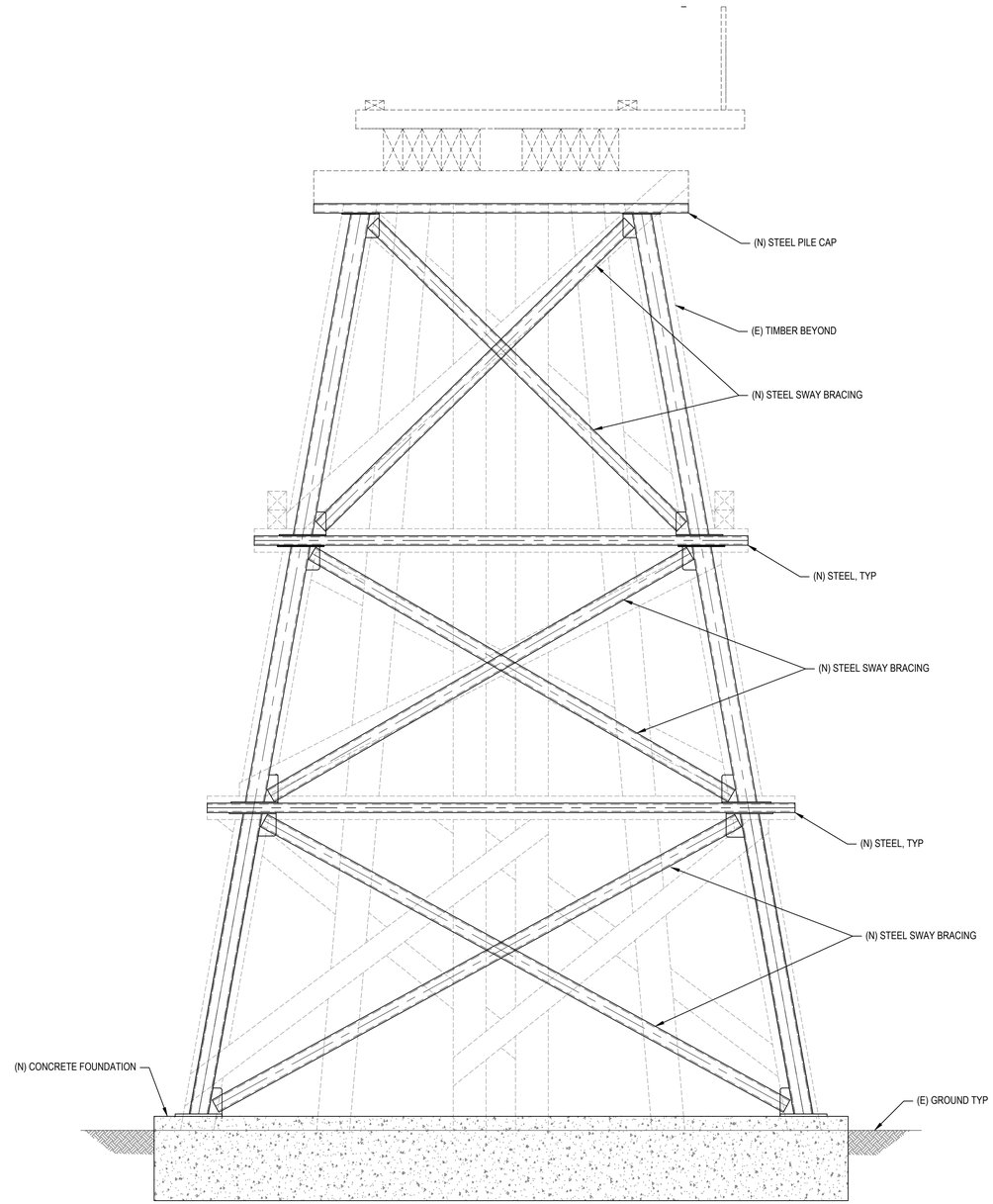
ANALYSIS OPTION 3 - RETROFIT FOR MULTI-USE PATH



A SECTION A - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'



B SECTION B - LOOKING SOUTH

SCALE 3/8" = 1'-0" (APPROXIMATE)

0 2' 4' 6'

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9966: FIVE WOUNDS TRAIL - TRESTLE EVALUATION

ELEVATIONS - ALT 2

6			
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	REVISIONS	INITIALS	DATE

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DATE 07-28-2022
SCALE AS SHOWN
SHEET NO. 5 OF 8 SHEETS

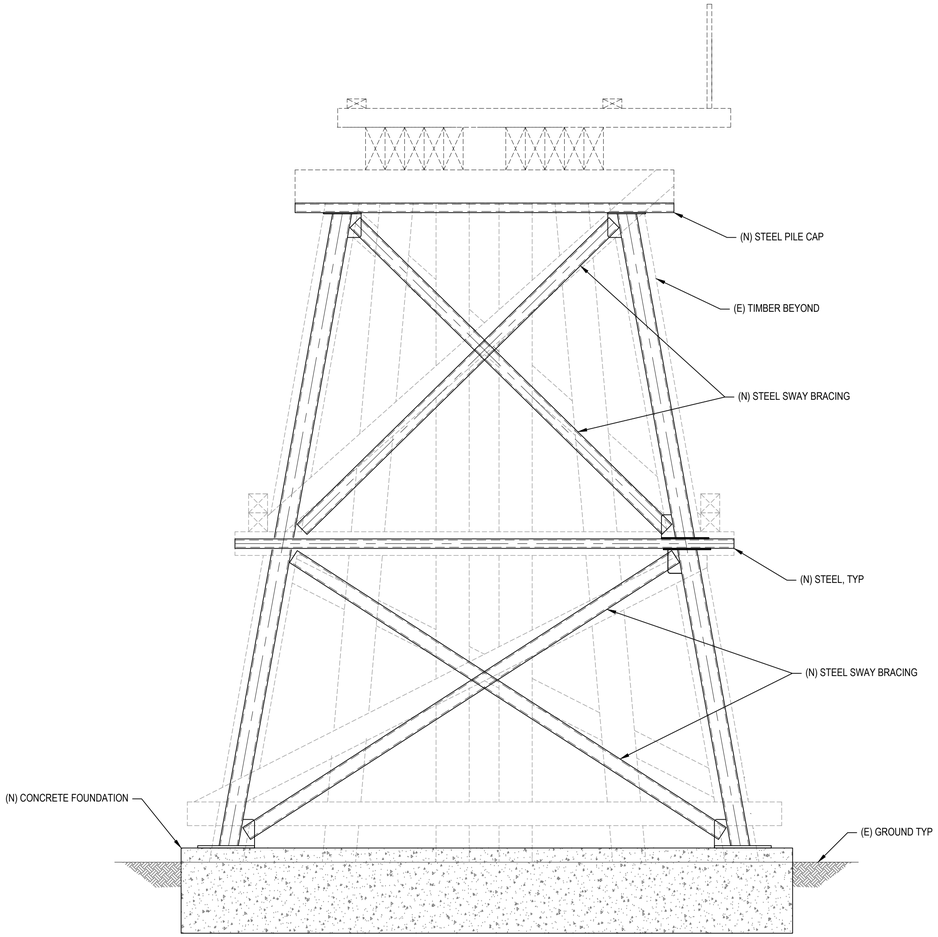
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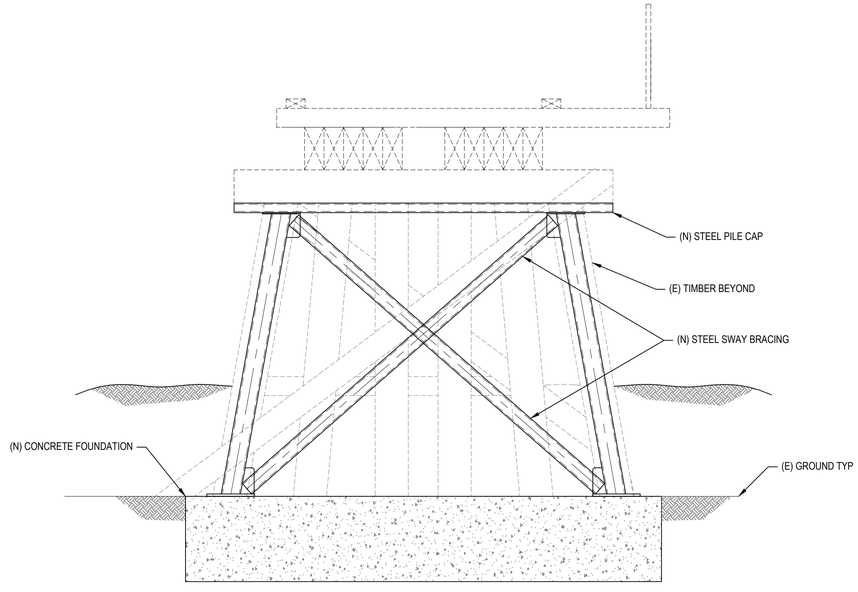
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ANALYSIS OPTION 3 - RETROFIT FOR MULTI-USE PATH



A **SECTION A - LOOKING SOUTH**
SCALE 3/8" = 1'-0" (APPROXIMATE)



B **SECTION B - LOOKING SOUTH**
SCALE 3/8" = 1'-0" (APPROXIMATE)

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ELEVATIONS - ALT 2

6			
5			
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	REVISIONS	INITIALS	DATE

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SCALE AS SHOWN
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