STRUCTURE TYPE SELECTION REPORT

Hammond Trail Bridge over the Mad River

Prepared For:
County of Humboldt Department of Public Works

Submitted On:
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Attachments

A. General Plans of Alternatives
B. Bridge General Plan Estimates
C. Preliminary Foundation Report
D. Preliminary Hydraulics Report

STRUCTURE TYPE SELECTION MEMO

<table>
<tr><th colspan="5">PROJECT IDENTIFICATION</th><th colspan="2">DATE</th></tr>
<tr><td colspan="5">Hammond Trail Bridge over the Mad River</td><td colspan="2">November 2024</td></tr>
<tr><th>DIST</th><th>CO</th><th>RTE</th><th>PM</th><th>EA</th><th colspan="2">CONSULTANT</th></tr>
<tr><td>01</td><td>HUM</td><td></td><td></td><td></td><td colspan="2">Mark Thomas</td></tr>
<tr><th colspan="5">BRIDGE NAME(S)</th><th>BR NO(S)</th><th>CONSTRUCTION COST IN 2024 DOLLARS</th></tr>
<tr><td colspan="5">Hammond Trail Bridge</td><td>TBD</td><td>$9,460,000 (Alternative 1)
$9,460,000 (Alternative 2)
$10,400,304 (Alternative 3)</td></tr>
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Brief Project Description

The County of Humboldt Department of Public Works (County) seeks to obtain recommendations and undertake type selection study for the replacement of an existing bridge over the Mad River, which carries the Hammond Trail pedestrian and bicycle traffic. The existing bridge is a six span, 540-ft long structure that crosses the Mad River approximately one mile inland from the Pacific Ocean. The two inner river spans consist of a 250-ft long, riveted steel through-truss and a 130-ft long riveted plate through-girder supported on concrete piers. These spans were erected at the site in 1941 and, together with timber truss approach spans that have since been removed, served as a railroad bridge until 1981. Available plans indicate that the truss and plate girders spans were erected on existing and newly constructed piers. The plans indicate that the piers are pile-supported, though specific information regarding the pile type, length and capacity is unknown. Standard construction practice of that era was to use timber piles and the piles are likely to be relatively short and likely not designed to accommodate seismic effects.

In 1983, the structure was converted to a pedestrian and bicycle bridge with the installation of a concrete deck on the truss and plate girder spans and the addition of new approach structures. Each 80-ft long approach structure comprises two spans of rolled steel girders with a non-composite concrete deck. The new approach spans are supported on abutments and bents with driven HP10x42 steel piles.

Recent inspection records indicate that the existing steel floor beams supporting the pedestrian walkways are severely corroded and were supplemented with wood timbers in 2011. Significant corrosion also exists on the truss members although based on the report by Morrison Structures, Inc in 2014, it does not appear any of the primary members are compromised to the point where the structure is unsafe. No field inspections were carried out by Mark Thomas as part of this scope of work. Available information indicates that extensive structural rehabilitation, cleaning and painting would be required to extend the useful life of the bridge. Additionally, the bridge is likely vulnerable to seismic events due to era of construction and shallow timber piles considering that the site is prone to seismically-induced liquefaction. As the required rehabilitation and retrofit upgrades are likely to come at a very high cost, the County has determined that replacing the bridge, either partially or completely, is the preferred option.

The County previously carried out Type Selection studies in 1998 and 2011 prepared by consultants CH2M Hill and Morrison Engineers, respectively. This Type Selection Study seeks to build on the previous work done while updating the design for the latest regulatory requirements, design standards, cost data and construction technology as well as the the operational, maintenance and construction cost requirements set by the County.

The project location is shown in Figure 1.

Figure 1 Project Vicinity Map

Structure Design Criteria

Design of the pedestrian bridge will follow the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 8th Edition with California Amendments; AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridge December 2009 with 2015 Interim Revisions; and 2023 Caltrans Standards. In addition to the 90 psf pedestrian load, the design will also consider loading from a single truck, design maintenance vehicle H10. This vehicle loading would accommodate common maintenance vehicles or an ambulance that may need to drive on the bridge.

Caltrans Seismic Design Criteria (SDC) Version 2.0 will be used for the Seismic Design of the structure. Based on information provided in the Preliminary Foundation Report, the horizontal peak ground acceleration is estimated at 1.03g.

Configuration, Clearances and Geometry

The primary physical constraint for the project is the Mad River, which the bridge crosses. The water surface level in the river is highly dependent on the season, but water is always present in at least some part of the river. The approximate distance from bank to bank of the river is 400 ft. While the river banks are steep, beyond the banks the project site is very flat aside from the imported fill embankments of the existing bridge. Due to the low water flows during summer and fall, there are no navigational horizontal or vertical clearance requirements that have been identified. This should be confirmed with Coast Guard during final design for the project as the project is located in a tidal influence zone which may require their approval. Therefore, the height of the bridge was set by providing the required clearance above the 50-year and 100-year flood events discussed in the hydraulics section. It is worth noting that the clearance of the existing bridge significantly exceeds the clearance required for the 50-year and 100-year flood events. Lowering the profile of the new bridge to minimum required by current standards will make it easier and less expensive to achieve the 5% maximum slope required by current Americans with Disabilities Act (ADA) standards.

The existing bridge is also a physical constraint; it is anticipated that the new bridge will be constructed approximately 50 ft upstream from the existing bridge and align with Fischer Ave and Mad River Rd placing the new bridge in the existing County Rd easement.

The trail profile was set with a maximum grade of 4.5% to achieve the 5% required by ADA standards accounting for construction tolerances. ADA standards also require a flat 5 ft.-long resting area every 400 ft along continuous gradients. The current preliminary profile (based on lidar contours) has continuous slopes just over 400 ft. The need for the flat landings will be determined during final design using detailed topographic survey and considering the final bridge geometry.

Typical dedicated bicycle and pedestrian bridges have a clear width ranging from 10 ft (minimum allowable by AASTHO) to 16 ft. Based on the expected usage of the bridge and per discussions with the County, it was decided that a clear width of 12 ft was appropriate for this structure.

The north end of the bridge has minimal constraints, however the south end of the project site is bounded by the Mad River Rd. To avoid modifications to the existing road, the new trail profile must conform to existing ground before the road. At the south end of the bridge there is also an existing parking lot at the Hammond Trail trailhead. This parking lot will likely be impacted by the new approach embankment and may need to be shifted to the west to where the existing trail is.

Corrosion Issues

Due to the proximity to the Pacific Ocean, the site is considered a harsh marine environment, which is demonstrated by the extensive corrosion of the existing bridge. Any structural steel used for the new bridge would require frequent and regular maintenance to keep the steel protective coating intact. The County desire to minimize maintenance costs and maximize the service life of the bridge preclude any structure alternatives with structural steel such as prefabricated steel trusses or steel plate girders.

Secondary components such as pedestrian railings, utility pipes and bearings should be galvanized. Additional protective measures such as stainless steel or a duplex paint system should also be considered to extend the service life of these components. Reinforcing steel within the concrete components will utilize additional cover as specified in California Amendments to AASHTO LRFD Bridge Design Specifications.

In addition to atmospheric corrosion, the bridge is located in a tidal zone and portions of the substructure in the splash zone will be exposed to additional chlorides. Testing of the water at the bridge site during low flow and high tide periods should be done in the next phase of the project to quantify the chloride concentrations. The California Amendments to AASHTO specify the required reinforcement cover for various chloride levels.

The soil testing also indicates that the soil is highly corrosive due to high chloride and sulfate levels. Special concrete mix design may be required with additional SCM as well as type II or type V cement. If steel piles are used, a corrosion allowance must be considered.

Foundations

The geotechnical consultant, Crawford and Associates, Inc, completed a preliminary foundation report, which is included as Attachment C. Their recommendations are based on a review of the previous geotechnical subsurface investigations conducted in 2015 (*Final Foundation Report, Hammond Trail Pedestrian Bridge Replacement Mad River Crossing, North of Arcata, Humboldt County, California, prepared by SHN Consulting Engineers & Geologists, Inc., May 2015*).

Below are the key findings and recommendation of the Preliminary Foundation Report:

- The reuse of the existing bridge foundations is not considerable feasible due to the lack of as-built documentation. Given the year of construction the foundations are likely timber and do not have adequate depth or capacity for seismic loading.
- The site is subject to strong ground shaking with a horizontal peak ground acceleration of 1.03g and potential for liquefaction in the upper 70 ft of soils.
- Due to significant depth of liquefaction, seismic downdrag will be a significant consideration, requiring pile tips well below the liquefiable zones.
- Lateral spreading is likely at the abutments, which will significantly impact the pile sizes.
- Given the required pile tip elevations, driven closed end piles, H-piles and small diameter are not considered feasible/economical due to the hard driving conditions.
- Large diameter Cast-in Drilled Hole (CIDH) or large diameter Cast-in Steel Shell (CISS) piles are considered the most feasible options. However, due to environmental restrictions, the CISS piles would need to be driven in dewatered cofferdam or on a temporary berm to limit the detrimental vibration/hydroacoustic effects.

Based on the recommendations and considerations above, large diameter CIDH piles were chosen as the preferred foundation type for both the river piers and abutments.

Aesthetics

As functionality, cost and long-term maintenance are the main considerations for the project it is not anticipated that significant architectural and aesthetic enhancements such as decorative railings, non-prismatic column shapes or concrete formliners would be included. Nevertheless, a classic “form follows function” appearance will be achieved by using appropriate span-depth ratios and proportions for structural members.

Hydraulics

A hydraulic analysis was completed for the project site using the US Army Corps of Engineers HEC-HMS hydrologic modeling software. The results are present in preliminary hydraulics report included as Attachment D.

A summary of the water surface elevations for the existing condition are shown in the table below. As discussed in the preliminary hydraulic report, the impact on water surface elevation from the proposed bridge will be evaluated in the next phase, but is expected to minimal and within the FEMA allowance.

Flood Recurrence Interval	Condition	Water Surface Elevation (ft)
Q50	Existing	19.85
	Proposed	To Be Determined
Q100	Existing	20.25
	Proposed	To Be Determined

Caltrans requires new bridges to pass the greater of the 100-year storm event or 50-year storm event plus two feet of freeboard. For this project, the 50-year storm plus two feet of freeboard is the governing case and dictate the required bridge soffit elevation.

Scour

A preliminary scour analysis was performed, and the results are provided in the preliminary hydraulic report. Below is a summary of the estimated scour at each support:

Support No.	Degradation/Contraction Scour (ft)	Short Term Local Scour (ft)
Abutment 1	TBD	To Be Determined
Bent 2	0.6	13.5
Bent 3	0.6	12.8
Abutment 4	TBD	To Be Determined

The bents in the river and the abutment CIDH piles will be designed for the anticipated scour as per current Caltrans design criteria. While the pile supported abutments will be protected with rock slope protection, current Caltrans practice does not allow accounting for it in the structural design.

Traffic Impacts

Demolition of the existing bridge and construction of the new bridge is not anticipated to result in significant traffic impacts as the existing bridge only carries bicycle and pedestrian traffic. Short duration traffic control in nearby streets may be required when transporting large components and increased truck traffic may be required for certain activities such as berm or embankment construction. However, these are not anticipated to be significant impacts and can likely be mitigated through traffic control strategies.

Aside from environmental issues associated with pile driving discussed in the following sections, there do not appear to be sensitive receivers near the site that would be significantly impacted by construction noise or vibrations.

Construction impacts and mitigations would be addressed in detail in the environmental documents, but they are not expected to influence the bridge type selection or details.

Environmental Issues During Construction

As the bridge is within a mile of the ocean and crosses the Mad River, additional environmental restrictions and considerations will be required and will have significant influence on construction means and methods and schedule.

Demolition of the existing bridge and construction of the new bridge will require construction access in the river. Given that in-water construction will only be allowed during the summer, as discussed below, the use of barges and floating cranes is not seen as feasible due to the low volume of water during the summer and the presence of the existing bridge which would restrict access to the new bridge upstream. Therefore, the most likely construction methods for working in the river are either a temporary trestle on driven piles or a temporary clean fill earthen berm. For either solution, the installation and removal of the temporary construction access will need to occur within a specific time period specified by permits, typically in the summer. For example, the recent Jacoby Creek Bridge Project done by Caltrans approximately 5 miles south on U.S. 101 did not allow any construction activities between October 15 and May 31 and in-water work was only allowed from July 1 to October 15. Therefore, the construction schedule must be developed around allowable work windows and the project start date should be mindful of when work can actually begin. Due to these restrictions, it is likely that 3 seasons would be required to construct the new bridge and demolish the existing as discussed in the following section.

It is also likely that driven piles within the water will not be allowed due to hydroacoustic effects from pile driving that may be harmful to nearby animal species. Piles can likely be driven on dry ground with hydroacoustic monitoring. For the foundations in the river this would need be accomplished using a clean fill berm or staging the work during low tide in the summer when portions of the riverbed are exposed, whose location is uncertain and difficult to predict.

Sediment and debris containment will also likely be required for activities in or above the river.

Stage Construction

Although it is technically feasible to maintain the existing Hammond Trail during construction, for safety considerations, the existing will likely need to be closed during the construction windows to keep people away from the work zone as there will be construction equipment moving around the site. Outside of the summer construction work windows it would be possible to reopen the trail by continuing to use the existing bridge. However, if the trail is permanently closed throughout the entire construction, the existing bridge could be demolished at the beginning of the project or concurrently with the new bridge construction. This would likely eliminate a construction season essentially shortening the project duration by 1 year. As the existing 16" ductile iron pipe recycled water line on the existing bridge is not in active use, there are no issues anticipated withit is not demolishing the existing bridge prior to constructing the new bridge.

The main consideration for construction staging will be coordinating the work around the allowable construction window from June 1 to October 15 to maximize efficiency and reduce the total number of construction seasons required. Below is a conceptual outline of the potential construction staging and sequencing assuming the existing bridge is left in place until the new bridge is constructed:

Season 1: install clean fill berm on approximate alignment of proposed bridge; construct foundations for bents and abutments; construct bents and abutments; remove portion of berm as required by hydraulic analysis.

Season 2: install clean fill berm upstream of proposed bridge; construct bridge superstructure, approaches and new treated wastewater line; remove portion of berm as required by hydraulic analysis.
Season 3: install clean fill berm downstream of existing bridge; remove existing bridge; complete any final miscellaneous items on the new bridge; remove berm.

As discussed previously, if the trail is closed throughout the entire construction the season 3 work can likely be performed in season 1 and/or 2 and season 3 can be eliminated.

Constructability

There are a number of constructability issues specific to the site that must be considered for both the new bridge and demolition of the existing bridge:

Limited Access
The two main access points to the site are Mad River Rd and School Rd/Fischer Ave. Both roads are fairly narrow and require sharp turns that may not be achievable for some construction equipment. In addition, all major highways leading to the project site have steep winding sections. Shipping large precast/prestressed girders to the site will be slow and costly. Based on discussions with precast manufacturers and trucking companies, shipping 140-ft long precast girders to the project site is feasible based on an initial assessment of the hauling route, but would require permits and California Highway Patrol escort most of the way. The preliminary cost estimates provided in Attachment B reflect the additional costs of transporting the precast girders to the site.

Allowable Construction Windows
Due to environmental considerations, construction work will likely only be allowed from June 1st to October 31st. This significantly reduces construction efficiency and will require design solutions and construction methods that accelerate construction within the allowable window. From this perspective, the use of precast concrete elements is attractive as they can be fabricated offsite ahead of time and assembled onsite thereby accelerating construction within the allowable windows.

River Access
All work within the river will be done from either a berm or a temporary trestle. This will also reduce construction efficiency and limit the size and amount of construction equipment that can be used for a given activity and should be considered when determining the number of working days for the project.

If a clean fill berm is used, at least a portion of clean fill berm would likely need to be removed at the end of the construction season to avoid significant impacts to the water surface elevations during high flows in the winter and spring. The amount of berm that may be left in the river during the winter would be determined by hydraulic analysis and permitting requirements. Therefore, the berms, either partially or completely, will need to be removed at the end of the construction window.

Temporary trestles are typically more costly than clean fill berms due the additional piles and structural components they require. However, it may be possible to build a temporary trestle between the existing bridge and the new bridge, which could then be used for both the new bridge construction and the demolition of the existing bridge. If the trestle decking is removed, the piles and structural beams could be left in place during the winter. Therefore, the main components of the trestle would only need to be installed and removed once. This may make a trestle a more cost-effective alternative compared to the clean fill berm which would need to be installed and removed multiple times. However, the downside of the trestle is that it would require piles to be driven in the river and as discussed previously, this likely will not be allowed due to environmental concerns. Some of the driven foundation piles could be installed on dry land during low tide/low flow periods, but the location and timing for this would be unpredictable and very difficult to schedule and plan for. Therefore, some piles would likely need to be CIDH, which will significantly increase the construction duration and cost of the trestle.

In the next phase of the project, the issues above should be further evaluated and discussed with the regulatory agencies to determine specific permitting requirements and restrictions. To the extent possible, the final design and specifications should be as flexible as possible regarding construction means and methods as the preferred option may vary significantly between contractors.

Given the likely restriction on pile driving for the temporary trestle, it was assumed that a clean fill berm will be used and that the berm will need to be installed and removed three times for purposes of preparing the general plan estimates provided in Attachment B . This provides a reasonable and conservative estimate of the project cost for planning and funding purposes, which can then be refined as the project progresses.

Construction Laydown and Staging
A construction staging and laydown area will be required on or adjacent to the project site. This may be used to stockpile berm material outside of the construction work window and to place equipment. Therefore, a temporary construction easement (TCE) may be needed for this purpose. If a TCE beyond the county road easement is required, a suitably-sized TCE for the laydown area should possible as the area around the project site is mainly agricultural . McKinleyville Community Services District (MCSD) also owns property in the project area, which could potentially be requested to be used for laydown and staging.

Bridge Demolition

Demolition of the existing bridge will require access to the river using a clean fill berm or a temporary trestle just like for the new bridge construction. The berm would likely need to be directly below, or downstream of the existing bridge to be removed.

While the means and methods to demolish the bridge would be the responsibility of the contractor, who would need to retain a licensed professional engineer to develop the detailed bridge removal plans and procedures, a general approach could be as follows. The first step of the demolition will likely be to remove the non-composite reinforced concrete deck on both the truss span and steel girder spans as well as any other components of the deck framing that can be safely deconstructed without compromising the structural stability of the bridge. This can typically be accomplished using lightweight construction equipment (e.g concrete saws, small excavators, etc.) on the bridge deck to cut the concrete into small pieces which can then be hauled away. After the concrete deck is removed, the primary components of the superstructure could be disassembled. The steel plate girders can be lifted out in entire segments with cranes using similar techniques as those used for their installation. The steel truss may be removed by either:
1) Supporting it with falsework from the berm or trestle and then removing it piece-by-piece; or
2) Using controlled explosions to drop it into the river or onto a berm. The steel components could be then cut into pieces on the ground and hauled away.

Typically option 2 is more cost-effective if allowed environmentally. The existing bridge likely contains lead paint and possibly asbestos, which will require safety plans and abatement. Explosive demolition was used for the Antlers Bridge for the Sacramento River arm of Shasta Lake. Silt curtains were deployed to collect any debris and an analysis was done to confirm dissolved lead levels would stay within the allowable limits.

Once the superstructure is removed, the piers can be removed with a wire saw and a crane.

Accelerated Bridge Construction

Given that the existing bridge only serves bicycle and pedestrian traffic and is mainly used for recreation, accelerating bridge construction is not a key consideration with respect to minimizing traffic disruptions. However, incorporating accelerated bridge construction techniques that maximize construction speed within the allowable environmental work windows will be a key consideration to reduce overages in construction schedule and cost. Using precast concrete girders and precast concrete deck panels may be a feasible means of accelerating bridge construction.

Utilities

The existing bridge carries a 16” ductile iron pipe for treated wastewater used for irrigation. The 16” pipe and existing bridge are owned by the McKinleyville Community Services District (MCSD). The improvements to the existing bridge for Hammond Trail and the continued operations of the trail are governed by a 1991 agreement between the County and MCSD, which appears to have expired in 2021 following the two 10-year automatic renewals. Under the 1991 agreement, the County was responsible for all maintenance costs of the trail and the maintenance costs of the bridge were split 50/50 with MCSD. Per article 7 of the agreement, the County has the right to abandon the trail at any time but is required to remove the trail at its own expense. It is unclear if this requirement is still enforceable as the agreement has now expired. Initial discussions with MCSD indicated that they would like to preserve the ability to carry the waterline across the river and wish to relocate the existing pipeline to the new crossing when the existing bridge is demolished. However, as the existing 16” pipe is not in active use and MCSD is mainly looking to maintain the pipeline as a potential future facility, a cost-saving alternative would be to design the bridge to accommodate a future pipe installation (e.g. abutment blockouts, inserts for pipe hangers, etc) but not actually install the new pipe during the initial bridge construction. MCSD could then install the pipeline at any point in the future when they determined the pipeline is needed.

For the purposes of this report and associated cost estimates, it is conservatively assumed that the existing bridge will be demolished, and a new pipeline will be installed on the new bridge. During the next phase of the project, this should be further discussed with MCSD as well as cost sharing for the new pipeline.

The location of the utility line and schematic rerouting to the new bridge is shown on the Foundation Plan provided in Attachment A.

Bridge Lighting

As the none of the trail is illuminated on either end of the bridge, no lighting is proposed on the bridge, either on the bridge deck or bridge soffit. Per discussions with the County, the design team understands that it would be undesirable to provide lighting on the bridge as it could be prone to vandalism.

Barriers and Railings

Combined bicycle and pedestrian railings will be provided on both sides of bridge. As earthen embankments with 2:1 slopes are proposed for the approaches, a railing is not required beyond the bridge abutments. The pedestrian railings will be 48” tall as required by California Amendments to AASTHO LRFD Bridge Design Specifications. It is anticipated that railings will be tubular HSS post rails with horizontal cables spaced vertically at 4”.

Bridge Drainage

It is anticipated that curbs will be provided on each side of the bridge deck to contain rainfall on the bridge. Since the total bridge length is only 400 ft, the clear width is 12 ft and the bridge is on a crest vertical curve, preliminary deck drainage calculations indicate that deck drains are not required. The water will run off the ends of the bridge onto the approach embankments. Erosion control will be provided at locations where channelized water exits the bridge deck. Actual deck drainage requirements will be determined and coordinated during final design.

Permits and Approvals

An exhaustive list of all the possible permits and approvals that may be required for the project is beyond the scope of this report, however, below is preliminary list of agencies that may have jurisdiction over the project and will require coordination with:

- United States Army Corp of Engineers (USACE) for construction activities in water. They will likely require Section 404 and Section 10 permits;
- California Department of Fish and Wildlife (CDFW) lake and stream bed alteration permit;
- California Coastal Commission will require coastal development permit due to the bridge proximity to the coast;
- North Coast Regional Water Quality Control Board water quality certification and/or waste discharge requirements;
- Caltrans District 1 Structures Local Assistance (SLA) oversight for use of state or federal funds;
- McKinleyville Community Services District (MCSD); and
- As the bridge is located in a tidal influence zone it may be classified as a navigable water per 33 CFR part 2.36, thereby requiring a permit from the United States Coast Guard.

It is recommended that these agencies be engaged early in the project approval process to understand and incorporate any design and/or construction requirements.

Alternatives Considered

Based on the site configuration, requirements and issues presented herein, multiple viable bridge alternatives were considered with the objective of minimizing both project cost and future maintenance.

Considering the site and available information, the most cost-effective location for the abutments was deemed to be at the top of the riverbanks. The existing bridge has abutments set back from the riverbank approximately 100 ft, however, based on the site topographic contours and hydraulic modeling it was determined that it was feasible to place the abutments on top of the bank and protect them from scour using appropriately sized rock slope paving (RSP). This approach reduces the overall structure length by approximately 200 ft, which is far greater savings than the added cost of the RSP. Minimal maintenance should be required if adequately sized RSP is used.

Even with the abutments moved forward to the top of the riverbanks, spanning the entire river without intermediate supports would require a 400 ft clear span. To achieve a 400 ft simple span an arch or cable-supported structure would likely be needed. Due to the high initial costs and high maintenance costs of such a structure, spanning the entire river was determined to be not economically feasible.

As discussed previously, due to the harsh marine environment and the County's need to minimize future maintenance, alternatives with structural steel such as a steel plate girder or steel truss structure were ruled out.

A two-span configuration with two equal spans of 200 ft and a pier in the river was also considered. However, a number of disadvantages were found with this solution:

- The larger spans will require a haunched concrete girder increasing the design complexity and construction cost,
- The high seismic demands including liquefaction, along with the need to design for scour would on a single pier, would require an excessively large foundation.
- Constructing a single pier in the middle of the river would require approximately the same length of temporary berm or trestle compared to constructing two piers.

Due to the issues above, a two-span configuration with a single pier in the river was discarded. Therefore, it was determined that the most cost-effective span configuration is a 3-span alternative with spans of 127.5'-145'-127.5'. For this span configuration, the three most economical solutions were determined to be:

1) Precast/Prestressed (PC/PS) California Wide Flange Concrete Girders with large diameter CIDH piles;
2) Cast-in-place post-tensioned concrete box girder with large diameter CIDH piles; and
3) Cast-in-place post-tensioned concrete box girder with driven pipe piles and pile cap

The General Plan for each of the three alternatives is provided in Attachment A and the associated General Plan Estimates are provided in Attachment B. Note that the General Plan Estimates include the construction costs of the approaches and sewer line to provide the County with a more comprehensive understanding of the project's total costs. Escalation costs are also provided assuming construction will start in 2029 and will last 3 years resulting in roughly 7 years of escalation from today's cost to the midpoint of the construction. The annual escalation was assumed to be 5%, which is based the California Construction Cost Index average over the previous 10 years.

__Alternative 1: PC/PS California Wide Flange Concrete Girders with Large Diameter CIDH Piles__

Advantages

- No falsework will be required in the river minimizing risk associated with summer storms that could wash away temporary works;
- Will accelerate construction during the allowable work windows as precast girders can fabricated offsite outside of the work windows.

- Better quality control as girders are fabricated in a controlled factory environment and can be inspected before being shipped to site;
- Will be slightly lighter compared to a cast-in-place post-tensioned concrete box structure thus reducing seismic demands on supporting elements and foundations; and
- Large diameter CIDH piles avoid the need for cofferdam and pile cap construction and pile driving in the river.

Disadvantages

- Shipping 145 ft long precast girders to the site will be difficult and costly due to the lack of trucking routes to the nearest precast manufacturing plants;
- Large cranes will be required to erect and place the girders requiring larger berms in the river;
- Generally considered less aesthetically pleasing than concrete box girders; and
- Single column piers with a large diameter CIDH pile in the wet is generally seen as higher risk construction.

<u>Alternative 2: Cast-in-Place Post-Tensioned Concrete Box Girder with Large Diameter CIDH Piles</u>

Advantages

- Does not require shipping large precast components to the site;
- More aesthetically pleasing than precast concrete girders and provides more flexibility to change the shape of fascia girder to suit any aesthetic requirements; and
- Large diameter CIDH piles avoid the need for cofferdam and pile cap construction the river.

Disadvantages

- Heavier superstructure compared to precast concrete girder alternative resulting in higher seismic demands and potentially longer piles
- Requires installation and removal of falsework which will reduce the amount of construction that can take place in a given season;
- Single column piers with a large diameter CIDH pile in the wet is generally seen as higher risk construction

<u>Alternative 3: Cast-in -Place Post-Tensioned Box Girder with Small Diameter Driven Piles and Pile cap</u>

Advantages

- Does not require shipping large precast components to the site
- More aesthetically pleasing than precast concrete girders and provides more flexibility to change the shape of fascia girder to suit any aesthetic requirements; and

- Multi-pile group with small driven piles typically viewed as less risky than single large diameter CIDH pile

Disadvantages

- Heavier superstructure compared to precast concrete girder alternative resulting in higher seismic demands and potentially longer piles
- Requires installation and removal of falsework which will reduce the amount of construction that can take place in a given season;
- Requires cofferdam for pile construction increasing construction cost and duration

Recommendation

Based on the general plan estimates provided in Attachment B, Alternative 3 with a pile cap foundation is significantly more expensive compared to the other two alternatives. This is reasonable as large pile cap foundations, particularly in a river are generally more expensive which is why they are avoided whenever possible. Therefore, Alternative 3 is not recommended. Risks associated with a single column CIDH pile in Alternatives 1 and 2 could be mitigated through appropriate design and construction specifications. For example, requiring that the contractor use full depth temporary casing, perform inspections of the pile base with a Miniature Shaft Inspection Device (MiniSID), develop an adequate pile anomaly mitigation plan and furnish minimum qualifications for any drilling subcontractors are some of the strategies that could be used to help mitigate risk.

The General Plan Estimates indicate that the cost of Alternatives 1 and 2 is essentially the same given the level of design at this preliminary stage and considering the various uncertainties with the construction schedule. Due to the short construction windows that will be available, Alternative 1 using precast girders is recommended as it will accelerate construction and reduce the risk for schedule delays. This alternative would also mitigate the risk associated with the falsework needed for alternative 2 being washed out by a summer storm. During the next phase of the project, a more detailed study should be carried out to confirm shipping costs for the precast girders. Further refinement in span lengths may also lead to cost savings, such as using 3 equal spans to reduce the maximum length of the girders. Splicing the precast girders on-site with post-tensioning could also be investigated as a way to further reduce shipping costs.

Additionally, it is recommended that the abutment foundations be an early focus of the next phase of the project. The General Plan Estimates are based on larger and deeper piles than would typically be required to reflect the recommendations of Caltrans Memo to Designers 20-14, which recommends a 200%-500% cost increase for foundations impacted by liquefaction and lateral spreading. However, a detailed assessment of the foundations for combined liquefaction, lateral spreading and scour was beyond the scope of this project, and therefore, there is a high degree of uncertainty associated with the design and cost estimates of the abutments at this stage in the project.

ATTACHMENT A

General Plans of Alternatives

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
01	HUM	X	X	####	####

REGISTERED CIVIL ENGINEER DATE

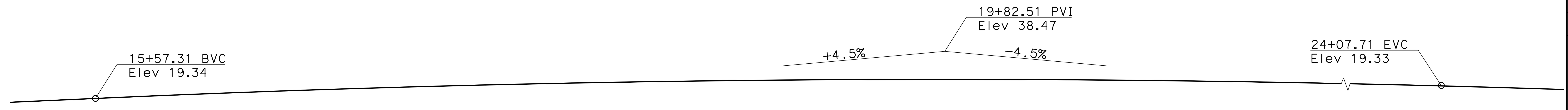
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER MATTHEW KLEYMANN No. 91100 Exp. 3/31/26 CIVIL STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

COUNTY OF HUMBOLDT - DEPARTMENT OF PUBLIC WORKS
1106 SECOND STREET
EUREKA, CALIFORNIA 95501

MARK THOMAS
701 UNIVERSITY AVENUE, SUITE 200
SACRAMENTO, CALIFORNIA 95825



PROFILE GRADE
NO SCALE

ELEVATION
1" = 30'
(BENTS FOR ALTERNATIVE 1 SHOWN, SEE TYPICAL CROSS SECTION SHEETS FOR BENT FOR ALTERNATIVES 2 AND 3)

PLAN
1" = 30'

LEGEND

Indicates New Construction

Indicates Existing Structure

DRAFT - NOT FOR CONSTRUCTION

DESIGN OVERSIGHT	DESIGN	BY X	CHECKED X	SEISMIC ANALYSIS	BY X	CHECKED X	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	MATTHEW KLEYMANN PROJECT ENGINEER
	DETAILS	BY X	CHECKED X	LAYOUT	BY X	CHECKED X		
SIGN OFF DATE	QUANTITIES	BY X	CHECKED X	SPECIFICATIONS	BY X	PLANS AND SPECS COMPARED X		

BRIDGE No. XX-XXXX

POST MILE XX.X

HAMMOND TRAIL BRIDGE

GENERAL PLAN - ALTERNATIVE 1 TO 3

STRUCTURES DESIGN GENERAL PLAN SHEET (ENGLISH) (REVISION 2/8/2022)

DATE PLOTTED => 11/6/2024 FILE => HTB_GP.dwg

TIME PLOTTED => 1:24 PM USERNAME => VDOMINGUEZ

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PROJECT NUMBER & PHASE: X

COUNTY/ROUTE: XXX/XXX
CONTRACT No.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 1/2/34

SHEET 1 OF 3

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
01	HUM	X	X	####	####

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER MATTHEW KLEYMANN No. 91100 Exp. 3/31/26 CIVIL STATE OF CALIFORNIA

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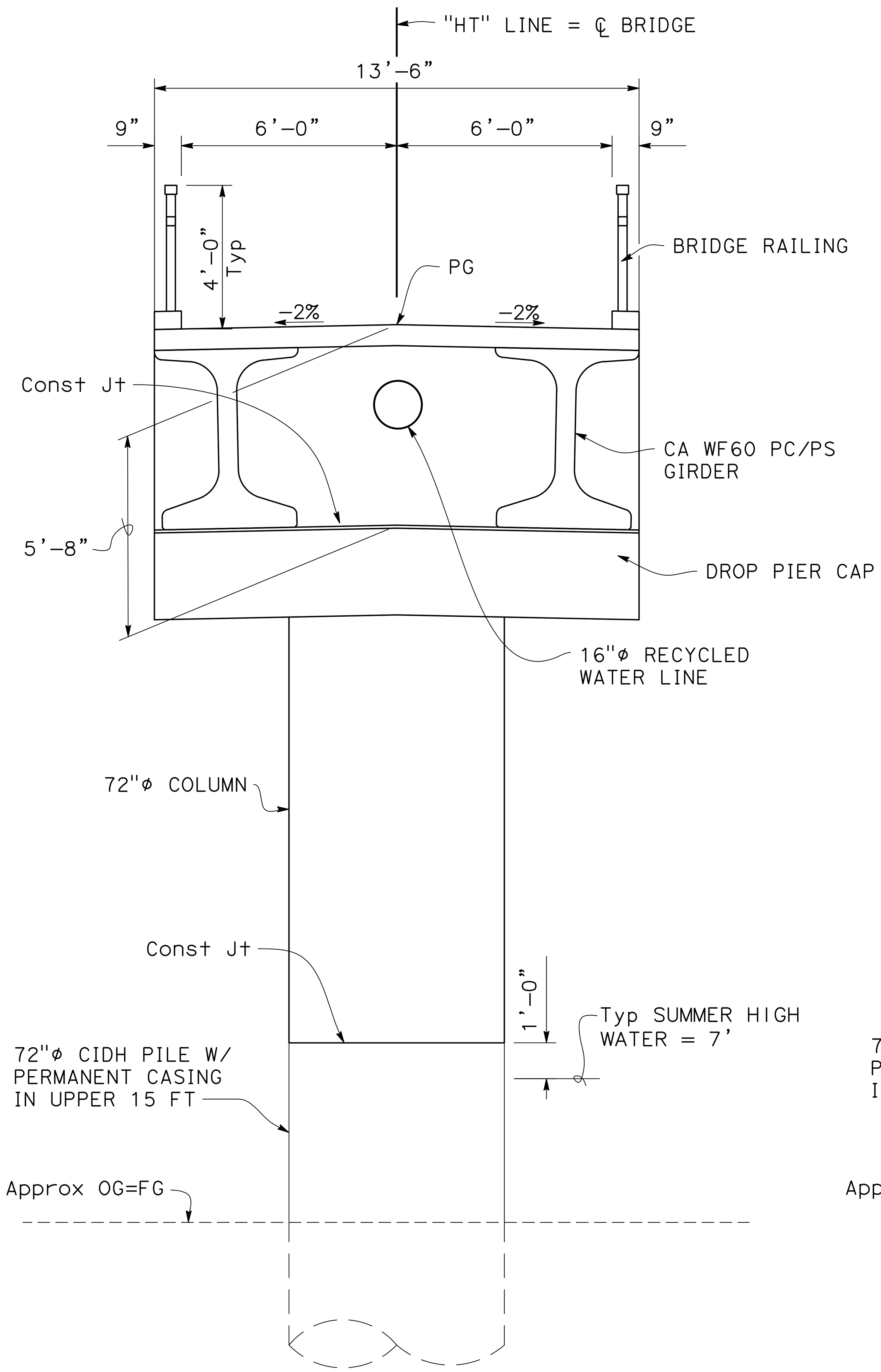
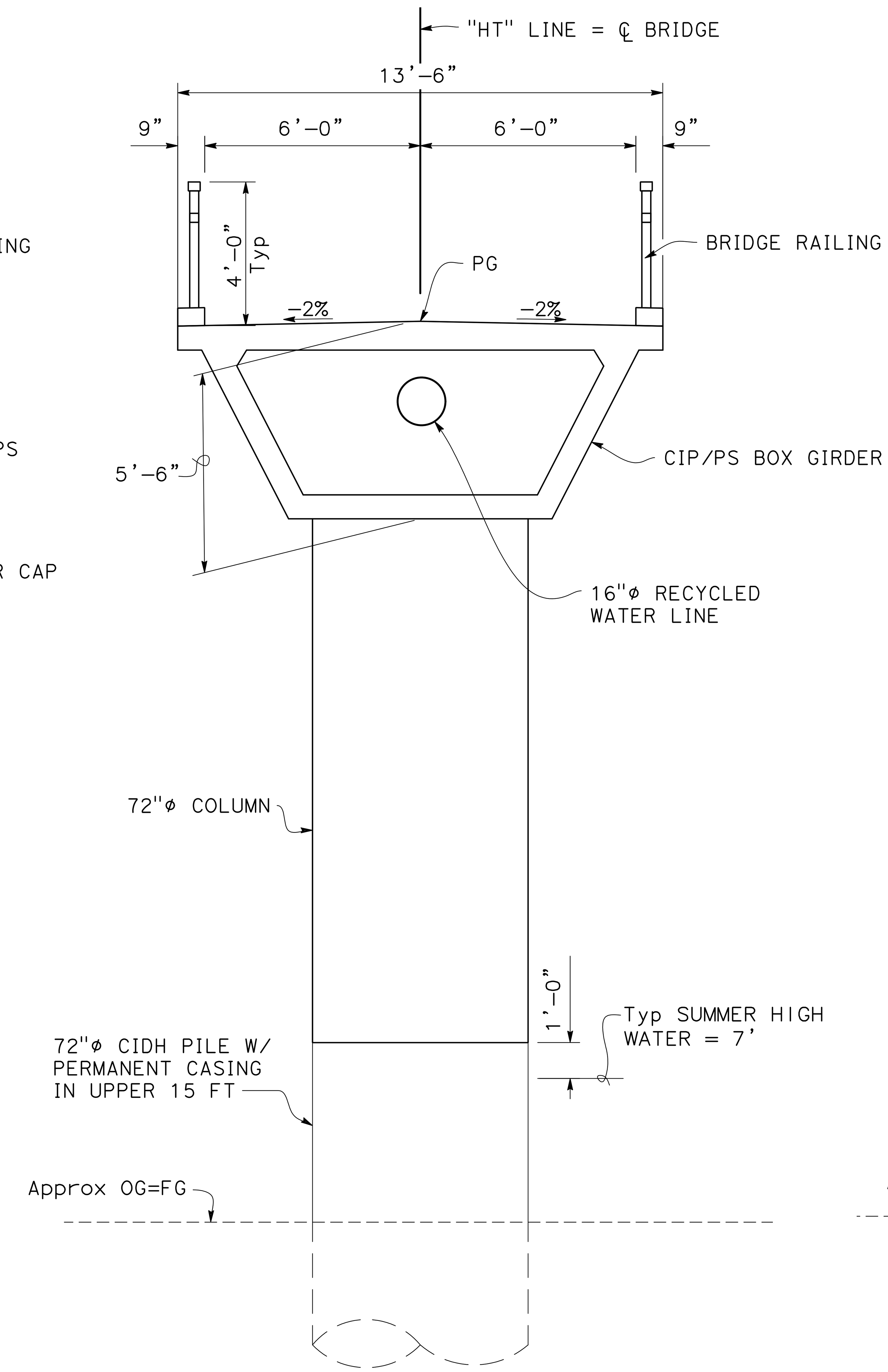
COUNTY OF HUMBOLDT - DEPARTMENT OF PUBLIC WORKS
1106 SECOND STREET
EUREKA, CALIFORNIA 95501

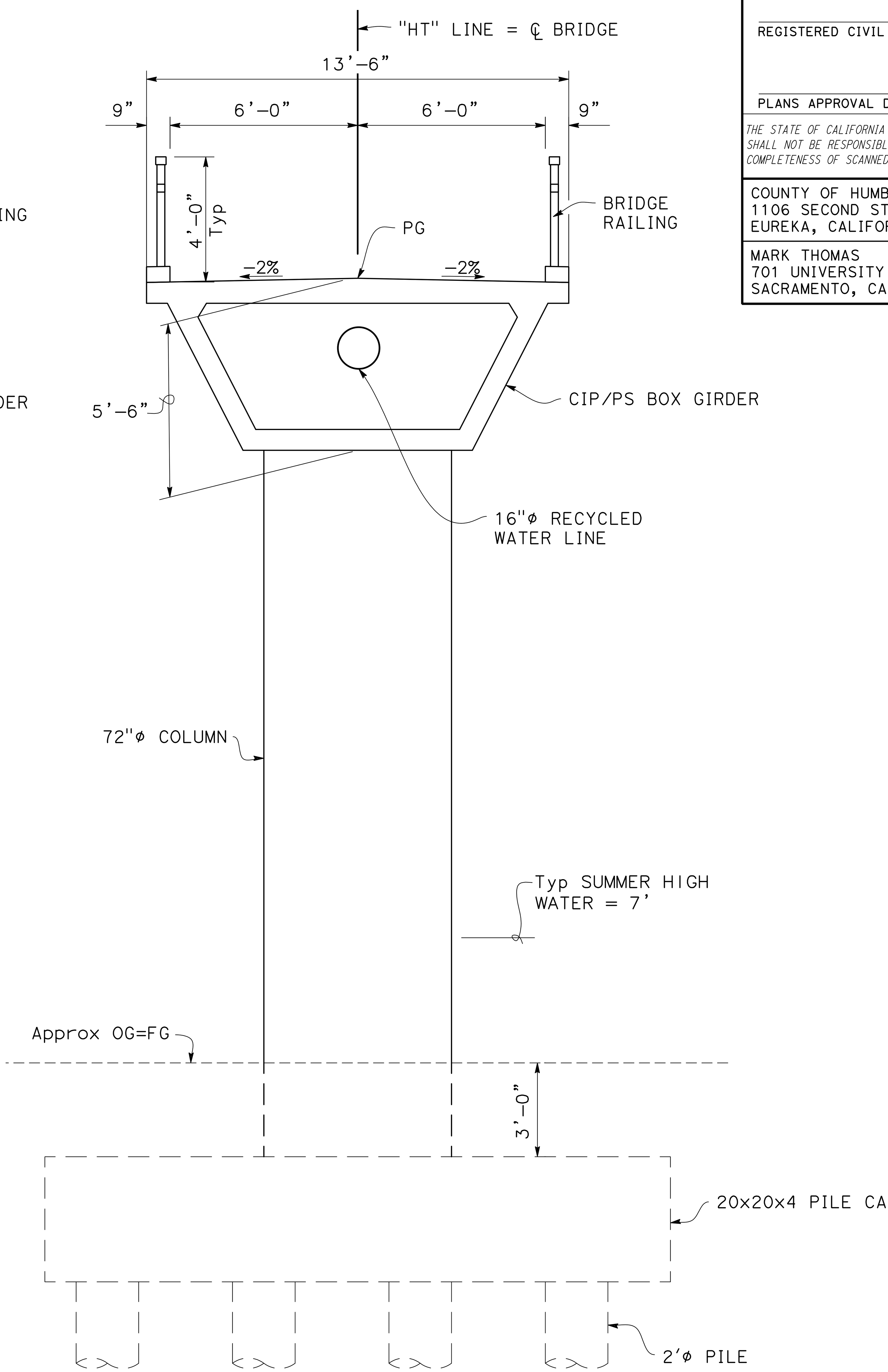
MARK THOMAS
701 UNIVERSITY AVENUE, SUITE 200
SACRAMENTO, CALIFORNIA 95825



TYPICAL SECTION - ALT. 1
3/8" = 1'-0"



TYPICAL SECTION - ALT. 2
3/8" = 1'-0"



TYPICAL SECTION - ALT. 3
3/8" = 1'-0"

DRAFT - NOT FOR CONSTRUCTION

DESIGN OVERSIGHT	DESIGN	BY X	CHECKED X	SEISMIC ANALYSIS	BY X	CHECKED X	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
	DETAILS	BY X	CHECKED X	LAYOUT	BY X	CHECKED X	
SIGN OFF DATE	QUANTITIES	BY X	CHECKED X	SPECIFICATIONS	BY X	PLANS AND SPECS COMPARED X	

MATTHEW KLEYMANN
PROJECT ENGINEER

BRIDGE No. XX-XXXX
POST MILE XX.X

HAMMOND TRAIL BRIDGE
TYPICAL SECTIONS ALTERNATIVE 1 TO 3

STRUCTURES DESIGN GENERAL PLAN SHEET (ENGLISH) (REVISION 2/8/2022)

DATE PLOTTED => 11/6/2024 TIME PLOTTED => 1:24 PM
FILE => HTB_GP.dwg USERNAME => VDOMINGUEZ

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PROJECT NUMBER & PHASE: X

COUNTY/ROUTE: XXX/XXX
CONTRACT No.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 1/2/34

SHEET 2 OF 3

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
01	HUM	X	X	####	####

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER — MATTHEW KLEYMANN — No. 91100 — Exp. 3/31/26 — CIVIL — STATE OF CALIFORNIA

COUNTY OF HUMBOLDT - DEPARTMENT OF PUBLIC WORKS
1106 SECOND STREET
EUREKA, CALIFORNIA 95501

MARK THOMAS
701 UNIVERSITY AVENUE, SUITE 200
SACRAMENTO, CALIFORNIA 95825

PILE DATA TABLE

Location	Pile Type	Nominal Resistance (kips)		Cut-off Elevation (ft)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression	Tension			
Abut 1	24" CIDH	TBD	TBD	TBD	TBD	TBD
PIER 2	72" CIDH	2510	N/A	8	-69 (a-I) -98 (a-II) -153 (a-III)	-153
PIER 3	72" CIDH	2510	N/A	8	-69 (a-I) -98 (a-II) -153 (a-III)	-153
Abut 4	24" CIDH	TBD	TBD	TBD	TBD	TBD

NOTES:
1. Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (a-III) Compression (Extreme Event - Downdrag).
2. Do not raise specified Tip Elevation.

SCOUR DATA TABLE

Support Location	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)	Approx Mudline (Elev)
Abut 1	N/A	N/A	N/A
PIER 2	2.4	-10.5	3.0
PIER 3	5.4	-7.5	6.0
Abut 4	N/A	N/A	N/A

BENCHMARK AND DATUM

MONUMENT	COORDINATES		ELEVATION	DESCRIPTION/LOCATION
	NORTH	EAST		

HYDROLOGIC SUMMARY

FOR BRIDGE No. XX-XXXX

Drainage Area: XX Square Miles

Frequency	Design Flood	Base Flood	Flood of Record
	50-year	100-year	1964
Discharge (cubic feet per second)	81,270	90,960	81,000
Water Surface Elev at Bridge (feet)	19.85	20.25	19.84

Flood plain data is based upon information available when the plans were prepared and is shown to meet federal requirements. The accuracy of said information is not warranted by Caltrans and interested or affected parties should make their own investigation.

UTILITIES

1. Exist Underground 16" D.I.P. recycled waterline
2. Exist 16" D.I.P. recycled waterline on Exist Bridge (Daylights near Existing abutment)
3. Exist 8" D.I.P. recycled water discharge pipe
4. New Underground 16" D.I.P. recycled waterline
5. New 16" D.I.P. recycled waterline on new bridge

LEGEND

XXX.X — Bottom of Footing Elevation (feet)

○ Indicates 42"ϕ CIDH Pile

○ Indicates 72"ϕ CIDH Pile

— — — Indicates Existing Structure

PLAN

1" = 30'

DRAFT - NOT FOR CONSTRUCTION

X	SCALE: X	VERT.DATUM NAVD88	HORZ.DATUM NAD83
DESIGN OVERSIGHT	PHOTOGRAMMETRY AS OF: X	ALIGNMENT TIES: X	
X	SURVEYED BY X	DRAFTED BY X	
SIGN OFF DATE	FIELD CHECKED BY X	CHECKED BY X	

DESIGN	BY X	CHECKED X
DETAILS	BY X	CHECKED X
QUANTITIES	BY X	CHECKED X

PREPARED FOR THE **STATE OF CALIFORNIA** DEPARTMENT OF TRANSPORTATION

MATTHEW KLEYMANN
PROJECT ENGINEER

BRIDGE No. X
POST MILE X

HAMMOND TRAIL BRIDGE

FOUNDATION PLAN - ALTERNATIVE 1 & 2

ATTACHMENT B

General Plan Estimates

MARK THOMAS

BRIDGE GENERAL PLAN ESTIMATE ☑ **OR PLANNING ESTIMATE** ☐

STRUCTURE Hammond Trail Pedestrian Bridge	BRIDGE NO			
TYPE Alt 1 - PC/PS WF Girders w/ CIDH Piles	DIST 1	CO HUM	RTE	PM
LENGTH 400.0	x WIDTH 13.5	= AREA 5400		SQ FT
QUANTITIES BY M. Kleymann	DATE 7/1/2024	CHECKED BY S. Varela		DATE 7/26/2024
PRICED BY	DATE			

		CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	600097	BRIDGE REMOVAL	LS	1	$500,000	$500,000
2	19XXXX	CONSTRUCTION BERM INSTALLATION (3X)	CY	3,520	$150	$528,000
3	19XXXX	CONSTRUCTION BERM EXCAVATION (3X)	CY	3,520	$75	$264,000
4	190101	ROADWAY EXCAVATION	CY	1,108	$50	$55,407
5	192003	STRUCTURE EXCAVATION (BRIDGE)	CY	147	$219	$32,120
6	260203	CLASS 2 AGGREGATE BASE (CY)	CY	116	$120	$13,867
7	390132	HOT MIX ASPHALT (TYPE A)	CY	48	$300	$14,444
8	490592	72" PERMANENT STEEL CASING	LF	30	$3,700	$111,000
9	490606	42" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	1,080	$1,500	$1,620,000
10	490611	72" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	300	$3,000	$900,000
11	510051	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	93	$1,600	$149,333
12	510053	STRUCTURAL CONCRETE, BRIDGE	CY	138	$2,800	$387,677
13	510054	STRUCTURAL CONCRETE, BRIDGE (POLYMER FIBER)	CY	133	$1,800	$238,919
14	512282	FURNISH PRECAST PRESTRESSED CONCRETE WIDE FLANGE GIRDER (120 TO 145')	EA	6	$55,000.00	$330,000
15	512401	ERECT PRECAST CONCRETE GIRDER	EA	6	$40,000	$240,000
16	519093	JOINT SEAL ASSEMBLY (MR 3")	LF	28	$1,000	$28,000
17	520102	BAR REINFORCING STEEL (BRIDGE)	LB	228,059	$2.20	$501,730
18	723030	ROCK SLOPE PROTECTION (1/2 T, CLASS VII, METHOD A) (CY)	CY	889	$325.00	$288,889
19	833088	TUBULAR HANDRAILING	LF	880	$600	$528,000
20		16" DUCTILE IRON PIPE	LF	600	$600	$360,000

SUBTOTAL	$7,091,387
MOBILIZATION (10 %)	787,932
SUBTOTAL COST ITEMS	$7,879,319
CONTINGENCIES (20 %)	1,575,864
PROJECT TOTAL ($ 1,750.96 /SQ FT)	$ 9,455,183
GRAND TOTAL IN 2024 DOLLARS	**$ 9,455,183**
FOR BUDGET PURPOSES - USE	$ 9,460,000
ANNUAL ESCALATION OF 5% FOR 7 YEARS	$ 3,784,000
TOTAL AT MID POINT OF CONSTRUCTION	**$ 13,244,000**

COMMENTS Costs do not include permits, right-of-way, design fees, CM fees

MARK THOMAS

BRIDGE GENERAL PLAN ESTIMATE ☑ **OR PLANNING ESTIMATE** ☐

STRUCTURE Hammond Trail Pedestrian Bridge	BRIDGE NO			
TYPE Alt 2 - PS CIP/PS Box Girder w/ CIDH Piles	DIST 1	CO HUM	RTE	PM
LENGTH 400.0	x WIDTH 13.5	= AREA 5400	SQ FT	
QUANTITIES BY M. Kleymann	DATE 7/1/2024	CHECKED BY		DATE
PRICED BY	DATE			

		CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	600097	BRIDGE REMOVAL	LS	1	$500,000	$500,000
2	19XXXX	CONSTRUCTION BERM INSTALLATION (3X)	CY	3,520	$150	$528,000
3	19XXXX	CONSTRUCTION BERM EXCAVATION (3X)	CY	3,520	$75	$264,000
4	190101	ROADWAY EXCAVATION	CY	1,108	$50	$55,407
5	192003	STRUCTURE EXCAVATION (BRIDGE)	CY	147	$219	$32,120
6	260203	CLASS 2 AGGREGATE BASE (CY)	CY	116	$120	$13,867
7	390132	HOT MIX ASPHALT (TYPE A)	CY	48	$300	$14,444
8	490592	72" PERMANENT STEEL CASING	LF	30	$3,700	$111,000
9	490606	42" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	1,080	$1,500	$1,620,000
10	490611	72" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	300	$3,000	$900,000
11	500001	PRESTRESSING CAST-IN-PLACE CONCRETE	LS	1	$77,585	$77,585
12	510051	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	93	$1,600	$149,333
13	510053	STRUCTURAL CONCRETE, BRIDGE	CY	279	$2,800	$781,755
14	510054	STRUCTURAL CONCRETE, BRIDGE (POLYMER FIBER)	CY	159	$1,800	$286,560
15	519093	JOINT SEAL ASSEMBLY (MR 3")	LF	28	$1,000	$28,000
16	520102	BAR REINFORCING STEEL (BRIDGE)	LB	251,126	$2.20	$552,477
17	723030	ROCK SLOPE PROTECTION (1/2 T, CLASS VII, METHOD A) (CY)	CY	889	$325.00	$288,889
18	833088	TUBULAR HANDRAILING	LF	880	$600	$528,000
19		16" DUCTILE IRON PIPE	LF	600	$600	$360,000

SUBTOTAL	$7,091,439
MOBILIZATION (10 %)	787,938
SUBTOTAL COST ITEMS	$7,879,377
CONTINGENCIES (20 %)	1,575,875
PROJECT TOTAL ($ 1,750.97 /SQ FT)	$9,455,252
GRAND TOTAL IN 2024 DOLLARS	**$ 9,455,252**
FOR BUDGET PURPOSES - USE	$ 9,460,000
ANNUAL ESCALATION OF 5% FOR 7 YEARS	$ 3,784,000
TOTAL AT MID POINT OF CONSTRUCTION	**$ 13,244,000**

COMMENTS Costs do not include permits, right-of-way, design fees, CM fees

MARK THOMAS

BRIDGE GENERAL PLAN ESTIMATE ☑ **OR PLANNING ESTIMATE** ☐

STRUCTURE Hammond Trail Pedestrian Bridge	BRIDGE NO			
TYPE Alt 3 - PS CIP Box Girder with Pile Cap Footing	DIST 1	CO HUM	RTE	PM
LENGTH 400.0	x WIDTH 13.5	= AREA 5400	SQ FT	
QUANTITIES BY M. Kleymann	DATE 7/1/2024	CHECKED BY	DATE	
PRICED BY	DATE			

		CONTRACT ITEMS	UNIT	QUANTITY	PRICE	AMOUNT
1	600097	BRIDGE REMOVAL	LS	1	$500,000	$500,000
2	19XXXX	CONSTRUCTION BERM INSTALLATION (3X)	CY	3,520	$150	$528,000
3	19XXXX	CONSTRUCTION BERM EXCAVATION (3X)	CY	3,520	$75	$264,000
4	190101	ROADWAY EXCAVATION	CY	1,108	$50	$55,407
5	192003	STRUCTURE EXCAVATION (BRIDGE)	CY	265	$219	$58,076
6	260203	CLASS 2 AGGREGATE BASE (CY)	CY	116	$120	$13,867
7	390132	HOT MIX ASPHALT (TYPE A)	CY	48	$300	$14,444
8	480600	TEMPORARY SHORING	LS	1	$201,600	$201,600
9	490606	42" CAST-IN-DRILLED-HOLE CONCRETE PILING	LF	1,080	$1,500	$1,620,000
10	495115	FURNISH 24" CAST-IN-STEEL SHELL CONCRETE PILING	LF	3,200	$275	$880,000
11	495116	DRIVE 24" CAST-IN-STEEL SHELL CONCRETE PILE	EA	32	$14,000	$448,000
12	500001	PRESTRESSING CAST-IN-PLACE CONCRETE	LS	1	$77,585	$77,585
13	510051	STRUCTURAL CONCRETE, BRIDGE FOOTING	CY	212	$1,600	$338,963
14	510053	STRUCTURAL CONCRETE, BRIDGE	CY	279	$2,800	$781,755
15	510054	STRUCTURAL CONCRETE, BRIDGE (POLYMER FIBER)	CY	159	$1,800	$286,560
16	519093	JOINT SEAL ASSEMBLY (MR 3")	LF	28	$1,000	$28,000
17	520102	BAR REINFORCING STEEL (BRIDGE)	LB	239,582	$2.20	$527,081
18	723030	ROCK SLOPE PROTECTION (1/2 T, CLASS VII, METHOD A) (CY)	CY	889	$325.00	$288,889
19	833088	TUBULAR HANDRAILING	LF	880	$600	$528,000
20		16" DUCTILE IRON PIPE	LF	600	$600	$360,000

SUBTOTAL	$7,800,228
MOBILIZATION (10 %)	866,692
SUBTOTAL COST ITEMS	$8,666,920
CONTINGENCIES (20 %)	1,733,384
PROJECT TOTAL ($ 1,925.98 /SQ FT)	$ 10,400,304
GRAND TOTAL IN 2024 DOLLARS	**$ 10,400,304**
FOR BUDGET PURPOSES - USE	$ 10,410,000
ANNUAL ESCALATION OF 5% FOR 7 YEARS	$ 4,164,000
TOTAL AT MID POINT OF CONSTRUCTION	**$ 14,574,000**

COMMENTS Costs do not include permits, right-of-way, design fees, CM fees

ATTACHMENT C

Preliminary Foundation Report

PRELIMINARY FOUNDATION REPORT

Hammond Trail Pedestrian Bridge

Task Order No. DPW2021-001-T09
Humboldt County, California

Prepared by:

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Crawford & Associates, Inc.
4701 Freeport Boulevard
Sacramento, CA 95822

July 26, 2024

Prepared for:

Mark Thomas & Company, Inc.
701 University Ave, Suite 200
Sacramento, CA 95825

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Sacramento
Eureka
Modesto
Pleasanton
Santa Rosa
Seattle
Ukiah

July 26, 2024
Crawford File No. 23-948.9

Sebastian Varela, PhD, PE, SE
Sr. Structures Technical Lead Engineer, Mark Thomas
701 University Ave, Suite 200
Sacramento, CA 95825

Subject: **Preliminary Foundation Report**
Hammond Trail Pedestrian Bridge
Task Order No. DPW2021-001-T09
Humboldt County, California

Crawford & Associates, Inc. (Crawford) prepared this Preliminary Foundation Report for the Hammond Trail Pedestrian Bridge project in Humboldt County, California. The report was prepared in accordance with Subcontract No. 24-00032 between Mark Thomas & Company, Inc. and Crawford, dated August 18, 2023.

This report provides a summary of the anticipated subsurface conditions at the site, based on existing subsurface data, and preliminary foundation recommendations for a replacement bridge structure to assist with the preparation of type selection. Once the foundation type(s) are selected and the foundation data and loading are fully defined, a design-level foundation report can be prepared.

Thank you for the opportunity to be part of your team. Please contact Crawford if you have questions or require additional information.

Sincerely,

Crawford & Associates, Inc.,

Ryan Houghton, PE
Senior Engineer

Reviewed by:

W. Eric Nichols, CEG, PE
Principal

REGISTERED PROFESSIONAL ENGINEER
RYAN R. HOUGHTON
C 87322
CIVIL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
WILLIAM E. NICHOLS
NO. 2229
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
WILLIAM E. NICHOLS
No. 82103
CIVIL
STATE OF CALIFORNIA

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APPENDIX A

Figure 1: Site Vicinity
Figure 2: Geologic Map
Figure 3: Fault Map
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APPENDIX B

Existing Subsurface Data (SHN, 2014-2015)

APPENDIX C

Shear Wave Velocity Calculations
Liquefaction Triggering and Seismic Settlement Calculations

APPENDIX D

Preliminary Geotechnical Parameters

APPENDIX E

Preliminary Axial Capacity – 72-inch CIDH Pier (Northern Channel)

1 INTRODUCTION

1.1 PURPOSE

Crawford prepared this Preliminary Foundation Report (PFR) for the Hammond Trail Pedestrian Bridge project in Humboldt County, California. This report provides preliminary foundation recommendations for use in type selection of a replacement bridge structure.

Following type selection, Crawford can complete additional subsurface exploration of the site (as recommended in Section 12.3). Based on the data obtained from the additional exploration (along with the existing subsurface data), design level geotechnical evaluation and analysis will be completed, and a Foundation Report (FR) will be prepared with recommendations for final design of the selected structure type and the associated foundation data and loading.

1.2 SCOPE OF GEOTECHNICAL SERVICES

To prepare this Preliminary Foundation Report, Crawford:

- discussed the project with the design team from Mark Thomas & Company, Inc. (Mark Thomas);
- reviewed the previous Type Selection Study[1] for the project, dated June 2011;
- reviewed the previous Foundation Report[2] for the project, dated May 2015;
- reviewed as-built plans of the existing Hammond Trail Bridge, dated 1979-1980;
- reviewed as-built plans of the railroad bridge (portion of the existing Hammond Trail Bridge that spans across the Mad River), dated 1928 and 1941;
- reviewed preliminary bridge alterative sketches provided by Mark Thomas on April 30, 2024;
- reviewed draft General Plan sheets for Alterative 1, provided by Mark Thomas on July 12, 2024;
- reviewed published topographic, geologic, and geohazards mapping pertinent to the project site; and
- performed preliminary geotechnical engineering evaluation and analysis to develop the preliminary recommendations contained in this report.

Limitations of this report are discussed in the Section 13.

1.3 PROJECT DATUM

All elevations referenced in this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted.

[1] Type Selection Study for the Hammond Trail Pedestrian Bridge over the Mad River, Humboldt County, California, Morrison Structures, Inc., June 2011
[2] Final Foundation Report, Hammond Trail Pedestrian Bridge Replacement Mad River Crossing, North of Arcata, Humboldt County, California, prepared by SHN Consulting Engineers & Geologists, Inc., May 2015

2 PROJECT DESCRIPTION

2.1 PROJECT LOCATION

The project site is located about 1.8 miles southwest of McKinleyville, Humboldt County, California. The existing bridge conveys Hammond Trail over the Mad River, located between Mad River Road (to the south) and Fischer Avenue (to the north) and about 1 mile inland of the Pacific Ocean. Project site coordinates are about latitude 40.9241° and longitude -124.1204°. Refer to Appendix A – Figure 1 for a Vicinity Map of the project site.

2.2 PROPOSED PROJECT

The proposed project will replace the existing bridge along an adjacent parallel alignment offset approximately 45 feet to the east (upstream). The replacement bridge structure is anticipated to convey a 12-foot-wide path across the river, with an overall width of 13.5 feet. Crawford understands that the following alternates have been developed for this project:

- **Alternative 1** is a 400-foot long, three-span bridge consisting of either a precast, prestressed (PC/PS) concrete girder or a cast-in-place (CIP) concrete box girder superstructure. The end spans are about 127.5 feet long, and the center span is about 145 feet long. Two piers are located within the river channel and the abutments are located on top of the banks, protected by rock slope protection (RSP). Crawford understands this is the preferred alternative currently, with the PC/PS girder superstructure as the preferred superstructure.

- **Alternative 2** is a 400-foot long, two-span (200-foot span lengths) bridge with a steel box girder superstructure. The pier is located within the center of the river channel and the abutments are located on top of the banks, protected by RSP.

- **Alternative 3** is a 545-foot long, four-span bridge consisting of either a PC/PS concrete girder or a CIP concrete box girder superstructure. The end spans are about 127.5 feet each and the intermediate spans are about 145 feet each. Three piers are located within the river channel. The abutments are setback from the top of bank so that RSP is not required.

A fourth alternative to replace the bridge superstructure and reuse the existing foundations was also considered, but it was ultimately eliminated from consideration due to the lack of as-built foundation data and thus the inability to assess the existing foundations. It is presumed based on the age of the structure (sections built in 1941 and 1981) and the lack of documentation that the bridge is not designed to modern seismic requirements, and it is likely not designed to withstand potential seismic hazards, including strong ground motions, deep liquefaction, lateral spreading, and tsunami impact/inundation (refer to Section 11 for detail).

3 EXCEPTIONS TO POLICIES AND PROCEDURES

There are no geotechnical design exceptions to Caltrans Departmental policies and procedures for this project.

4 GEOTECHNICAL INVESTIGATION (SHN 2014-2015)

SHN Consulting Engineers & Geologists (SHN) completed four borings at the project site in 2014 and 2015. The boring information is provided below in Table 1. Refer to Appendix B for SHN's boring logs and interpreted geologic cross section figure, which includes a plan view of the boring locations.

Table 1: Previous Subsurface Investigation Summary

Boring No.	Location	Completion Date	Drill Rig Type	Hammer Type[1]	Top Elevation (feet)	Boring Depth (feet)	Bottom Elev. (feet)
B-1	South Bank	07/01/2014	Not Available	Automatic	18	101.5	-83.5
B-2	North Bank	07/01/2014	Not Available	Automatic	18	101.5	-83.5
B-3	North Chanel	01/20/2015	Barge	Automatic	7[2]	201	-194
B-4	South Channel	01/22/2015	Barge	Automatic	9[2]	201	-192

1. SHN did not report the hammer energy; it was assumed to be 80%, which is typical for an automatic hammer.
2. The top elevation for the barge borings was referenced to the water surface elevation at high tide, which was about 7 feet from the channel bottom at each location. A tide gauge was not utilized to correct for surface water changes during drilling.

The borings were drilled primarily with mud rotary techniques; solid-stem augers were utilized in Borings B-1 and B-2 to depths of 7.5 and 5 feet, respectively. Soils were logged in general accordance with ASTM International (ASTM) Test Method D2488.

Soil samples were recovered by means of a 1.4-inch inside diameter (ID) Standard Penetration Test (SPT) split-spoon sampler, a 2.5-inch ID California Modified (MCS) split-spoon sampler, and a 3.0-inch ID Shelby Tube sampler. The SPT and MCS samplers were advanced with a standard 350 ft-lb striking force using a 140-lb automatic hammer and a drop height of 30-inches. The sampler penetration resistance (N-value) in blows per 0.5 feet foot (bpf) was recorded on the boring logs. The Shelby Tube sampler was hydraulicly pushed into the soil. Sampling was completed generally at 5 to 20-foot depth intervals.

5 LABORATORY TESTING

The following laboratory tests were completed by SHN on representative soil samples obtained from their borings completed in 2014 and 2015:

- Density by Drive-Cylinder Method (ASTM D2937)
- Percent Passing No. 200 Sieve (ASTM D1140)
- Liquid Limit, Plastic Limit, and Plasticity Index (ASTM D4318)
- Sieve Analysis (ASTM C136)
- Unconfined Compression-Soil (no ASTM test method noted)
- Consolidated Undrained Triaxial Test (ASTM D4767)
- Chemical Analysis Testing (California Test Method (CTM) 226, 417, 422, and 643)

6 GEOTECHNICAL CONDITIONS

6.1 GEOLOGY

The project site lies within the Coast Ranges Geomorphic Province[3], that is characterized by a series of northwest-trending mountain ranges with intermountain valleys and sub-parallel to the active San Andreas Fault Zone. The Coast Ranges is composed of thick Cenozoic sedimentary and volcanic strata overlying Mesozoic metamorphic rock. The northern Coast Ranges are dominated by the irregular, knobby, landslide-topography of the Franciscan Complex.

Published geologic mapping[4] (Appendix A – Figure 2) shows the site immediately underlain by Holocene- to late Pleistocene-age Alluvial Deposits (Qal). This unit is typically comprised of clay, silt, sand, gravel, and boulders deposited in stream beds, terraces, and flood plains. SHN interpreted that this formation extended to depths of about 60 to 75 feet below the surface. The older sediments (sand and gravel) below this formation to the full depth explored were interpreted by SHN as middle Pleistocene to middle Pliocene-age Falor Formation, which is included within the Marine and Nonmarine Overlap Deposits (QTw) shown on Figure 2 to the east of the project site.

Based on the United States Geologic Survey (USGS) and California Geological Survey (CGS) fault data and mapping[5,6] (Appendix A – Figure 3), the nearest active fault (defined as surface displacement within the last 11,000 years per CGS criteria) is a trace of the Holocene-age Mad River Fault Zone, located about 1,700 feet northeast of the site. The site is not located within an Alquist-Priolo Special Studies Zone[7]. Preliminary seismic design data is provided in Section 11.

6.2 SURFACE CONDITIONS

The existing bridge was originally built in 1941 to carry rail traffic across the river and was comprised of a steel truss span and a steel through girder span (overall length of about 380 feet) supported by piers with unknown foundation type/depths. In 1981 the existing bridge was repurposed to a pedestrian trail. Two approach spans (steel girder superstructure) were added to each end of the bridge (creating a 6-span, approximately 540-foot-long bridge structure).

The existing bridge conveys an approximately 8-foot-wide, mixed-use trail across the Mad River along a north to south alignment. The Mad River flows year-round and outlets into the Pacific Ocean about 1 mile to the west. The banks of the river are densely vegetated with trees and thick undergrowth. Immediately beyond the banks are essentially flat grass fields surrounded by wire fencing, which appear to be used for cattle grazing.

Based on discussions with Mark Thomas, the existing bridge has experienced severe corrosion of the truss and through girder span members. Currently, it is not believed that any primary truss or through girder members have been compromised to the point the structure is unsafe for

[3] California Geologic Survey, California Geomorphic Provinces, Note 36, 2002
[4] McLaughlin et al., 2000, Geology of the Cape Mendocino, Eureka, Garberville, and southwestern part of the Hayfork 30x60 minute quadrangles and adjacent offshore area, northern CA, with digital database, USGS, MF-2336, 1:100,000
[5] United States Geologic Survey, U.S. Quaternary Faults GIS data
[6] California Geologic Survey, 2010 Fault Activity Map of California GIS data
[7] http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

pedestrian traffic. However, the existing steel floor beams supporting the pedestrian walkway have severely corroded and were supplemented by wood timbers in 2011.

A 16-inch recycled water line owned by the McKinleyville Community Service District is carried across the Mad River by the existing bridge.

6.3 SUBSURFACE CONDITIONS

Based on SHN's boring data, the subsurface materials underlying the site are divided into two general material units, as summarized below. Refer to Appendix B for SHN's interpreted geologic cross section figure of the site.

Unit 1 – Holocene Alluvium:
This unit extended to depths of about 60 to 75 feet from the ground surface (or the channel bottom for the barge borings), which corresponds to an elevation range of about -42 to -63 feet. The materials generally consisted of very loose to medium dense silts, sands, and gravels, with minor amounts of clay. The upper 10 feet of B-2 was classified as historical fill. Radiocarbon dating on wood debris collected from B-1 at elevation -52 feet was estimated to be about 6,690 years old.

Unit 1 is considered consistent with the mapped geologic unit Qal discussed in Section 6.1.

Unit 2 – Pleistocene Falor Formation(?):
Materials of this unit were encountered in all the SHN borings below Unit 1 and extended to the maximum depth explored of 201 feet (elevation -194 feet). The materials in this unit generally consisted of dense to very dense sand and gravel. Some layers contained cobbles. Starting at elevations between -62 to -72 feet, SPT blow counts were greater than 50 blows per foot (and generally reached sampler refusal).

Unit 2 appears to be older, consolidated alluvium, which is considered consistent with the mapped geologic unit QTw discussed in Section 6.1 (identified by SHN as Falor Formation).

7 GROUNDWATER

SHN recorded groundwater at a depth of approximately 15 feet (elevation 3 feet) in Borings B-1 and B-2 in July 2014. Borings B-3 and B-4 were completed in January 2015 and drilled over the water from a barge. SHN reported the surface water level at elev. 0 and that the water level within the channel varied by approximately 4 feet due to tidal fluctuations. The groundwater levels encountered/recorded in the geotechnical test borings completed by SHN are shown in Table 2.

Table 2: Summary of Groundwater

Boring Location	Boring Identification	Ground Surface Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Date Measured
North Bank	B-2	18.0	15.0	3.0	07/14/14
North Channel	B-3	Not Measured. Boring drilled over-water from barge.			
South Channel	B-4	Not Measured. Boring drilled over-water from barge.			
South Bank	B-1	18.0	15.0	3.0	07/02/14

Groundwater at the project site is expected to generally coincide with the surface water elevation of the river. The surface water elevation is expected to fluctuate over time due to seasonal changes and tidal influence. The groundwater level used for preliminary design is elevation 3 feet for borings on the banks of the river and the bottom of channel for the barge borings.

8 AS-BUILT FOUNDATION DATA

As-built foundation information was limited for the existing bridge. The pier foundations of the railroad bridge (1941) are indicated to be supported by piles (type not specified), but pile depths/lengths are not provided. Available plans show that in 1981 steel HP10x42 piles driven to "20 tons each" were installed at the piers and abutments to support the approach spans; however, there was no available record of the actual pile depths/lengths.

9 SCOUR DATA

Crawford understands that the hydraulic study for this project is currently in progress, with preliminary scour results provided below in Table 3 (sourced from the draft Foundation Plan for Alterative 1 provided by Mark Thomas).

Table 3: Preliminary Scour Data

Support No.	Approximate Mudline Elev. (feet)	Long Term Scour (Degradation and Contraction) Elevation (feet)	Short Term Scour (Local) Depth (feet)
Abut 1	N/A	N/A	N/A
Bent 2	3.0	2.4	-10.0
Bent 3	6.0	5.4	-7.0
Abut 4	N/A	N/A	N/A

10 CORROSION EVALUATION

SHN completed corrosion testing on "two composited soil samples collected from the upper fine-grained alluvial soils encountered in Borings B-1 and B-3." The depths of the samples were not noted. Results of SHN's corrosion tests are summarized in Table 4.

Table 4: Soil Corrosion Test Summary

Boring / Sample	Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	Corrosive
B-1 / Composite	N/A	6.98	260	490	990	No
B-3 / Composite	N/A	7.49	330	880	200	Yes

For structural elements, Caltrans[8] defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1,500 ppm or greater, or has a pH of 5.5 or less. Except for MSE wall design, Caltrans does not include the minimum resistivity as a parameter to define corrosive area for structures, and soil and water are not required to be tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

Based on the test results summarized above and the 2021 Caltrans guidelines, the site is considered corrosive to structural concrete/steel foundation elements based on chloride concentration. The project is in a marine environment and a tidal channel that is considered corrosive, and the design of reinforced concrete and steel foundation elements should consider potential exposure to corrosive salt water and marine atmosphere.

The designer should consult with a corrosion engineer if the above test result values are considered significant. Section 12 of Caltrans' Corrosion Guidelines (Version 3.2) provides information regarding corrosion mitigation measures for structural elements and lists additional Caltrans guideline documents regarding corrosion mitigation.

11 SEISMIC INFORMATION

11.1 SHEAR WAVE VELOCITY AND CLASSIFICATION OF SOILS

A correlated shear wave velocity (V_{S30}) in the upper 100 feet of the soil profile equal to 180 meters per second (about 591 feet per second) was used for preliminary seismic analysis. The V_{S30} value was determined based on the subsurface data obtained from SHN's borings and correlations with SPT blow count N-values corrected for hammer efficiency using equations outlined by Caltrans[9]. Site coordinates of latitude 40.9241° and longitude -124.1204° were used for analysis.

The correlated V_{S30} values estimated from SHN's boring logs are shown in Table 5. The shear wave velocity calculations (input data/output results) for each boring are included in Appendix C.

[8] Caltrans, Corrosion Guidelines Version 3.2, May 2021
[9] Empirical Correlations for Estimating Shear Wave Velocity, Caltrans Geotechnical Manual, Design Acceleration Response Spectrum, Attachment 2, January 2021.

Table 5: Correlated Shear Wave Velocity

Boring Designation	Top of Boring Elevation (feet)	Bottom of Boring Elevation (feet)	Total Boring Depth (feet)	Correlated Shear Wave Velocity in Upper 100 ft	
				V_{S30} (m/s)	V_{S30} (ft/s)
B-1	18	-83.5	101.5	166	545
B-2	18	-83.5	101.5	210	689
B-3	7	-194	201	168	551
B-4	9	-192	201	211	692
			Average V_{S30} =	189	619
			Design V_{S30} =	**180**	**591**

11.2 SOIL CLASSIFICATION

For seismic design, Caltrans classifies soil as either Class S1 or Class S2. The Class S1 soil classification represents competent soil. The Class S2 soil classification represents non-competent soils, including marginal soil, poor soil and soil susceptible to lateral spreading.

According to Caltrans' Seismic Design Criteria (SDC) Version 2.0, Class S1 soil must meet all the following criteria:

- Standard Penetration Test, $(N_1)_{60} \geq 30$ (Granular Soils)
- Undrained Shear Strength, s_u > 2,000 psf (Cohesive Soils)
- Shear Wave Velocity, VS30 > 886 ft/sec
- Not susceptible to liquefaction, lateral spreading, or scour

Soil that does not satisfy the requirements listed above is to be classified as Class S2 soil.

For soil classification, Crawford considered SHN's borings. Based on the boring data and criteria listed above, site soils are classified as Class S2 (non-competent). The simplified design method as specified in Section 6.2.3.2 of SDC is not allowed for piles founded in Class S2 soil and lateral analysis as specified in Section 6.2.4.2 of SDC is required.

11.3 GROUND MOTION HAZARD

11.3.1 METHODOLOGY

For preliminary evaluation, the Caltrans ARS Online (V3.1.0)[10] web-based tool was used to calculate the probabilistic acceleration response spectra for the site based on criteria outlined in Appendix B of Caltrans SDC.

The design spectrum was determined based on the Safety Evaluation Earthquake (SEE) spectrum for an ordinary bridge. A probabilistic evaluation approach was used to determine the

[10] https://arsonline.dot.ca.gov/, accessed 04/19/2024.

SEE design spectrum taken as the spectrum based on the 2014 USGS Seismic Hazard Map for the 5% in 50 years probability of exceedance (or 975-year return period).

Caltrans structure design practice requires an increase to spectra due to fault proximity (near-fault factor) and when the site is located over a deep sedimentary basin (basin factor). The near-fault adjustment factor is applied for locations with a site to rupture plane distance (Rrup) of 25 kilometers (15.6 miles) or less to the causative fault and is based on the deaggregated mean distance for spectral acceleration at a period of 1.0 second. The near-fault adjustment factor applies to this site, while the basin factor does not.

The mean magnitude value reported by ARS Online is not used in the ground motion calculation. It is included to support simplified liquefaction analysis and is obtained from a hazard deaggregation performed at the Peak Ground Acceleration (PGA).

11.3.2 RECOMMENDED SEISMIC DATA

The following preliminary seismic data presented herein is considered conditional. Due to the presence of liquefiable layers, a V_{s30} of 180 m/s was used for preliminary analysis, which is the lowest value allowed in the Caltrans ARS Online tool. Therefore, a site-specific seismic hazard analysis is recommended (consistent with Caltrans guidelines) to be completed for final design that would supersede the preliminary seismic data presented below.

Based on the above information, the Caltrans SDC v2.0 seismic design parameters are shown in Table 6. The Design Ground Motion Data Sheet presenting the SEE Design ARS data, curve, and other relevant information is attached as Figure 4 in Appendix A.

Table 6: Ground Motion Parameters

Site Parameters			Design Ground Motion Parameters[1] (Return Period = 975 years)			Soil Profile Class
Latitude (degrees)	Longitude (degrees)	Shear-Wave Velocity[2], V_{S30} (m/sec)	Horizontal Peak Ground Acceleration (g)	Deaggregated Mean Earthquake Moment Magnitude for PGA	Deaggregated Mean Site-to-Fault Distance for 1.0 Period Spectral Acceleration (kilometers)	
40.9241	-124.1204	180	1.03	8.63	14.8	S2

1. Based on the Caltrans web tool ARS Online (Version 3.1.0).
2. Shear wave velocity determined by SPT correlations.

11.4 OTHER SEISMIC HAZARDS

11.4.1 SURFACE FAULT RUPTURE

The site is not located within an Alquist–Priolo Earthquake Fault Zone (EFZ), or within 1,000 feet of an unzoned fault that is Holocene (11,000 years) in age or younger. Also, no faults with displacement in the last 15,000 years (Holocene-Latest Pleistocene age or younger) are mapped by the CGS or the USGS within or through the project area. Refer to Section 6.1 for additional discussion regarding nearby faults.

Per Caltrans' Memo to Designer 20-15, the structure is not considered susceptible to surface fault rupture hazard.

11.4.2 LIQUEFACTION EVALUATION

Soil liquefaction can occur when saturated, relatively loose sand and specific soft, fine-grained saturated soils (typically within the upper 50 to 70 feet) are subject to ground shaking strong enough to create soil particle separation that results from increased pore pressure. This separation and subsequent pore pressure dissipation can lead to decreased soil shear strength and settlement. Liquefaction is known to occur in soils ranging from low plasticity silts to gravels. However, soils most susceptible to liquefaction are clean sands to silty sands and non-plastic silts. Granular soils with SPT blow count $(N_1)_{60} \geq 30$, rock and most clay soil are not liquefiable.

Liquefaction susceptibility of a soil deposit is a function of the soil grain size, relative density, percent fines, plasticity of the fines, degree of saturation, age of deposit, and earthquake ground motion. According to Caltrans[11] guidance, liquefaction potential is evaluated using the "simplified procedure" to a depth of 70 feet in the soil profile below the channel bottom. The Caltrans guidelines cite Boulanger and Idriss[12], which recommend considering a soil to have clay-like behavior (i.e., not susceptible to liquefaction) when the Plasticity Index (PI) is greater than or equal to 7. Predominately fine-grained (cohesive) soils such as clay and elastic silts would be considered subject to cyclic softening with a potential for reduction in shear strength rather than "classic" cyclically induced liquefaction associated with loose, saturated granular soils.

To evaluate the potential for soil liquefaction to occur at the project site, Crawford used the "simplified procedure" by Youd et al.[13] and guidelines/modifications consistent with Caltrans liquefaction evaluation procedures, SHN's boring data and laboratory test results, groundwater at elevation 3 feet (for bank borings) and at channel bottom (for channel borings), a site-to-fault distance of 9.2 miles, Maximum Moment Magnitude (M_{max}) of 8.63, and a PGA of 1.03g. Refer to Appendix C for liquefaction triggering analysis results.

Based on the foregoing, subsurface materials encountered throughout the upper 70 feet of the site's subsurface profile are susceptible to liquefaction. Table 7 summarizes the potentially critical liquefiable material zones (Factor of Safety < 1.0) identified based on preliminary analysis.

[11] Caltrans Geotechnical Manual, Liquefaction Evaluation, January 2020.
[12] Liquefaction Susceptibility Criteria for Silts and Clays, November 2006.
[13] Youd, T. L., et al, 2001, *"Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,"* Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001, pp. 817-833.

Table 7: Potentially Liquefiable Soil Zones/Layers

Boring No.	Potentially Liquefiable Soil Zones/Layers		Layer Thickness (feet)	Generalized Soil Description	Liquefaction Factor of Safety	Residual Soil Strength** (psf)
	Depth (feet)	Elevation (feet)				
B-1	15 to 27.5	3 to -9.5	12.5	SP-SM, GW	0.07 to 0.16	143 to 620
	45 to 70	-27 to -52	25	ML, SM, SP-SM	0.12 to 0.13	365 to 766
B-2	15 to 25	3 to -7	10	GW	0.10 to 0.13	328 to 331
	30 to 35	-12 to -17	5	GW	0.12	601
	40 to 70	-22 to -52	30	ML, SP-SM	0.09 to 0.20	213 to 2,026
B-3	0 to 33	0 to -33	33	ML, SM	0.08 to 0.10	157 to 293
	38 to 63	-38 to -63	24	SM, ML	0.09 to 0.20	235 to 905
B-4	0 to 18	2 to -16	18	GP	0.05	110
	53 to 63	-51 to -61	10	ML	0.22	1,044

11.4.3 SEISMICALLY INDUCED SETTLEMENT

The liquefaction analysis indicates seismically induced settlement of 8.3 to 12.6 inches within the saturated materials underlying the project site. Surface manifestation of the liquefaction effects is also indicated at this site. Bridge design will need to address the potential adverse effects associated with liquefaction, primarily the downdrag load induced on deep foundations due to negative skin friction, which is a significant consideration for final bridge foundation design.

Additionally, during a seismic event, ground shaking can cause densification of dry to moist, loose to medium dense granular soils above the water table, which can result in settlement of the ground surface. Based on the SHN boring data (B-1 and B-2) and Crawford's analysis, the magnitude of seismically induced settlement of the material above groundwater is estimated to be less than 0.5-inches.

11.4.4 SEISMIC SLOPE INSTABILITY

No indications of gross slope instability were observed at the site. The potential for seismic instability of the existing river banks is considered low and likely limited to minor (surficial) bank distortion. The potential for seismically induced slides on engineered fill slopes constructed at 1.5H:1V (horizontal to vertical), or flatter, with RSP per Caltrans Standard Section 72-2, or at 2H:1V, or flatter, with no RSP is considered low. Therefore, seismic instability of the existing banks and potential engineered fill slopes is considered low and not a design consideration.

11.4.5 LATERAL SPREADING POTENTIAL

Lateral spread, characterized by incremental flow-failure within liquefiable soil on sloping ground or a free face, can produce horizontal ground displacement during a seismic event. Youd et al.

(2002)[14] indicates that potentially liquefiable soil layers with SPT $(N_1)_{60}$ values less than 15 are susceptible to lateral spread. Based on the SHN boring data (B-1 and B-2), soil layers to a depth of about 60 feet have $(N_1)_{60}$ values less than 15. Therefore, there is potential for lateral spreading to occur at this site and is a geotechnical design consideration for foundation design.

Analysis of lateral spreading was not included in the scope of this report and will be addressed in the foundation report for final design. Tentatively, lateral spreading on order of 5 feet or more at the abutment locations is considered likely and will be a significant consideration for final bridge foundation design. For final design, lateral spreading will be analyzed consistent with procedures outlined in Caltrans Memo to Designers (MTD) 20-15 to evaluate the design displacement demand for deep foundations.

11.4.6 TSUNAMI INUNDATION

A tsunami is a series of ocean waves generated by sudden displacements in the sea floor, landslides, or volcanic activity. Tsunami inundation hazard mapping[15] by CGS shows the project site within a tsunami inundation hazard zone. The likely cause of tsunami in this area is via seismic activity along offshore faults (e.g. the Cascadia Megathrust located about 40 miles offshore). The bridge should be designed to withstand impact from a tsunami.

12 GEOTECHNICAL RECOMMENDATIONS

Based on the available subsurface data and regional mapping, foundation support for new bridge foundations is considered available within the underlying Unit 2 dense to very dense granular materials at depth. The site is in an area with several geologic hazards and key geotechnical engineering design elements considered significant to new bridge foundations for this project include:

- loose, saturated soils in the upper 60 to 75 feet of the site’s soil profile;
- shallow groundwater;
- potential corrosive soils environment and marine atmosphere;
- strong seismic design ground motions (PGA = 1.03g);
- potentially liquefiable soils in the upper 70 feet of the soil profile across the site (up to 12.6 inches of associated seismically induced ground settlement estimated);
- downdrag load on deep foundations due to seismic settlement;
- potential lateral spreading at the river banks/channel due to liquefaction;
- lateral loading on foundation elements due to lateral spreading;
- depth of scour; and
- tsunami inundation hazard area.

With the above considerations, deep foundations penetrating the dense to very dense granular materials (Unit 2) underlying the loose sediments (Unit 1) are recommended. Large diameter Cast-In-Drilled-Hole (CIDH) or driven Cast-in-Steel-Shell (CISS) piles are considered most appropriate for structure support. Also, it is understood that the Mad River is an environmentally sensitive area with respect to protected aquatic species. Therefore, CISS piles at intermediate supports within the channel would need to be driven within a dewatered cofferdam to reduce/mitigate detrimental vibration/hydroacoustic effects to sensitive/protected aquatic species.

[14] American Society of Civil Engineers (ASCE) Journal of Geotechnical and Geoenvironmental Engineering, December 2002.
[15] https://maps.conservation.ca.gov/cgs/informationwarehouse/ts_evacuation/

Driven concrete piles, closed-ended steel pipe piles, and steel H-piles are not considered appropriate for this site due to required long pile lengths, significant lateral/downdrag loads, and hard driving conditions within Unit 2.

Shallow, spread footing foundations are not recommend at this site due to weak bearing materials in the upper 60 to 75 feet and potential for significant seismically induced settlements.

Preliminary bridge foundation alternatives are discussed below. Refer to Appendix D for Preliminary Geotechnical Parameters, including L-Pile parameters for completing lateral analysis.

12.1 DEEP FOUNDATIONS

12.1.1 CAST-IN-DRILLED-HOLE (CIDH) PILES

The use of large diameter CIDH piles for new bridge foundations is considered feasible for this project and such piles are considered suitable for use at all support locations. The CIDH piles will be required to penetrate Unit 2 materials and can provide large vertical and lateral resistance. They will also help mitigate noise/vibration associated with driven piles. Due to the presence of shallow groundwater and surface water in the channel the CIDH piles will need to be installed by the “wet” method, including slurry drilling and concrete placed under slurry using tremie pipe. The “wet” method requires placement of inspection tubes to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) of the CIDH pile.

Tentatively, a pile diameter of 4 feet or greater is anticipated to be required to meet the CIDH pile constructability limit of 30 times the pile diameter due to the presence of potentially liquefiable soils that extend to depths on order of 60-75 feet below ground surface and anticipated pile length required to meet axial/lateral pile demand to accommodate downdrag and resist lateral spreading. For 5-foot diameter and larger Type-II shafts, permanent casing will be required to at least 5 feet below the construction joint to permit access for workers.

The use of temporary casing for ground control to the full depth of Unit 1 soils is expected to be required at all support locations due to the presence of saturated, loose gravel, sand, and silt which can be prone to caving. Casing extensions above the river water level will be required for constructing the piers located in the active channel. Construction of piers over water will also require a temporary work trestle or berm to support the drilling equipment/operation.

At the piers it is expected that installation of casing using an impact hammer will not be allowed due to environmental project constraints with respect to noise and vibration and that rotation/oscillation methods will need to be used instead. Casing should be equipped with suitable cutting teeth welded to the tip to help penetrate coarse alluvium (including cobbles/boulders).

For preliminary design, Mark Thomas requested Crawford evaluate a 72-inch diameter CIDH pile in axial compression for the proposed pier supports of Alternative 1. Based on the loading data provided by Mark Thomas (900 kips, 1200 kips, and 780 kips at the service limit, strength limit, and extreme limit, respectively), as well as the estimated additional loading induced by downdrag

(1,410 kips), the tip elevation for 72-inch CIDH piles is preliminarily estimated at -153 feet. Refer to Appendix E for an analysis summary.

The design tip elevation will ultimately vary depending on the actual pile diameter, defined axial/lateral loading requirements, design scour elevations, and pile cutoff elevation.

12.1.2 DRIVEN PILES

CAST-IN-STEEL-SHELL (CISS) PILES

As an alternative to CIDH piles, large diameter CISS piles can also be considered. The steel shell can be driven open-ended and then filled with concrete for additional lateral resistance. This type of pile can provide excellent structural resistance against horizontal loads and is a suitable option where poor soil conditions exist, sites where potential liquefaction or scour would cause long unsupported lengths and sites with anticipated large lateral soil displacements.

To utilize CISS piles for pier supports, the channel area would need to be dewatered, which would require construction of a cofferdam and seal coarse. Dewatering should allow for driving without harmful hydroacoustic effects to sensitive fish species.

At this site, hard driving conditions are expected below an elevation range of -62 to -72 feet, near the top of Unit 2, where blow counts typically were greater then 100 bpf. Due to the anticipated hard driving conditions within Unit 2, CISS plies would likely require center relief drilling to achieve specified tip elevation.

CONCRETE PILES, CLOSED-ENDED STEEL PIPE PILES, AND STEEL H-PILES

Other driven pile types such as Caltrans Standard concrete and closed-ended steel pipe piles are not considered appropriate at this site due to long expected lengths (i.e., over 100 feet) and hard driving conditions within Unit 2. Similarly, steel H-piles would likely require significant penetration into Unit 2 soils to achieve specified tip elevation.

12.2 SHALLOW FOUNDATIONS

Spread footing foundations are not recommended at this site due to the presence of relatively weak bearing materials and liquefiable soils within the upper 60 to 75 feet of the soil profile. Significant ground improvement measures to the bottom of Unit 1, such as soil-cement mixing, would be needed for spread footings to become a viable alternative at this site.

12.3 ADDITIONAL FIELDWORK AND LABORATORY TESTING

The following additional subsurface exploration and laboratory testing is recommended to be completed after type selection to support the final foundation design recommendations. The existing channel borings were drilled downstream of the existing bridge while the proposed supports of preferred Alterative 1 are located upstream of the existing bridge. Additionally, the sampling interval for the channel borings was wider than typically expected, so there are gaps in the existing data that additional boring data would help fill-in.

- One channel boring as close as possible to a proposed pier support in the southern portion of the river channel (Bent 2 of Alternative 1). This boring would extend approximate 200 feet below the channel bottom, with sampling completed every 5 feet in the upper 100 feet and every 10 feet below 100 feet. Sampling would include split-spoon drive samples (i.e., SPT and Modified California). This boring would be completed via a barge with tide gauge to correct for water surface elevation changes during drilling.

- A geophysical survey consisting of two Multichannel Analysis of Surface Wave (MASW) seismic profiles approximately 200 feet long, along the southern and northern side of the river channel to determine the average shear wave velocity within the upper 100 feet of the soil profile (lines would run through Abutment 1 and Bent 3 of Alternative 1). Preliminarily, it appears one line could be completed along the banks on the south side of the channel, and another could be completed along the gravel bar on the northside of the channel during low flow periods (i.e., summer to early fall). Property owner right-of-entry (ROE) and vegetation clearing would be needed for access to complete these tests.

The following laboratory tests would be completed on samples obtained from the additional boring; additional types of testing not listed may be needed depending on material encountered.

- ASTM D1140 – Amount of Material Finer than No. 200 Sieve in Soils by Washing
- ASTM D2216 – Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 – Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D6913 – Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis
- ASTM D7263 – Density and Unit Weight of Soil Specimens

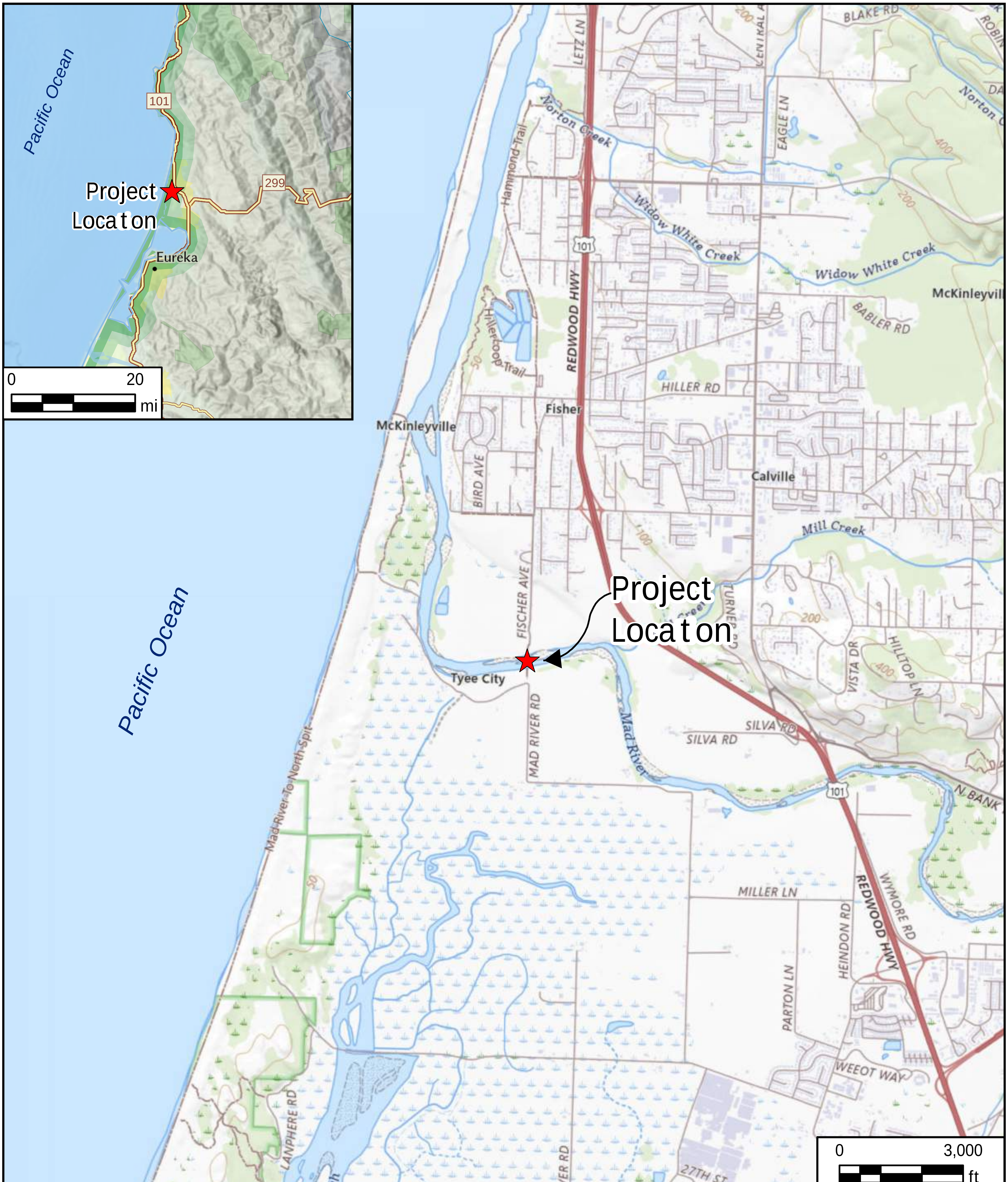
13 LIMITATIONS

Crawford performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. This report applies only to the Hammond Trail Pedestrian Bridge project. Do not use or rely on this report for different locations or improvements without the written consent of Crawford.

This report is preliminary and not to be used for final design. Crawford prepared this report for planning purposes and preliminary design only. Crawford will complete engineering analysis and a Foundation Report for final bridge design after type selection and foundation type/details are defined. The basis of geotechnical design and supporting documentation for final recommendations will be provided in the Foundation Report, including specific foundation and approach recommendations based on the design criteria developed for this project.

APPENDIX A

Figures



NORTH

References:

1. Base Map: USGS Topographic Map Layer, ArcGIS Pro, ESRI

2. Insert Base Map: National Geographic Style, ArcGIS Pro, ESRI

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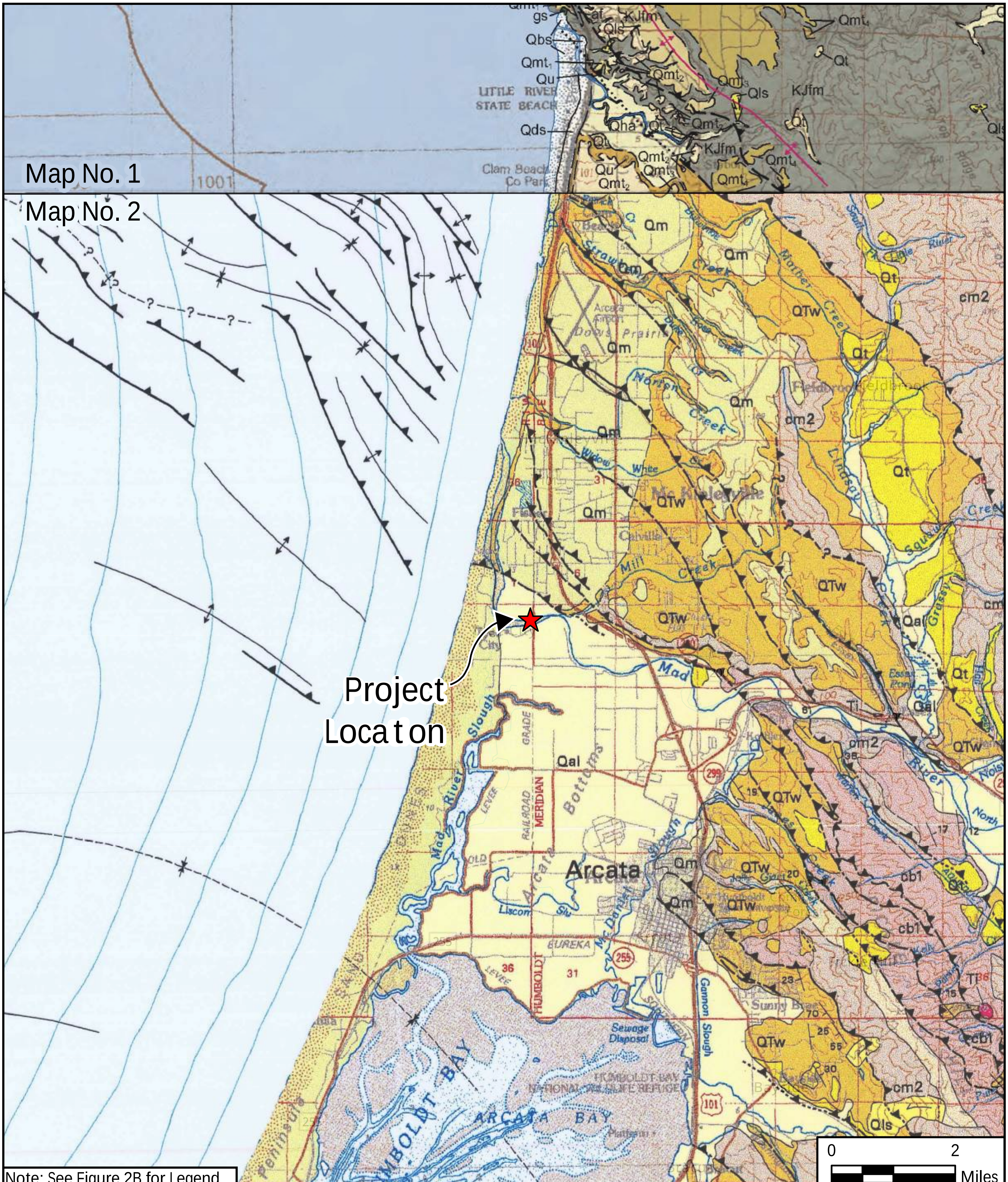
Geotechnical Engineering, Design and Construction Services

HAMMOND TRAIL
PEDESTRIAN BRIDGE

HUMBOLDT COUNTY, CA

Figure 1
Vicinity Map

Prj. No: 23-948.9
Date: 07/09/2024



NORTH

References:

1. Dela t re, Marc and Rosinski, Anne, 2012, Preliminary geologic map of onshore por t ons of the Crescent City and Orick 30x60 quadrangles, CA, CGS, PGM-12-05, 1:100,000.

2. McLaughlin et al., 2000, Geology of the Cape Mendocino, Eureka, Garberville, and southwestern part of the Hayfork 30x60 minute quadrangles and adjacent o f shore area, northern CA, with digital database, USGS, MF-2336, 1:100,000.

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HAMMOND TRAIL
PEDESTRIAN BRIDGE

HUMBOLDT COUNTY, CA

Figure 2A
Geologic Map

Prj. No: 23-948.9
Date: 07/09/2024

Map No. 1 Legend

af	Artificial fill (historical)
Qhc	Stream channel deposits (latest Holocene)
Qbs	Beach deposits (latest Holocene)
Qha	Young alluvial deposits, undifferentiated (Holocene)
Qhf	Young alluvial fan deposits (Holocene)
Qht	Young stream terrace deposits (Holocene)
Qds	Dune sand (Holocene)
Qe	Estuarine deposits (Holocene)
Qls	Landslide deposits (historical to Pleistocene)
Qa	Alluvial deposits, undifferentiated (Holocene to latest Pleistocene)
Qf	Alluvial fan deposits (Holocene to Pleistocene)
Qt	Stream terrace deposits (early Holocene to Pleistocene)
Qby	Battery Formation (late Pleistocene)
Qmt	Marine terrace deposits (Pleistocene)
	Trinidad marine terraces; names and approximate ages (ka=1000 years) from Woodward-Clyde Consultants (1980), and Carver (1992):
Qmt_1	Patricks Pt. terrace, age 64 ka
Qmt_2	Savage Creek terrace, age 83 ka, and McKinleyville terrace, age 96 ka
Qmt_3	Westhaven terrace, 103 ka
Qmt_4	Fox Farm terrace, 120 ka, and Sky Horse terrace, 130 ka
Qmt_5	A-Line terrace, 176 ka and older
Qmt_6	Maple Stump terrace, 200+ ka
Qsc	Terrace gravels of Surpur Creek (Pleistocene)
Qu	Undifferentiated marine and nonmarine overlap deposits (Pleistocene to late Pliocene?)
QTpc	Prairie Creek Formation (early Pleistocene to late Pliocene)
Twi	Wimer Formation (late Miocene)
Tsg	St. George Formation (late Miocene)

Franciscan Complex - Central Belt

KJfm	Mélange of the Central Belt (Late Cretaceous to Late Jurassic)
gs	Greenstone block within mélange

Map No. 2 Legend

Qal	**Alluvial deposits (Holocene and late Pleistocene?)**-Clay, silt, sand, gravel, and boulders, deposited in stream beds, alluvial fans, terraces, flood plains and ponds; and soils formed on these deposits. Includes largely Holocene deposits in modern stream channels and on flood plains
Qm	**Undeformed marine shoreline and aolian deposits (Holocene and late Pleistocene)**-Gravel and sand deposited in marine terraces, on benches, and on dunes along present shorelines. In northern Eureka quadrangle, near Arcata, includes older late Pleistocene dune sands (Carver and others, 1984)
Qt	**Undifferentiated nonmarine terrace deposits (Holocene and Pleistocene)**-Dissected and (or) uplifted gravel, sand, silt, and clay, deposited in fluvial settings. In western Eureka quadrangle (Sheet 1) unit includes minor shallow marine intertongues and warped and tilted beds of late Pleistocene Hookton and Rohnerville Formations of Ogle (1953), in addition to younger late Pleistocene and Holocene fluvial terrace units a few feet to a few tens of feet higher than normal modern high-water level
Qls	**Landslide deposits (Holocene and Pleistocene)**-Unsorted clay- to boulder-size debris and broken rock masses that have moved downslope in debris flows, earth flows, and as more-or-less intact rotational or translational blocks, largely from Pleistocene to present. Only large landslides, occupying tens to hundreds of acres, are depicted here.
QTog	**Older alluvium (Pleistocene and (or) Pliocene)**-Weakly consolidated to unconsolidated alluvial sand and gravel in patches as much as 700 feet higher than the present stream level; locally may be related to remnants of old upland surfaces such as preserved along the upper reaches of Plummer, Naufus, and Bear Wallow Creeks in the Pickett Peak area of the Garberville quadrangle (Sheet 3). Also includes alluvium in upland basins to the west of the Eastern belt of the Franciscan Complex, such as Hoaglin and Kettenpom Valleys in the Zenia and Lake Mountain 1:24,000 quadrangles (Sheet 3)
QTw	**Marine and nonmarine overlap deposits (late Pleistocene to middle Miocene)**-Thin-bedded to massive, weakly lithified siltstone, fine- to medium-grained sandstone, silty to diatomaceous mudstone and locally soft, scaly mudstone. Locally includes lenses of pebble to boulder conglomerate, carbonate concretions, abundant molluscan fossils, woody debris, and horizons of rhyolitic volcanic ash that are greater than 1 meter thick in some areas. Includes the Wildcat Group (Ogle, 1953), the Bear River beds (Haller, 1980), and related outlier Neogene deposits isolated along faults near Briceland, Garberville, Benbow, Piercy, Bridgeville and northeast of Weott. Unit also includes minor fault-bounded blocks along or near the coast between Bear River and the Mattole River that are incorporated into melange of the Coastal terrane; the Neogene Falor Formation northeast of Eureka (Manning and Ogle, 1950); and equivalent deposits in the offshore area deposited in shelf, slope, and slope basin settings. A few poorly exposed erosional remnants of shallow marine to brackish water strata mapped along high ridge crests overlying the Franciscan Complex in the 1:24,000 Zenia quadrangle are tentatively assigned to this unit. South of this map, unit correlates with valley-fill, perched gravel and shallow marine to nonmarine coal-bearing sedimentary rocks of Quaternary and Tertiary age in the Round Valley area of Covelo 1:100,000 quadrangle (Jayko and others, 1989)
cm1	**Melange**-Predominantly penetratively sheared, locally tuffaceous, scaly meta-argillite and less abundant blocks of metasandstone. Exhibits rounded, poorly incised, lumpy and irregular topography
cm2	**Melange**-Subequal amounts of metasandstone and meta-argillite. Exhibits irregular topography that lacks well incised sidehill drainages, but is less lumpy than unit cm1

References:

1. Dela t re, Marc and Rosinski, Anne, 2012, Preliminary geologic map of onshore por t ons of the Crescent City and Orick 30x60 quadrangles, CA, CGS, PGM-12-05, 1:100,000.

2. McLaughlin et al., 2000, Geology of the Cape Mendocino, Eureka, Garberville, and southwestern part of the Hayfork 30x60 minute quadrangles and adjacent o f shore area, northern CA, with digital database, USGS, MF-2336, 1:100,000.

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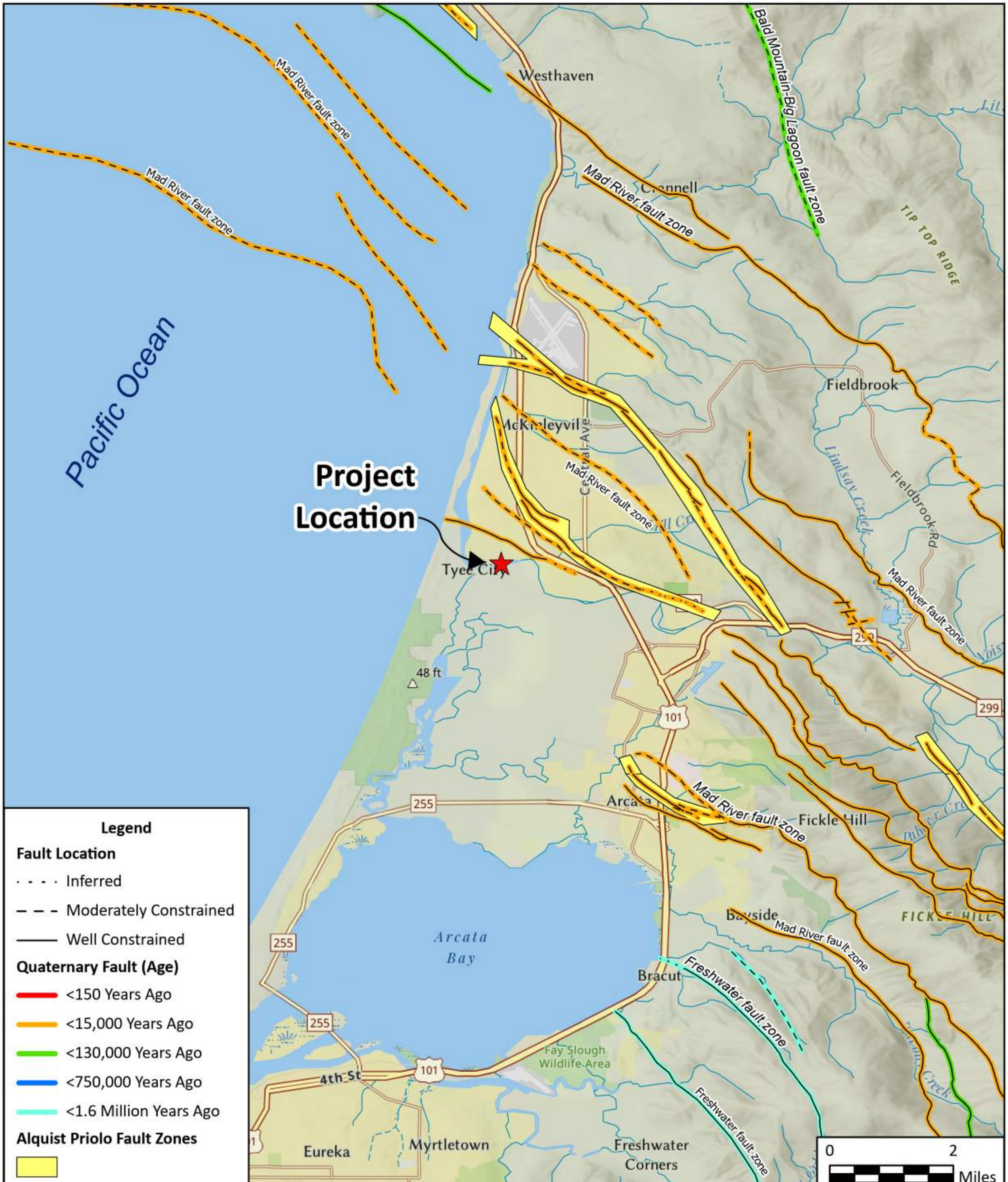
HAMMOND TRAIL PEDESTRIAN BRIDGE

HUMBOLDT COUNTY, CA

Figure 2B
Geologic Map Legend

Prj. No: 23-948.9
Date: 07/09/2024



NORTH

References:

1. Fault Data: USGS GIS Data
2. Base Map: National Geographic Style, ArcGIS Pro, ESRI
3. Alquist-Priolo Earthquake Fault Zones, CGS

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HAMMOND TRAIL PEDESTRIAN BRIDGE

HUMBOLDT COUNTY, CA

Figure 3
Fault Map

Prj. No: 23-948.9
Date: 07/09/2024

SEISMIC DESIGN DATA

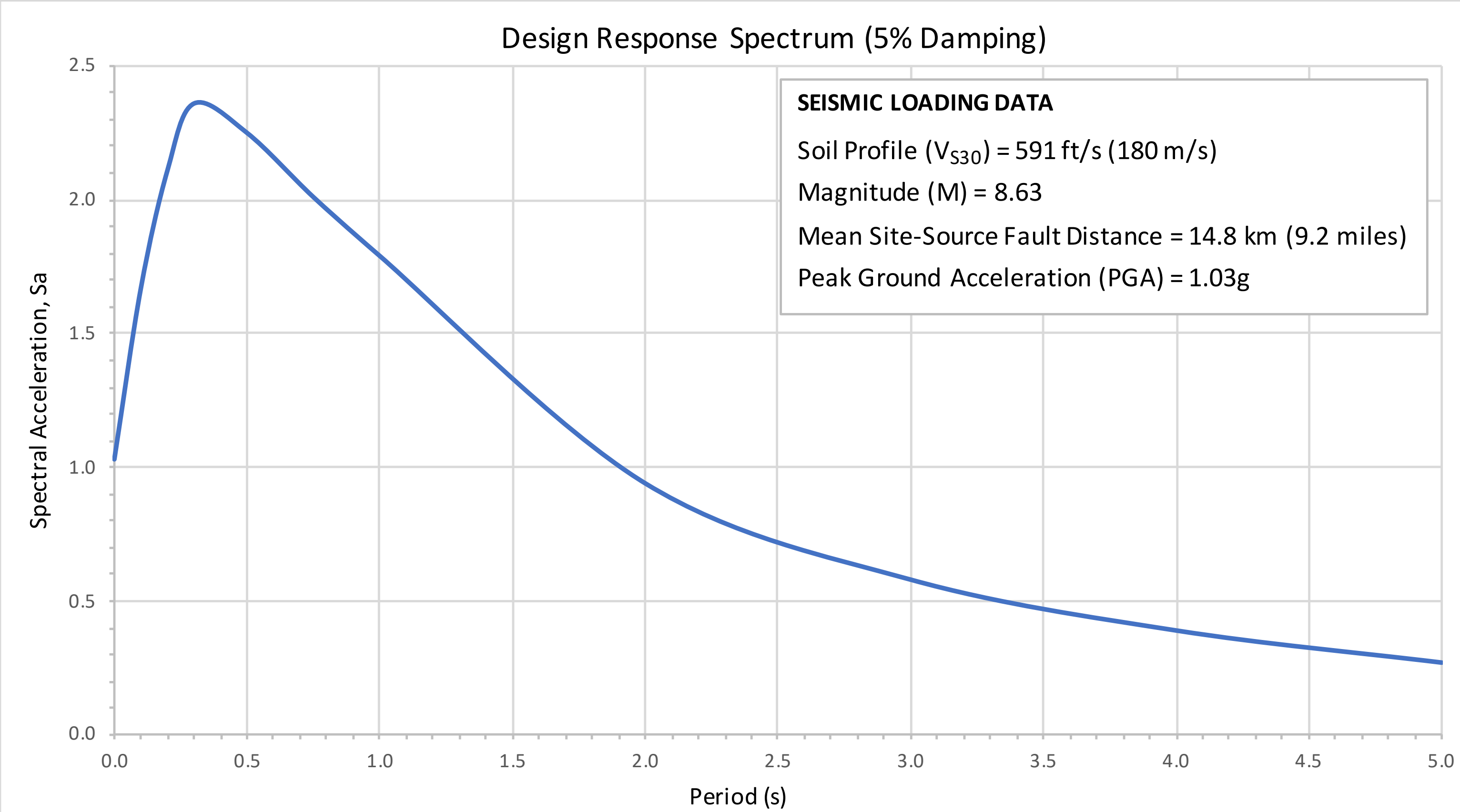
Hammond Trail Pedestrian Brige

McKinleyville, Humboldt County, California

Crawford Project Number: 23-948.9
Caltrans ARS Online Version: V3.1.0
Date Accessed: 4/19/2024

Period (s)	Spectral Acceleration, Sa (g)
0.00	1.03
0.10	1.67
0.20	2.11
0.30	2.36
0.50	2.25
0.75	2.01
1.00	1.79
2.00	0.94
3.00	0.58
4.00	0.39
5.00	0.27

Note: Seismic Loading Data provided consistent with Attachment 1 of Caltrans Memo to Designers 1-47



The Design Response Spectrum is based on the probabilistic response spectrum obtained for a 975-year return period (5% probability of exceedance in 50 years) from the USGS 2014 hazard data (v4.2.0) with adjustment factors required by the Caltrans Seismic Design Criteria (SDC) V2.0.

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Sacramento
Eureka
Modesto
Pleasanton
Santa Rosa
Seattle
Ukiah

Site Latitude: 40.9241°
Site Longitude: -124.12040°

FIGURE 4

APPENDIX B

Existing Subsurface Data (SHN, 2014-2015)

SHN CONSULTING ENGINEERS & GEOLOGISTS

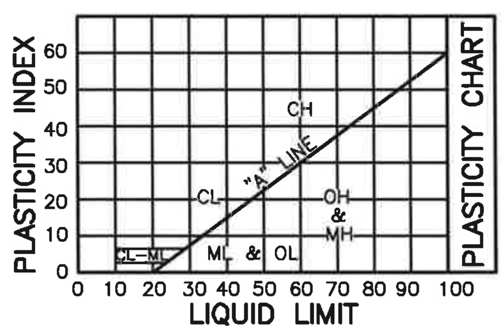
METHOD OF SOIL CLASSIFICATION

CLASSIFICATION CHART

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN 1/2 OF SOIL >NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN 1/2 OF COARSE FRACTION > NO.4 SIEVE SIZE)	GW	WELL GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (MORE THAN 1/2 OF COARSE FRACTION < NO.4 SIEVE SIZE)	SW	WELL GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SM	SILTY SANDS, SAND-SILT MIXTURES
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN 1/2 OF SOIL <NO. 200 SIEVE SIZE)	SILTS & CLAYS LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS & CLAYS LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS, ORGANIC SILTS
HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZE CHART

CLASSIFICATION	U.S. STANDARD SIEVE SIZE
BOULDERS	ABOVE 12"
COBBLES	12" TO 3"
GRAVEL	3" TO NO. 4
COARSE	3" TO 3/4"
FINE	3/4" TO NO. 4
SAND	NO. 4 TO NO. 200
COARSE	NO. 4 TO NO. 10
MEDIUM	NO. 10 TO NO. 40
FINE	NO. 40 TO NO. 200
SILT & CLAY	BELOW NO. 200



CONSISTENCY OF FINE GRAINED SOILS		DENSITY OF COARSE GRAINED SOILS	
CLASSIFICATION	COHESION (PSF)	CLASSIFICATION	STANDARD PENETRATION (BLOW COUNT)
VERY SOFT	0-250	VERY LOOSE	0-4
SOFT	250-500	LOOSE	4-10
MEDIUM STIFF	500-1000	MEDIUM	10-30
STIFF	1000-2000	DENSE	30-50
VERY STIFF	2000-4000	VERY DENSE	50+
HARD	4000+		

MOISTURE CLASSIFICATIONS
DRY
DAMP
MOIST
WET

BASED ON UNIFIED SOILS CLASSIFICATION SYSTEM

CONSULTING ENGINEERS
& GEOLOGISTS

BORING LOG KEY

SAMPLE TYPES

DISTURBED SAMPLE (BULK)

HAND DRIVEN TUBE SAMPLE

1.4" I.D. STANDARD PENETRATION TEST SAMPLE (SPT)

2.5" I.D. MODIFIED CALIFORNIA SAMPLE (SOLID WHERE RETAINED)

CORE BARREL SAMPLE (NOT RETAINED)

CORE BARREL SAMPLE (RETAINED)

SYMBOLS

INITIAL WATER LEVEL

STABILIZED WATER LEVEL

GRADATIONAL CONTACT

WELL DEFINED CONTACT

SS SPLIT SPOON

SHN Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge
LOCATION: South bridge approach
GROUND SURFACE ELEVATION: 18 Feet NAVD 88
EXCAVATION METHOD: Mud Rotary Wash
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 7/1/14
TOTAL DEPTH OF BORING: 101.5 Feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER ~15 feet

BORING NUMBER B-1

ELEVATION (FT)	DEPTH (FT)	BLOWS PER 0.5'	USCS	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
18.0	0.0		ML	SILT WITH SAND, dark grayish brown (2.5Y 4/2), soft, moist, fine sand, non-plastic, low dry strength.							Drilled to 7.5 feet with solid flight auger; hole caving; drove 5 feet of conductor casing and switched to mud rotary.
17.0	1.0										
16.0	2.0										
15.0	3.0										
14.0	4.0										
13.0	5.0	5, 6, 8	SP	POORLY GRADED SAND, dark gray (10YR 4/1), loose , dry, subangular to subrounded medium to coarse sand, few fine subangular gravels composed of chert and graywacke, non-plastic, non-cohesive.							
12.0	6.0										
11.0	7.0										
10.0	8.0										
9.0	9.0										
8.0	10.0	5, 5, 4		Grades coarser.							
7.0	11.0										
6.0	12.0										
5.0	13.0										
4.0	14.0										
3.0	15.0 ▽	5, 4, 4	SP/SM	POORLY GRADED SAND WITH SILT, dark bluish gray (Gley 2 4/5PB) , loose, wet, non-plastic, non-cohesive, about 10% fines, fine sand, abundant subrounded quartz grains.	30	90					
2.0	16.0										
1.0	17.0										
0.0	18.0										
-1.0	19.0										
-2.0	20.0	7, 5, 6	GW	WELL-GRADED GRAVEL WITH SAND, medium dense, wet, subangular to subrounded fine to coarse gravel, coarse sand, non-plastic, non-cohesive.				5			
-3.0	21.0										
-4.0	22.0										
-5.0	23.0										
-6.0	24.0										
-7.0	25.0	11, 9, 5									No recovery.
-8.0	26.0										
-9.0	27.0										
-10.0	28.0	4, 6, 7	ML	SILT, dark gray (Gley 1 4/N), stiff, wet, low plasticity, medium toughness, cohesive; organic layer at 27.5-27.8 feet, strong odor of decayed organics, sharp basal contact.	31	88	1376				Used catcher.
-11.0	29.0										
-12.0	30.0	3, 3, 3									Used catcher. No recovery.
-13.0	31.0										
-14.0	32.0			Becomes medium stiff.							
-15.0	33.0										
-16.0	34.0										
-17.0	35.0	4, 8, 11		Becomes stiff, medium plastic.	37	84	1230				Used catcher. Pocket Pen: 1.75 tsf
-18.0	36.0										
-19.0	37.0										

LOG OF BORING

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SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER ~15 feet

BORING NUMBER B-1

ELEVATION (FT)	DEPTH (FT)	BLOWS PER 0.5'	USCS	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Liquid Limit	Plastic Index	REMARKS
-20.0	38.0										
-21.0	39.0										
-22.0	40.0	3 4 4		Becomes medium stiff to stiff, interbedded lenses of faintly visible organic horizons.					32	9	Used catcher.
-23.0	41.0										
-24.0	42.0										
-25.0	43.0										
-26.0	44.0										
-27.0	45.0	4 9 13		SANDY SILT, dark gray, stiff, wet, fine sand, low plasticity.	23	102		68			Used catcher.
-28.0	46.0										
-29.0	47.0										
-30.0	48.0										
-31.0	49.0										
-32.0	50.0	4 5 7	SM	SILTY SAND, dark gray, medium dense, wet, fine sand, non-plastic, non-cohesive, trace of shell fragments.							Used catcher.
-33.0	51.0										
-34.0	52.0										
-35.0	53.0										
-36.0	54.0										
-37.0	55.0										
-38.0	56.0										
-39.0	57.0										
-40.0	58.0										
-41.0	59.0										
-42.0	60.0	13 18 21	SP/SM	POORLY GRADED SAND WITH SILT, very dark gray, medium dense, wet, fine sand with trace of medium sand composed of subrounded quartz and greenstone, non-plastic, non-cohesive, contains large woody debris.	18	108					Used catcher. Consolidatied Undrained Triaxial Shear Test. (See Attachment 2 for test results).
-43.0	61.0										
-44.0	62.0										
-45.0	63.0										
-46.0	64.0										
-47.0	65.0										
-48.0	66.0										
-49.0	67.0										
-50.0	68.0										
-51.0	69.0										
-52.0	70.0	4 5 7	OL	ORGANIC SOIL, dark grayish brown, stiff, wet, medium plasticity, medium toughness, weathered sandstone grains in silt matrix. contains large woody debris at top of sample, grades sandy above wood.							Used catcher.
-53.0	71.0										
-54.0	72.0										
-55.0	73.0										
-56.0	74.0										
-57.0	75.0										

LOG OF BORING

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LOCATION: South bridge approach
GROUND SURFACE ELEVATION: 18 Feet NAVD 88
EXCAVATION METHOD: Mud Rotary Wash
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 7/1/14
TOTAL DEPTH OF BORING: 101.5 Feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER ~15 feet

BORING NUMBER
B-1

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-57.0	75.0		8 16 32	SC		CLAYEY SAND WITH GRAVEL, gray (2.5Y 5/1), dense, wet, fine to medium micaceous sand, subangular fine gravel, medium plasticity matrix, cohesive; top of FALOR FORMATION.	20	108					Used catcher. Consolidatied Undrained Triaxial Shear Test. (See Attachment 2 for test results).
-58.0	76.0												
-59.0	77.0												
-60.0	78.0												
-61.0	79.0												
-62.0	80.0		13 12 21	SW		WELL-GRADED SAND WITH SILT AND GRAVEL, gray, dense, wet, fine to coarse sand, few subangular to subrounded fine gravels, non-plastic, non-cohesive.							
-63.0	81.0												
-64.0	82.0												
-65.0	83.0												Used catcher. Drill rig chattering from 79 to 90 feet.
-66.0	84.0												
-67.0	85.0												
-68.0	86.0												
-69.0	87.0												
-70.0	88.0												
-71.0	89.0												
-72.0	90.0		43 50/3"	SW		WELL-GRADED SAND WITH GRAVEL, olive brown (2.5Y 4/4), very dense, wet, medium to coarse sand, subrounded fine to coarse gravel, non-plastic, non-cohesive, slightly clayey matrix; cobble-sized clast fractured with hammer blow lodged in cutting shoe.							Used catcher. Sampler penetration refusal at 90.8 feet.
-73.0	91.0												
-74.0	92.0												
-75.0	93.0												
-76.0	94.0												
-77.0	95.0												
-78.0	96.0												
-79.0	97.0												
-80.0	98.0					POORLY GRADED SAND WITH CLAY, dark yellowish brown (10YR 4/6), very dense, wet, medium to coarse sand, few subrounded fine gravels, low plasticity, cohesive.							
-81.0	99.0												
-82.0	100.0		23 25 28	SP/SC									Used catcher.
-83.0	101.0												
-84.0	102.0					Borehole completed to 101.5 feet; backfilled with neat cement.							
-85.0	103.0												
-86.0	104.0												
-87.0	105.0												
-88.0	106.0												
-89.0	107.0												
-90.0	108.0												
-91.0	109.0												
-92.0	110.0												

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge	**JOB NUMBER:** 014099.100
LOCATION: North bridge approach	**DATE DRILLED:** 7/2/14
GROUND SURFACE ELEVATION: 18 Feet NAVD 88	**TOTAL DEPTH OF BORING:** 101.5 Feet
EXCAVATION METHOD: Mud Rotary Wash	**SAMPLER TYPE:** MCS/SPT
LOGGED BY: G. Vadurro	**DEPTH TO GROUNDWATER** ~ 15 feet

BORING NUMBER B-2

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
18.0	0.0			Fill		GRAVEL FILL.							Drilled to 5 feet with solid flight auger; hole caving; drove 5 feet of conductor casing and switched to mud rotary.
17.0	1.0												
16.0	2.0												
15.0	3.0												
14.0	4.0												
13.0	5.0		2 20 17	SM		GRAVEL FILL to SILTY SAND, dark grayish brown, dense, dry.							
12.0	6.0												
11.0	7.0												
10.0	8.0												
9.0	9.0												
8.0	10.0		5 7 9	SW		WELL GRADED SAND, medium dense, moist, medium to coarse sand, trace of fine sand, few fine rounded gravels, non-plastic, non-cohesive.							Used catcher.
7.0	11.0						18	105		5			
6.0	12.0												
5.0	13.0												
4.0	14.0												
3.0	15.0		5 4 4	GW		WELL GRADED GRAVEL, loose, wet, subrounded fine to coarse gravel, non-plastic, non-cohesive.				3			Used catcher.
2.0	16.0												
1.0	17.0												
0.0	18.0												
-1.0	19.0												
-2.0	20.0		5 9 12			Becomes medium dense.							Used catcher.
-3.0	21.0						11	113					
-4.0	22.0												
-5.0	23.0												
-6.0	24.0												
-7.0	25.0		16 23 16			Becomes dense, gravels coarsen with depth, fractured cobble from hammer blow in cutting shoe.							Used catcher.
-8.0	26.0												
-9.0	27.0												
-10.0	28.0												
-11.0	29.0												
-12.0	30.0		19 16 16			Becomes fine subangular to subrounded gravel with medium to coarse sand.							Used catcher.
-13.0	31.0						9	121					
-14.0	32.0												
-15.0	33.0												
-16.0	34.0												
-17.0	35.0		3 3 5	CL/ML		LEAN CLAY, dark gray, medium stiff, wet, high plasticity, cohesive.							Used catcher.
-18.0	36.0										48	22	Pocket Pen:1.75 tsf
-19.0	37.0												

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge
LOCATION: North bridge approach
GROUND SURFACE ELEVATION: 18 Feet NAVD 88
EXCAVATION METHOD: Mud Rotary Wash
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 7/2/14
TOTAL DEPTH OF BORING: 101.5 Feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER ~ 15 feet

BORING NUMBER B-2

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-20.0	38.0												
-21.0	39.0												
-22.0	40.0		4	ML									Used catcher.
-23.0	41.0		5 5			SILT, dark gray (2.5Y 4/1), medium stiff, moist, low plasticity, low toughness, cohesive, trace of organics.	34	92	1298				
-24.0	42.0												
-25.0	43.0												
-26.0	44.0												
-27.0	45.0		2										
-28.0	46.0		2 3			Becomes, medium plastic, slightly clayey.							
-29.0	47.0												
-30.0	48.0												
-31.0	49.0												
-32.0	50.0		5	SM/ML									Used catcher. Consolidatied Undrained Triaxial Shear Test. (See Attachment 2 for test results).
-33.0	51.0		8 6			SILTY SAND to SANDY SILT (alternates in top and bottom of liner), dark gray, loose to stiff, wet, fine sand, non-plastic, slightly cohesive.	25	98					
-34.0	52.0												
-35.0	53.0												
-36.0	54.0												
-37.0	55.0												
-38.0	56.0												
-39.0	57.0												
-40.0	58.0												
-41.0	59.0												
-42.0	60.0		9	SP/SM									
-43.0	61.0		10 12			POORLY GRADED SAND WITH SILT, very dark gray (Gley 1 3/N), medium dense, wet, fine sand, non-plastic, non-cohesive.							
-44.0	62.0												
-45.0	63.0												
-46.0	64.0												
-47.0	65.0												
-48.0	66.0												
-49.0	67.0												
-50.0	68.0												
-51.0	69.0												
-52.0	70.0		14			POORLY GRADED SAND WITH SILT at 70-71' with sharp basal contact.							
-53.0	71.0		22 36	SM		SILTY SAND, olive gray (5Y 5/2), very dense, wet, fine sand, >30% fines, non-plastic, consolidated; top of FALOR FORMATION.	15	112					Used catcher. Consolidatied Undrained Triaxial Shear Test. (See Attachment 2 for test results). Drill rig chattering beginning at 73 feet.
-54.0	72.0												
-55.0	73.0												
-56.0	74.0												
-57.0	75.0												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge
LOCATION: North bridge approach
GROUND SURFACE ELEVATION: 18 Feet NAVD 88
EXCAVATION METHOD: Mud Rotary Wash
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 7/2/14
TOTAL DEPTH OF BORING: 101.5 Feet
SAMPLER TYPE: MCS/SPT
DEPTH TO GROUNDWATER ~ 15 feet

BORING NUMBER B-2

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-57.0	75.0												
-58.0	76.0												
-59.0	77.0												
-60.0	78.0												
-61.0	79.0												
-62.0	80.0		22 46 35	SW		WELL-GRADED SAND WITH GRAVEL; very dense, olive brown (2.5Y 4/4), moist, fine to coarse sand, subangular to subrounded fine gravel, some gravel deeply weathered in-place, non-plastic, consolidated,							
-63.0	81.0												
-64.0	82.0												
-65.0	83.0												
-66.0	84.0												
-67.0	85.0												
-68.0	86.0												
-69.0	87.0												
-70.0	88.0												
-71.0	89.0												
-72.0	90.0		35 50/3"			Gravels grade coarser, clayey matrix, fractured chert cobble from hammer blow in cutting shoe.							
-73.0	91.0												
-74.0	92.0												
-75.0	93.0												
-76.0	94.0												
-77.0	95.0												Drill rig chattering from 95 to 100 feet.
-78.0	96.0												
-79.0	97.0												
-80.0	98.0												
-81.0	99.0												
-82.0	100.0		11 26 50/5"	GW		No recovery; fractured cobble from hammer blow in cutting shoe.							
-83.0	101.0												
-84.0	102.0					Borehole completed to 101.5 feet; backfilled with neat cement.							
-85.0	103.0												
-86.0	104.0												
-87.0	105.0												
-88.0	106.0												
-89.0	107.0												
-90.0	108.0												
-91.0	109.0												
-92.0	110.0												

LOG OF BORING

Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER B-3

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
7.0	0.0												Depths relative to barge deck.
6.0	1.0					Water surface.							
5.0	2.0												
4.0	3.0												
3.0	4.0												
2.0	5.0												Drove 20' of conductor casing to ~13' below channel bottom. Drilled to 20 feet (below barge deck).
1.0	6.0					Channel bottom.							
0.0	7.0			GW									
-1.0	8.0					GRAVEL, graywacke sandstone and quartz-rich gravels.							
-2.0	9.0												
-3.0	10.0												Soil description at 7-20' from drill cuttings.
-4.0	11.0												
-5.0	12.0												
-6.0	13.0												
-7.0	14.0												
-8.0	15.0												
-9.0	16.0												
-10.0	17.0												
-11.0	18.0					Silt beginning at ~18 feet.							
-12.0	19.0												
-13.0	20.0		3 1 3	ML		SILT, dark gray (2.5Y 4/1), soft, wet, medium plasticity, low toughness, trace organics.				97			Switched to mud rotary wash.
-14.0	21.0												
-15.0	22.0												
-16.0	23.0												
-17.0	24.0												
-18.0	25.0												
-19.0	26.0												
-20.0	27.0												
-21.0	28.0												Silt in cuttings to 30'.
-22.0	29.0												
-23.0	30.0		5 6 6			Becomes medium stiff.							No recovery.
-24.0	31.0												
-25.0	32.0												
-26.0	33.0												
-27.0	34.0												Silt in cuttings to 35'.
-28.0	35.0		6 8 10	SM									
-29.0	36.0					SILTY SAND, dark gray, medium dense, wet, low plasticity, fine sand, 30-40% silt, coarsens downward.			579				
-30.0	37.0												

LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER B-3

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-31.0	38.0												
-32.0	39.0												
-33.0	40.0		10, 8, 7	SM/ML		SILTY SAND and SILT WITH SAND, alternating lenses, medium dense/stiff, wet, non-plastic, fine sand with trace medium sand.				44			Used catcher.
-34.0	41.0												
-35.0	42.0												
-36.0	43.0												
-37.0	44.0												
-38.0	45.0		9, 14			Becomes medium dense/very stiff.							No recovery. Auto hammer anvil sheared off; done for day.
-39.0	46.0												
-40.0	47.0												
-41.0	48.0												
-42.0	49.0												
-43.0	50.0			ML		SILT, very dark greenish gray (Gley 2 3/5 BG), stiff, wet, low plasticity, cohesive, trace fine sand.			1245				Drilled to 50 feet. Piston core sample. Pushed 24"; 20" recovery.
-44.0	51.0												
-45.0	52.0												
-46.0	53.0												
-47.0	54.0												
-48.0	55.0		6, 7, 7	SM/ML		SILTY SAND and SILT WITH SAND, alternating lenses, dark gray, medium dense/stiff, wet, low plasticity, slightly cohesive, fine sand, coarsens downward.				92			Used catcher.
-49.0	56.0												
-50.0	57.0												
-51.0	58.0												
-52.0	59.0												
-53.0	60.0		3, 4, 4	ML		SILT, very dark gray (2.5Y 3/1), medium siff, wet, medium plasticity, cohesive, trace wood fragments, trace fine sand.				62			Used catcher.
-54.0	61.0												
-55.0	62.0												
-56.0	63.0												
-57.0	64.0												
-58.0	65.0												
-59.0	66.0												
-60.0	67.0												
-61.0	68.0												Increased drill resistance at 69 feet.
-62.0	69.0												
-63.0	70.0		12, 16, 20	SP/SM		POORLY GRADED SAND WITH SILT, light olive brown (2.5Y 5/4), dense, wet, non-plastic, weak cementation, fine to medium sand, few fine subrounded gravels, mostly chert; top of FALOR FORMATION.							Used catcher. 50% recovery.
-64.0	71.0												
-65.0	72.0												
-66.0	73.0												
-67.0	74.0												
-68.0	75.0												

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER B-3

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-68.0	75.0		35										
-69.0	76.0		46 51	SP		POORLY GRADED SAND WITH GRAVEL, dark yellowish brown (10YR 4/4), very dense, wet, non-plastic, weak cementation, medium to coarse subrounded sand, ~40% fine to coarse chert and sandstone gravel, trace silt.	9.8	132					100% recovery. Consolidatied Undrained Triaxial Shear Test. (See Attachment 2 for test results).
-70.0	77.0												
-71.0	78.0												
-72.0	79.0												
-73.0	80.0		19	SM									40% recovery.
-74.0	81.0		39 43			SILTY SAND WITH GRAVEL, dark yellowish brown, very dense, moist, strong cementation, oxidized fracture surfaces, fine to medium sand, fine to coarse subrounded gravel.							
-75.0	82.0												
-76.0	83.0												
-77.0	84.0												
-78.0	85.0												
-79.0	86.0												
-80.0	87.0												
-81.0	88.0												
-82.0	89.0												
-83.0	90.0		38 52/3"	GW		GRAVEL, very dense, fractured cobble in cutting shoe, recovered few fine to coarse gravels and cobbles.							Poor recovery.
-84.0	91.0												
-85.0	92.0												
-86.0	93.0												
-87.0	94.0												
-88.0	95.0												
-89.0	96.0												
-90.0	97.0												
-91.0	98.0												Drill rig chattering to 100'.
-92.0	99.0												
-93.0	100.0		48	SM									80% recovery.
-94.0	101.0		50 50/5"			SILTY SAND WITH GRAVEL, yellowish brown (10YR 5/6), very dense, wet, low plasticity , slightly cohesive, weak cementation, fine to medium sand, fine rounded gravel.							
-95.0	102.0												
-96.0	103.0												
-97.0	104.0												
-98.0	105.0												
-99.0	106.0												
-100.0	107.0												
-101.0	108.0												
-102.0	109.0												
-103.0	110.0		50/3"	GP/GM		POORLY GRADED GRAVEL WITH SILT AND SAND, very dense, wet, medium cementation, fine subrounded gravel.							
-104.0	111.0												
-105.0	112.0												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER B-3

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-106.0	113.0												
-107.0	114.0												
-108.0	115.0												
-109.0	116.0												
-110.0	117.0												
-111.0	118.0												
-112.0	119.0												
-113.0	120.0		104	SM		SILTY SAND WITH GRAVEL, very dense, wet, fine to coarse gravel and cobbles.							30% recovery.
-114.0	121.0												
-115.0	122.0												
-116.0	123.0												
-117.0	124.0												
-118.0	125.0												
-119.0	126.0												
-120.0	127.0												
-121.0	128.0												
-122.0	129.0												
-123.0	130.0		49 50/4"	SP		POORLY GRADED SAND WITH GRAVEL, strong brown, very dense, wet, strong cementation, fine to medium sand, fine subrounded gravel.							50% recovery.
-124.0	131.0												
-125.0	132.0												
-126.0	133.0												
-127.0	134.0												
-128.0	135.0												
-129.0	136.0												
-130.0	137.0												
-131.0	138.0												
-132.0	139.0												
-133.0	140.0		52 50/3.5"	GP		POORLY GRADED GRAVEL, very dense, fractured cobble in cutting shoe, fine to coarse cobbles, fractured from hammer blow, subrounded where intact.							Used catcher. 30% recovery.
-134.0	141.0												
-135.0	142.0												
-136.0	143.0												
-137.0	144.0												
-138.0	145.0												
-139.0	146.0												
-140.0	147.0												
-141.0	148.0												
-142.0	149.0												
-143.0	150.0												

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.

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PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER B-3

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-143.0	150.0												
-144.0	151.0												
-145.0	152.0												
-146.0	153.0												
-147.0	154.0												
-148.0	155.0			SP									Out of gravels at 155'.
-149.0	156.0												
-150.0	157.0												
-151.0	158.0												
-152.0	159.0												
-153.0	160.0		31 37 38										Used catcher. 60% recovery.
-154.0	161.0					POORLY GRADED SAND, very dark bluish gray (Gley 2 3/10B), very dense, wet, non-plastic, weak cementation, fine sand with some medium sand, <10% fines.							
-155.0	162.0												
-156.0	163.0												
-157.0	164.0												
-158.0	165.0												Gravel in cuttings at 165'.
-159.0	166.0												
-160.0	167.0												
-161.0	168.0												
-162.0	169.0												
-163.0	170.0												
-164.0	171.0												
-165.0	172.0												
-166.0	173.0												
-167.0	174.0												
-168.0	175.0												Drill rig chattering at 175'.
-169.0	176.0												
-170.0	177.0												
-171.0	178.0												
-172.0	179.0												
-173.0	180.0		80	SM		SILTY SAND WITH GRAVEL, yellowish red (5YR 4/6), very dense, moist, non-plastic, medium cementation, medium to coarse sand, rounded fine gravel.							Used catcher.
-174.0	181.0												
-175.0	182.0												
-176.0	183.0												
-177.0	184.0												
-178.0	185.0												
-179.0	186.0												
-180.0	187.0												

SHN Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: North edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 0 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/19/15-1/20/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: ~7 feet At High Tide (11:30)

BORING NUMBER **B-3**

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-181.0	188.0												
-182.0	189.0												
-183.0	190.0												
-184.0	191.0												
-185.0	192.0												
-186.0	193.0												
-187.0	194.0												
-188.0	195.0												
-189.0	196.0												
-190.0	197.0												
-191.0	198.0												
-192.0	199.0												
-193.0	200.0		106	SP/GP		GRAVEL WITH SAND, fractured cobbles in shoe, coarse sand.							Used catcher.
-194.0	201.0												
-195.0	202.0					Borehole completed to 201 feet below barge deck (194 feet below channel bottom); backfilled with neat cement.							
-196.0	203.0												
-197.0	204.0												
-198.0	205.0												
-199.0	206.0												
-200.0	207.0												
-201.0	208.0												
-202.0	209.0												
-203.0	210.0												
-204.0	211.0												
-205.0	212.0												
-206.0	213.0												
-207.0	214.0												
-208.0	215.0												
-209.0	216.0												
-210.0	217.0												
-211.0	218.0												
-212.0	219.0												
-213.0	220.0												
-214.0	221.0												
-215.0	222.0												
-216.0	223.0												
-217.0	224.0												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: South edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 2 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/21/15-1/22/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

BORING NUMBER B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
9.0	0.0				Water							Depths relative to barge deck.
8.0	1.0											
7.0	2.0											
6.0	3.0											Drilled pilot hole with tri-cone bit to 13 feet below channel bottom; drove 20' of conductor casing to ~13' below channel bottom; difficult driving due to gravels and large woody debris (encountered log). Woody material in drill cuttings.
5.0	4.0											
4.0	5.0											
3.0	6.0				Channel bottom.							
2.0	7.0			GP	GRAVEL, fine subrounded sandstone and quartz-rich gravels, and woody debris in drill cuttings.							
1.0	8.0											
0.0	9.0											
-1.0	10.0											
-2.0	11.0											
-3.0	12.0											
-4.0	13.0											
-5.0	14.0											
-6.0	15.0											
-7.0	16.0											
-8.0	17.0											
-9.0	18.0											
-10.0	19.0											
-11.0	20.0		10 5 4	ML/ CL	No recovery: cohesive clay/silt on cutting shoe.							No recovery. Drove additional 5 feet of conductor casing to 18 feet below channel bottom.
-12.0	21.0											
-13.0	22.0			CL/ GC	GRAVELLY CLAY drill cuttings @ 21.5-25 feet.							
-14.0	23.0											
-15.0	24.0											
-16.0	25.0			ML	Grades to dark grayish brown SILT.							
-17.0	26.0											
-18.0	27.0											
-19.0	28.0											
-20.0	29.0											
-21.0	30.0		3 4 5	CL	LEAN CLAY WITH GRAVEL, gray (Gley 1 5/N), stiff, wet, medium plasticity, cohesive, ~10% fine gravel.				65			Used catcher. 10% recovery.
-22.0	31.0											
-23.0	32.0											
-24.0	33.0											
-25.0	34.0											
-26.0	35.0			MH	ELASTIC SILT WITH SAND, dark gray (Gley 1 4/N), stiff, wet, high plasticity, cohesive, fine sand.							Piston core sample. Pushed 24"; 6" recovery.
-27.0	36.0											
-28.0	37.0											

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

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812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Hammond Bridge Replacement
LOCATION: South edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 2 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/21/15-1/22/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

BORING NUMBER B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-29.0	38.0												
-30.0	39.0												
-31.0	40.0		4 7 7	ML		SILT, very dark gray (Gley 1 3/N), stiff, moist, medium plasticity, cohesive; thin layers (<10mm) of interbedded organics at 40.5-40.8'.				90			Used catcher. 100% recovery.
-32.0	41.0												
-33.0	42.0												
-34.0	43.0												
-35.0	44.0												
-36.0	45.0												
-37.0	46.0												
-38.0	47.0												
-39.0	48.0												
-40.0	49.0												
-41.0	50.0		7 7 8	ML/SM		Alternating lenses of SANDY SILT and SILTY SAND, stiff to medium dense, wet, non-plastic, slightly cohesive where mostly silty, fine sand.				71			Used catcher. 100% recovery.
-42.0	51.0												
-43.0	52.0												
-44.0	53.0												
-45.0	54.0												
-46.0	55.0												
-47.0	56.0												
-48.0	57.0												
-49.0	58.0												
-50.0	59.0												
-51.0	60.0		5 7 8	ML		SILT, dark gray, stiff, moist, stiff, non-plastic to low plasticity, slightly cohesive, trace fine sand; trace organics at 61'.				96			Used catcher. 100% recovery.
-52.0	61.0												
-53.0	62.0												
-54.0	63.0												
-55.0	64.0												
-56.0	65.0												
-57.0	66.0												
-58.0	67.0												
-59.0	68.0												
-60.0	69.0												
-61.0	70.0		12 18 25	GM		SILTY GRAVEL WITH SAND, greenish gray (Gley 1 5/10Y), wet, dense, non-plastic matrix, clast supported, fine subrounded gravel, fresh to slightly weathered fine to medium sand; top of FALOR FORMATION.							Used catcher. 75% recovery. Falor Formation
-62.0	71.0												
-63.0	72.0												
-64.0	73.0												
-65.0	74.0												
-66.0	75.0												

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LOG OF BORING

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PROJECT: Hammond Bridge Replacement
LOCATION: South edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 2 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/21/15-1/22/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

BORING NUMBER B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-66.0	75.0												
-67.0	76.0												
-68.0	77.0												
-69.0	78.0												
-70.0	79.0												
-71.0	80.0		19 28 59	SM		SILTY SAND WITH GRAVEL, dark yellowish brown (10 YR 4/6), very dense, wet, non plastic, no cementation, fine subangular to subrounded gravels, gravels are weathered and oxidized.							Used catcher. 50% recovery.
-72.0	81.0												
-73.0	82.0												
-74.0	83.0												
-75.0	84.0												
-76.0	85.0		28 42 36	GW		WELL-GRADED GRAVEL WITH SAND, fractured cobbles in cutting shoe, very dense, wet, fine to coarse gravel, few fractured cobbles with subrounded intact surfaces.							Used catcher. 30% recovery. Stop for day.
-77.0	86.0												
-78.0	87.0												
-79.0	88.0												Begin drilling 1/22/15 at 0800 hrs.
-80.0	89.0												
-81.0	90.0												
-82.0	91.0												
-83.0	92.0												
-84.0	93.0												
-85.0	94.0												
-86.0	95.0												
-87.0	96.0												
-88.0	97.0												
-89.0	98.0												
-90.0	99.0												
-91.0	100.0		65	GP		POORLY GRADED GRAVEL, dark yellowish brown, very dense, cobbles fractured from hammer blow, mostly sandstone fragments with slightly weathered subrounded to subangular intact faces, medium to coarse sand matrix.							Used catcher.
-92.0	101.0												
-93.0	102.0												
-94.0	103.0												
-95.0	104.0												
-96.0	105.0												
-97.0	106.0												
-98.0	107.0												
-99.0	108.0												
-100.0	109.0												
-101.0	110.0												
-102.0	111.0												
-103.0	112.0												

LOG OF BORING

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PROJECT: Hammond Bridge Replacement
LOCATION: South edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 2 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/21/15-1/22/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

BORING NUMBER B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-104.0	113.0												
-105.0	114.0												
-106.0	115.0		115	SW		WELL-GRADED SAND WITH GRAVEL, brown (7.5 YR 4/3), very dense, wet, non-plastic, medium cementation, consolidated, fine to coarse sand, trace silt, subrounded fine gravels, oxidized, matrix supported.							Used catcher. 50% recovery.
-107.0	116.0												
-108.0	117.0												
-109.0	118.0												
-110.0	119.0												
-111.0	120.0												
-112.0	121.0												
-113.0	122.0												
-114.0	123.0												
-115.0	124.0												
-116.0	125.0												
-117.0	126.0												
-118.0	127.0												
-119.0	128.0												
-120.0	129.0												
-121.0	130.0												
-122.0	131.0												
-123.0	132.0												
-124.0	133.0												
-125.0	134.0												
-126.0	135.0		157/ 5.5"	SM		SILTY SAND WITH GRAVEL, strong brown, very dense, wet, non-plastic, medium cementation, well consolidated, fine rounded gravel, matrix supported, oxidized sandstone clasts, fresh.							Used catcher. 75% recovery.
-127.0	136.0												
-128.0	137.0												
-129.0	138.0												
-130.0	139.0												
-131.0	140.0												
-132.0	141.0												
-133.0	142.0												
-134.0	143.0												
-135.0	144.0												
-136.0	145.0												
-137.0	146.0												
-138.0	147.0												
-139.0	148.0												
-140.0	149.0												
-141.0	150.0												

LOG OF BORING

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PROJECT: Hammond Bridge Replacement
LOCATION: South edge of channel, downstream from bridge
GROUND SURFACE ELEVATION: 2 feet NAVD 88
EXCAVATION METHOD: Mud Rotary
LOGGED BY: G. Vadurro

JOB NUMBER: 014099.100
DATE DRILLED: 1/21/15-1/22/15
TOTAL DEPTH OF BORING: 201 feet
SAMPLER TYPE: SPT/MCS/Piston Core
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

BORING NUMBER
B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-141.0	150.0												
-142.0	151.0												
-143.0	152.0												
-144.0	153.0												
-145.0	154.0												
-146.0	155.0		48 68	SP		POORLY GRADED SAND, olive brown (2.5 Y 4/3), very dense, wet, non-plastic, no cementation, fine to medium quartz-rich sand, trace silt.							Used catcher. 100% recovery.
-147.0	156.0												
-148.0	157.0												
-149.0	158.0												
-150.0	159.0												
-151.0	160.0												
-152.0	161.0												
-153.0	162.0												
-154.0	163.0												
-155.0	164.0												
-156.0	165.0												
-157.0	166.0												
-158.0	167.0												
-159.0	168.0												
-160.0	169.0												
-161.0	170.0												Gravelly beginning at about 170'.
-162.0	171.0												
-163.0	172.0												
-164.0	173.0												
-165.0	174.0												
-166.0	175.0		85	SM		SILTY SAND WITH GRAVEL, dark greenish gray (Gley 2 4/5 BG), very dense, wet, low plasticity matrix, weak cementation, fine to coarse sand, subrounded fine gravel, slightly weathered to fresh.							Used catcher. 100% recovery.
-167.0	176.0												
-168.0	177.0												
-169.0	178.0												
-170.0	179.0												
-171.0	180.0												
-172.0	181.0												
-173.0	182.0												
-174.0	183.0												
-175.0	184.0												
-176.0	185.0												
-177.0	186.0												
-178.0	187.0												

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

Consulting Engineers & Geologists, Inc.

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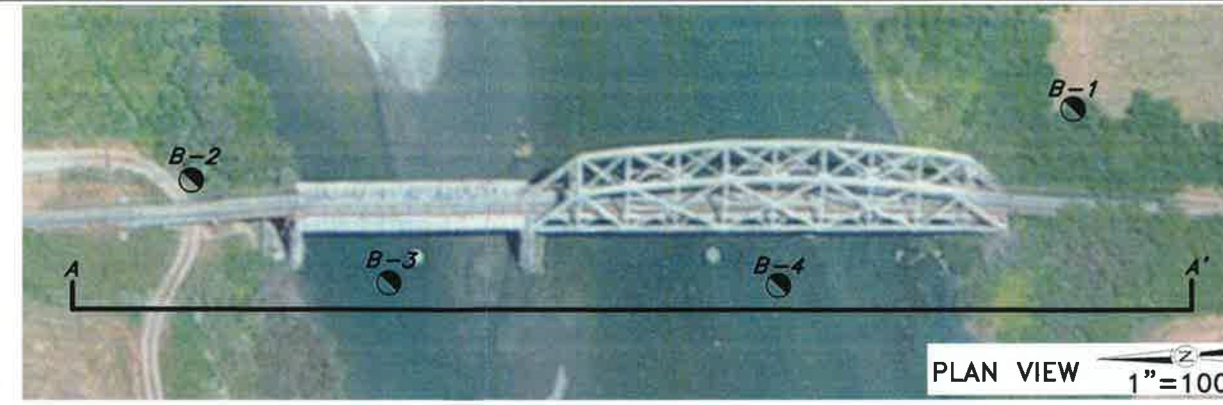
PROJECT: Hammond Bridge Replacement
JOB NUMBER: 014099.100
LOCATION: South edge of channel, downstream from bridge
DATE DRILLED: 1/21/15-1/22/15
GROUND SURFACE ELEVATION: 2 feet NAVD 88
TOTAL DEPTH OF BORING: 201 feet
EXCAVATION METHOD: Mud Rotary
SAMPLER TYPE: SPT/MCS/Piston Core
LOGGED BY: G. Vadurro
DEPTH OF WATER TO CHANNEL BOTTOM: 7 feet At High Tide (11:30)

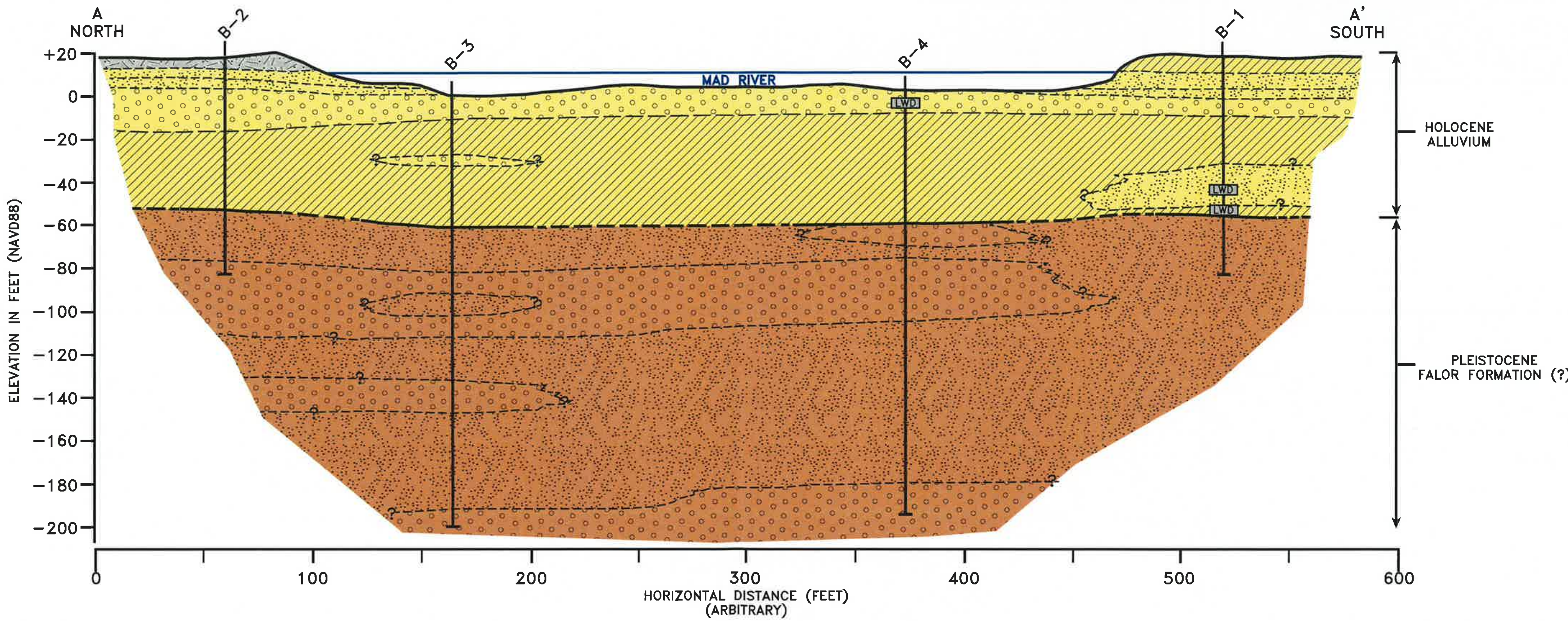
BORING NUMBER B-4

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	REMARKS
-179.0	188.0												
-180.0	189.0												
-181.0	190.0		155	GP		POORLY GRADED GRAVEL, very dense, coarse rounded gravel and cobbles fractured from hammer blow, medium to coarse sand and slightly clayey matrix.							Used catcher. 10% recovery.
-182.0	191.0												
-183.0	192.0												Significant increase in drill resistance at 192-197'.
-184.0	193.0												
-185.0	194.0												
-186.0	195.0												
-187.0	196.0												
-188.0	197.0					SILTY GRAVEL, strong brown (7.5 YR 5/6), very dense, moist, weak cementation, well consolidated, fine to coarse subangular to subrounded gravel, few fractured cobbles.							
-189.0	198.0												
-190.0	199.0												
-191.0	200.0		48 138/ 4.5"	GM									Used catcher; sampler penetration refusal. 60% recovery.
-192.0	201.0												
-193.0	202.0					Borehole completed to 201 feet below barge deck (194 feet below channel bottom); backfilled with neat cement.							
-194.0	203.0												
-195.0	204.0												
-196.0	205.0												
-197.0	206.0												
-198.0	207.0												
-199.0	208.0												
-200.0	209.0												
-201.0	210.0												
-202.0	211.0												
-203.0	212.0												
-204.0	213.0												
-205.0	214.0												
-206.0	215.0												
-207.0	216.0												
-208.0	217.0												
-209.0	218.0												
-210.0	219.0												
-211.0	220.0												
-212.0	221.0												
-213.0	222.0												
-214.0	223.0												
-215.0	224.0												

LOG OF BORING

INTERPRETED GEOLOGIC CROSS SECTION
HAMMOND TRAIL PEDESTRIAN BRIDGE





EXPLANATION

FILL	HISTORIC FILL	CONTACT; DASHED WHERE INFERRED, QUERRIED WHERE UNCERTAIN
SILT/CLAY	HOLOCENE ALLUVIUM INCLUDES ORGANIC MATERIAL	BORING LOCATION
SILTY SAND/SAND	PLEISTOCENE FALOR FORMATION CONTAINS SAND, GRAVELS AND COBBLES	LWD LOCATION WHERE LARGE WOODY DEBRIS WAS ENCOUNTERED
GRAVEL/COBBLE		

SCALE: 1"=50'
NO VERTICAL EXAGGERATION
VIEW UPSTREAM; TO THE EAST

SHN Consulting Engineers & Geologists, Inc.

Humboldt County Public Works
Hammond Bridge Geotech
Arcata, California

Geologic Cross Section
A - A'
SHN 014099

May 2015 | Figure6_GeologicCrossSection.pdf | Figure 6

APPENDIX C

Shear Wave Velocity Calculations

Liquefaction Triggering and Seismic Settlement Calculations

Shear Wave Velocity (Vs)

Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021

B1
South Abutment
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
Job No: **23-948.9**
Date: **4/16/24**
Boring: **B1**
Support: **South Abutment**

Hammer Efficiency (ER): 80.0 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	**Sand**	**Clay/Silt**	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Class.	Soil Type	Quaternary, Holocene or Pleistocene Enter Q, H or P *	Age Scaling Factor ASF (dim.)	Undrained Shear Strength S_u (psf)	Rock **	N (bpf)	N_{SPT} (bpf) ***	N_{60} (bpf)	N_{60} (bpf) (≤100)	σ'$_v$ (ksf)	σ'$_v$ (kPa)	d Layer Thickness in upper 30 m (m)
1	6.5	7.5	7.5	2.4	SP	SAND	H	0.90			14	6	8	8	0.75	36.10	2.29
2	11.5	15.0	7.5	1.4	SP	SAND	H	0.90			9	9	12	12	1.33	63.87	2.29
3	16.5	20.0	5.0	2.4	SP-SM	SAND	H	0.90			8	3	4	4	1.82	87.23	1.52
4	21.5	25.0	5.0	1.4	GW	GRAVEL	H	NA			11	11	15	15	2.11	101.24	1.52
5	26.5	27.5	2.5	2.4	GW	GRAVEL	H	NA			14	6	8	8	2.45	117.42	0.76
6	29.0	30.0	2.5	2.4	ML	SILT	H	0.88	610		13	5	7	7	2.60	124.44	0.76
7	31.5	35.0	5.0	1.4	ML	SILT	H	0.88			6	6	8	8	2.73	130.73	1.52
8	36.0	40.0	5.0	2.4	ML	SILT	H	0.88	550		19	8	10	10	2.97	142.06	1.52
9	41.5	45.0	5.0	1.4	ML	SILT	H	0.88			8	8	11	11	3.26	155.92	1.52
10	46.5	50.0	5.0	2.4	ML	SILT	H	0.88			22	9	12	12	3.53	169.23	1.52
11	51.5	60.0	10.0	1.4	SM	SAND	H	0.90			12	12	16	16	3.85	184.21	3.05
12	61.5	70.0	10.0	2.4	SP-SM	SAND	H	0.90			39	16	21	21	4.48	214.33	3.05
13	71.5	75.0	5.0	1.4	OL	CLAY	H	0.88			12	12	16	16	5.11	244.76	1.52
14	76.5	80.0	5.0	2.4	SC	SAND	P	1.17			48	20	26	26	5.41	259.27	1.52
15	81.5	90.0	10.0	1.4	SW	SAND	P	1.17			33	33	44	44	5.75	275.45	3.05
16	90.8	100.0	10.0	2.4	SW	SAND	P	1.17			100	41	55	55	6.38	305.39	2.57
17	101.5	101.5	1.5	1.4	SP-SC	SAND	P	1.17			53	53	71	71	7.10	340.18	NA
																Sum (d) =	30.00

Sample Number	Layer Shear Wave Velocity, Vs: SAND (m/sec)	GRAVEL (m/sec)	SILT/CLAY[1] (m/sec)	SILT/CLAY[2] (m/sec)	Sedimentary Rock (m/sec)	Soil/Rock Profile: Profile Vs (m/sec)	Profile D/Vs (sec)
1	99					99	0.023
2	124					124	0.018
3	104					104	0.015
4		204				204	0.007
5		186				186	0.004
6			112			112	0.007
7				155		155	0.010
8			107			107	0.014
9				173		173	0.009
10				180		180	0.008
11	170					170	0.018
12	187					187	0.016
13				213		213	0.007
14	267					267	0.006
15	305					305	0.010
16	329					329	0.008
17	358					NA	NA
						Sum (D/Vs) =	0.181

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
*** Corrected for sample diameter

Estimated Shear Wave Velocity (V_{S30}) for Depth of Exploration 10 to 29 m	**Time Averaged Shear Wave Velocity in Upper 30 m (V_{S30})**
V_{S30} =[1.45 - (0.015*d)]*Vs(d)	$V_{S30} = \dfrac{30\ \text{meters}}{[\ (D_1/Vs_1) + (D_2/Vs_2) + \ldots + (D_N/Vs_N)\ ]}$

d (m)	30.00
Vs(d) (m/sec)	166
V_{S30} (m/sec)	**166**
V_{S30} (ft/sec)	**545**

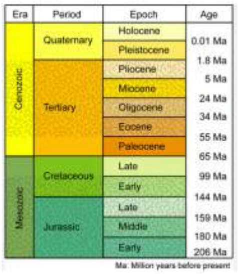
Soil Profile Type

'E' (Vs < 180 m/s)

Check for Soil Profile Type F

Shear Wave Velocity Correlations (valid for $3 \le N_{60} \le 100$)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$ [meters/second]
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ [meters/second] for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ [meters/second] for Pleistocene
Clay[1]: $Vs = 203(S_u/P_a)^{0.475}$ [meters/second]
Clay[2]: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$ [meters/second]
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \le 560$ meters/second
P_a = Atmospheric Pressure = 2116.2 psf



Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) based on SPT correlations are valid where $3 \le N_{60} \le 100$.
4) For Vs calculation the Undrained Shear Strength (S_u) is based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane in psf.

The geo-professional should be aware of the limitations of each correlation used. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data that are reliable and representative of the actual site conditions. If correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available in a region, then in-situ measurements are recommended.

Shear Wave Velocity (Vs)

B2
North Abutment
Hammond Trail Pedestrian Bridge

Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021

Project: **Hammond Trail Pedestrian Bridge**
Job No: **23-948.9**
Date: **4/16/24**
Boring: **B2**
Support: **North Abutment**

Hammer Efficiency (ER): 80.0 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	**Sand**	**Clay/Silt**	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Class.	Soil Type	Quaternary, Holocene or Pleistocene Enter Q, H or P *	Age Scaling Factor ASF (dim.)	Undrained Shear Strength S_u (psf)	Rock **	N (bpf)	N_{SPT} (bpf) ***	N_{60} (bpf)	N_{60} (bpf) (≤100)	σ'_v (ksf)	σ'_v (kPa)	d Layer Thickness in upper 30 m (m)
1	6.5	10.0	10.0	1.4	SM	SAND	H	0.90			37	37	49	49	0.85	40.46	3.05
2	11.5	15.0	5.0	2.4	SW	SAND	H	0.90			16	7	9	9	1.49	71.15	1.52
3	16.5	20.0	5.0	1.4	GW	GRAVEL	H	NA			8	8	11	11	2.01	96.43	1.52
4	21.5	25.0	5.0	2.4	GW	GRAVEL	H	NA			21	9	11	11	2.33	111.41	1.52
5	26.5	30.0	5.0	1.4	GW	GRAVEL	H	NA			39	39	52	52	2.65	126.90	1.52
6	31.5	35.0	5.0	2.4	GW	GRAVEL	H	NA			32	13	17	17	3.00	143.56	1.52
7	36.5	40.0	5.0	1.4	CL	CLAY	H	0.88			8	8	11	11	3.34	159.79	1.52
8	41.5	45.0	5.0	2.4	ML	SILT	H	0.88			10	4	5	5	3.65	174.81	1.52
9	46.5	50.0	5.0	1.4	ML	SILT	H	0.88			5	5	7	7	3.95	189.31	1.52
10	51.5	60.0	10.0	2.4	ML	SILT	H	0.88			14	6	8	8	4.26	203.82	3.05
11	61.5	70.0	10.0	1.4	SP-SM	SAND	P	1.17			22	22	29	29	4.87	232.98	3.05
12	71.5	80.0	10.0	2.4	SM	SAND	P	1.17			58	24	32	32	5.50	263.24	3.05
13	81.5	85.0	5.0	1.4	SW	SAND	P	1.17			81	81	108	100	6.17	295.20	1.52
14	90.8	95.0	10.0	2.4	SW	SAND	P	1.17			100	41	55	55	6.79	325.14	3.05
15	101.5	101.5	6.5	1.4	GW	GRAVEL	P	NA			100	100	134	100	7.52	359.93	1.04
																Sum (d) =	**30.00**

Sample Number	Layer Shear Wave Velocity, Vs: SAND (m/sec)	GRAVEL (m/sec)	SILT/CLAY[1] (m/sec)	SILT/CLAY[2] (m/sec)	Sedimentary Rock (m/sec)	Soil/Rock Profile: Profile Vs (m/sec)	Profile D/Vs (sec)
1	155					155	0.020
2	119					119	0.013
3		190				190	0.008
4		195				195	0.008
5		268				268	0.006
6		222				222	0.007
7				174		174	0.009
8				157		157	0.010
9				171		171	0.009
10				179		179	0.017
11	267					267	0.011
12	281					281	0.011
13	374					374	0.004
14	334					334	0.009
15		510				510	0.002
						Sum (D/Vs) =	**0.143**

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
*** Corrected for sample diameter

Estimated Shear Wave Velocity (Vs_{30}) for Depth of Exploration 10 to 29 m	**Time Averaged Shear Wave Velocity in Upper 30 m (Vs_{30})**
$Vs_{30} = [1.45 - (0.015*d)]*Vs(d)$	$Vs_{30} = \dfrac{30 \text{ meters}}{[(D_1/Vs_1) + (D_2/Vs_2) + \dots + (D_N/Vs_N)]}$

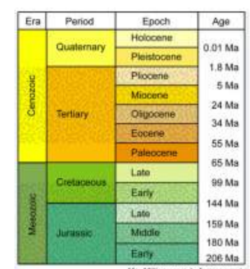
d (m)	30.00
Vs(d) (m/sec)	210
V_{S30} (m/sec)	**210**
V_{S30} (ft/sec)	**689**

Soil Profile Type

'D' (180 m/s < Vs < 360 m/s)

Shear Wave Velocity Correlations (valid for $3 \le N_{60} \le 100$)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$ [meters/second]
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ [meters/second] for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ [meters/second] for Pleistocene
Clay[1]: $Vs = 203(S_u/P_a)^{0.475}$ [meters/second]
Clay[2]: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$ [meters/second]
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \le 560$ meters/second
P_a = Atmospheric Pressure = 2116.2 psf



Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) based on SPT correlations are valid where $3 \le N_{60} \le 100$.
4) For Vs calculation the Undrained Shear Strength (S_u) is based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane in psf.

The geo-professional should be aware of the limitations of each correlation used. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data that are reliable and representative of the actual site conditions. If correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available in a region, then in-situ measurements are recommended.

Shear Wave Velocity (Vs)

Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021

B3
North Pier
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
Job No: **23-948.9**
Date: **4/16/24**
Boring: **B3**
Support: **North Pier**

Hammer Efficiency (ER): 80.0 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	**Sand**	**Clay/Silt**	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Class.	Soil Type	Quaternary, Holocene or Pleistocene — Enter Q, H or P *	Age Scaling Factor ASF (dim.)	Undrained Shear Strength S_u (psf)	Rock **	N (bpf)	N_{SPT} (bpf) ***	N_{60} (bpf)	N_{60} (bpf) (≤100)	σ'_v (ksf)	σ'_v (kPa)	d Layer Thickness in upper 30 m (m)
1	14.5	23.0	23.0	1.4	ML	SILT	H	0.88			4	4	5	5	0.91	43.46	7.01
2	24.5	28.0	5.0	2.4	ML	SILT	H	0.88			12	5	7	7	1.53	73.43	1.52
3	29.5	33.0	5.0	2.4	SM	SAND	H	0.90			18	7	10	10	1.85	88.42	1.52
4	34.5	38.0	5.0	1.4	SM	SAND	H	0.90			15	15	20	20	2.16	103.26	1.52
5	39.0	43.0	5.0	2.4	SM	SAND	H	0.90			14	6	8	8	2.43	116.32	1.52
7	49.5	53.0	10.0	1.4	ML	SILT	H	0.88			14	14	19	19	3.08	147.41	3.05
8	54.5	62.0	9.0	1.4	ML	SILT	H	0.88			8	8	11	11	3.39	162.39	2.74
9	64.5	68.0	6.0	1.4	SP-SM	SAND	P	1.17			36	36	48	48	4.03	192.97	1.83
10	69.5	73.0	5.0	2.4	SP	SAND	P	1.17			97	40	53	53	4.37	209.15	1.52
11	74.5	83.0	10.0	1.4	SM	SAND	P	1.17			82	82	110	100	4.71	225.33	3.05
12	83.8	93.0	10.0	2.4	GW	GRAVEL	P	NA			100	41	55	55	5.33	255.27	3.05
13	94.5	103.0	10.0	1.4	SM	SAND	P	1.17			100	100	134	100	6.06	290.07	1.65
14	103.3	113.0	10.0	1.4	GP-GM	GRAVEL	P	NA			100	100	134	100	6.65	318.39	NA
15	113.5	123.0	10.0	2.4	SM	SAND	P	1.17			100	41	55	55	7.34	351.56	NA
16	123.8	133.0	10.0	1.4	SP	SAND	P	1.17			100	100	134	100	8.04	384.90	NA
17	133.8	148.0	15.0	1.4	GP	GRAVEL	P	NA			100	100	134	100	8.71	417.27	NA
18	154.5	173.0	25.0	1.4	SP	SAND	P	1.17			75	75	100	100	10.11	484.27	NA
19	173.5	193.0	20.0	1.4	SM	SAND	P	1.17			100	100	134	100	11.40	545.76	NA
20	194.0	194.0	1.0	1.4	GP	GRAVEL	P	NA			100	100	134	100	12.78	612.12	NA
																Sum (d) =	**30.00**

Sample Number	Layer Shear Wave Velocity, Vs: SAND (m/sec)	GRAVEL (m/sec)	SILT/CLAY[1] (m/sec)	SILT/CLAY[2] (m/sec)	Sedimentary Rock (m/sec)	Soil/Rock Profile: Profile Vs (m/sec)	Profile D/Vs (sec)
1				101		101	0.070
2				126		126	0.012
3	129					129	0.012
4	156					156	0.010
5	130					130	0.012
7				187		187	0.016
8				175		175	0.016
9	287					287	0.006
10	299					299	0.005
11	352					352	0.009
12		442				442	0.007
13	373					373	0.004
14		502				NA	NA
15	340					NA	NA
16	398					NA	NA
17		519				NA	NA
18	420					NA	NA
19	431					NA	NA
20		543				NA	NA
						Sum (D/Vs) =	**0.179**

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
*** Corrected for sample diameter

Estimated Shear Wave Velocity (Vs_{30}) for Depth of Exploration 10 to 29 m	Time Averaged Shear Wave Velocity in Upper 30 m (Vs_{30})
Vs_{30} =[1.45 - (0.015*d)]*Vs(d)	$Vs_{30} = \dfrac{30 \text{ meters}}{[\,(D_1/Vs_1) + (D_2/Vs_2) + \ldots + (D_N/Vs_N)\,]}$

d (m)	30.00
Vs(d) (m/sec)	168
V_{S30} (m/sec)	**168**
V_{S30} (ft/sec)	**551**

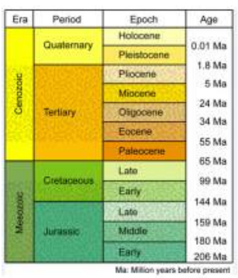
Soil Profile Type

'E' (Vs < 180 m/s)

Check for Soil Profile Type F

Shear Wave Velocity Correlations (valid for $3 \le N_{60} \le 100$)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$ [meters/second]
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ [meters/second] for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ [meters/second] for Pleistocene
Clay[1]: $Vs = 203(S_u/P_a)^{0.475}$ [meters/second]
Clay[2]: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$ [meters/second]
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \le 560$ meters/second
P_a = Atmospheric Pressure = 2116.2 psf



Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) based on SPT correlations are valid where $3 \le N_{60} \le 100$.
4) For Vs calculation the Undrained Shear Strength (S_u) is based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane in psf.

The geo-professional should be aware of the limitations of each correlation used. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data that are reliable and representative of the actual site conditions. If correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available in a region, then in-situ measurements are recommended.

Shear Wave Velocity (Vs)

Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021

B4
South Pier
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
Job No: **23-948.9**
Date: **4/16/24**
Boring: **B4**
Support: **South Pier**

Hammer Efficiency (ER): 80.0 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	Sand	Clay/Silt	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Class.	Soil Type	Quaternary, Holocene or Pleistocene Enter Q, H or P *	Age Scaling Factor ASF (dim.)	Undrained Shear Strength S_u (psf)	Rock **	N (bpf)	N_{SPT} (bpf) ***	N_{60} (bpf)	N_{60} (bpf) (≤100)	σ'_v (ksf)	σ'_v (kPa)	d Layer Thickness in upper 30 m (m)	Layer Shear Wave Velocity, Vs: SAND (m/sec)	GRAVEL (m/sec)	SILT/CLAY[1] (m/sec)	SILT/CLAY[2] (m/sec)	Sedimentary Rock (m/sec)	Soil/Rock Profile: Profile Vs (m/sec)	Profile D/Vs (sec)
1	14.5	18.0	18.0	2.4	GP	GRAVEL	H	NA			9	4	5	5	1.03	49.44	5.49		145				145	0.038
2	24.5	28.0	10.0	1.4	CL	CLAY	H	0.88			9	9	12	12	1.66	79.41	3.05				142		142	0.022
4	34.5	43.0	15.0	1.4	ML	SILT	H	0.88			14	14	19	19	2.28	109.38	4.57				170		170	0.027
5	44.5	53.0	10.0	1.4	ML	SILT	H	0.88			15	15	20	20	2.91	139.35	3.05				185		185	0.016
6	54.5	63.0	10.0	1.4	ML	SILT	H	0.88			15	15	20	20	3.54	169.33	3.05				197		197	0.015
7	64.5	73.0	10.0	1.4	GM	GRAVEL	P	NA			43	43	58	58	4.17	199.66	3.05		433				433	0.007
8	74.5	78.0	5.0	1.4	SM	SAND	P	1.17			87	87	116	100	4.85	232.03	1.52	354					354	0.004
9	79.5	93.0	15.0	1.4	GW	GRAVEL	P	NA			78	78	104	100	5.18	248.21	4.57		488				488	0.009
10	93.5	108.0	15.0	1.4	GP	GRAVEL	P	NA			100	100	134	100	6.13	293.52	1.65		498				498	0.003
11	108.5	128.0	20.0	1.4	SP	SAND	P	1.17			100	100	134	100	7.14	342.07	NA	387					NA	NA
12	128.5	148.0	20.0	1.4	SM	SAND	P	1.17			100	100	134	100	8.50	406.81	NA	403					NA	NA
13	149.0	168.0	20.0	1.4	SP	SAND	P	1.17			100	100	134	100	9.88	473.16	NA	417					NA	NA
14	168.5	183.0	15.0	1.4	SM	SAND	P	1.17			100	100	134	100	11.20	536.28	NA	430					NA	NA
15	183.5	193.0	10.0	1.4	GP	GRAVEL	P	NA			100	100	134	100	12.21	584.83	NA		540				NA	NA
16	194.0	194.0	1.0	1.4	GM	GRAVEL	P	NA			100	100	134	100	12.92	618.81	NA		544				NA	NA
																Sum (d) =	30.00						Sum (D/Vs) =	0.142

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
*** Corrected for sample diameter

Estimated Shear Wave Velocity (Vs_{30}) for Depth of Exploration 10 to 29 m	Time Averaged Shear Wave Velocity in Upper 30 m (Vs_{30})
Vs_{30} =[1.45 - (0.015*d)]*Vs(d)	$Vs_{30} = \frac{30 \text{ meters}}{[(D_1/Vs_1) + (D_2/Vs_2) + \ldots + (D_N/Vs_N)]}$

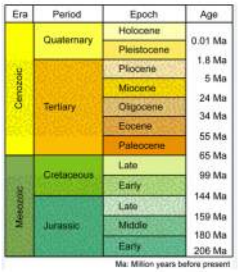
d (m)	30.00
Vs(d) (m/sec)	211
V_{S30} (m/sec)	**211**
V_{S30} (ft/sec)	**692**

Soil Profile Type

'D' (180 m/s < Vs < 360 m/s)

Shear Wave Velocity Correlations (valid for $3 \le N_{60} \le 100$)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$ [meters/second]
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ [meters/second] for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ [meters/second] for Pleistocene
Clay[1]: $Vs = 203(S_u/P_a)^{0.475}$ [meters/second]
Clay[2]: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$ [meters/second]
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \le 560$ meters/second
P_a = Atmospheric Pressure = 2116.2 psf



Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) based on SPT correlations are valid where $3 \le N_{60} \le 100$.
4) For Vs calculation the Undrained Shear Strength (S_u) is based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane in psf.

The geo-professional should be aware of the limitations of each correlation used. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data that are reliable and representative of the actual site conditions. If correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available in a region, then in-situ measurements are recommended.

Summary of Liquefaction Analysis

Boring: B1
South Abutment
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
File No.: **23-948.9**
Date: **04/18/24**
Boring: **B1**
Support: **South Abutment**
By: **KBH**

Boring Elevation: 18.0 feet
Analysis Ground Water Depth = 15.0 feet
Analysis Ground Water Elevation = 3.0 feet

Hammer Energy (ER) = 80 %

Mw = 8.63
PGA = 1.03 g
Distance to Fault (R) = 9.2 miles

Liquefaction Factor of Safety (FS_L): 1.00

NA = Not Applicable - Soil layer above groundwater
NL = Non-Liquefiable
Note that soils with $(N1)_{60cs}$ > 30 are not considered susceptible to liquefaction irrespective of the other criteria or conditions.

Soil Layer	Depth to Bottom of Layer Below Ground Surface (feet)	Elevation to Bottom of Layer Below Ground Surface (feet)	Layer Thickness: Total Layer Thickness (feet)	Layer Thickness: Unsaturated Layer Thickness (feet)	Layer Thickness: Saturated Layer Thickness (feet)	Soil Type (USCS)	SPT N (blows/foot)	$(N1)_{60}$ (blows/foot)	$(N1)_{60cs}$ (blows/foot)	Soil Total Unit Weight (pcf)	Plasticity Index (PI)	Fines Content (%)	Factor of Safety Against Liquefaction FS_L	Potential Liquifiable Soil Layer FS_L < 1.00	Seismic Settlement: Dry Layers (inches)	Seismic Settlement: Saturated Layers (inches)	Seismic Settlement: Total (inches)	Undrained Residual Shear Strength Sr (psf)
1	7.5	10.5	7.5	7.5	0.0	SP	6	10	10	116		5	NA	NA	0.22		0.22	
2	15.0	3.0	7.5	7.5	0.0	SP	9	15	15	116		5	NA	NA	0.19		0.19	
3	20.0	-2.0	5.0	0.0	5.0	SP-SM	3	4	5	117		10	0.07	x		2.28	2.28	143
4	25.0	-7.0	5.0	0.0	5.0	GW	11	17	17	130		5	0.16	x		1.11	1.11	620
5	27.5	-9.5	2.5	0.0	2.5	GW	6	7	7	130		5	0.07	x		0.95	0.95	224
6	30.0	-12.0	2.5	0.0	2.5	ML	5	6	12	115	23	60	NL	NL				
7	35.0	-17.0	5.0	0.0	5.0	ML	6	9	15	115	23	60	NL	NL				
8	40.0	-22.0	5.0	0.0	5.0	ML	8	9	16	115	23	60	NL	NL				
9	45.0	-27.0	5.0	0.0	5.0	ML	8	10	17	115	23	60	NL	NL				
10	50.0	-32.0	5.0	0.0	5.0	ML	9	9	16	125		68	0.12	x		1.13	1.13	365
11	60.0	-42.0	10.0	0.0	10.0	SM	12	14	17	125		15	0.13	x		2.16	2.16	667
12	70.0	-52.0	10.0	0.0	10.0	SP-SM	16	15	15	127		8	0.12	x		2.34	2.34	766
														SUM:	0.41	9.97	10.38	

Liquefaction Analysis - Youd et. Al., (2001): Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils October, 2001.
Fines Correction - Seed and Idris formula (1997).
Fines Susceptibility - Boulanger and Idriss (2006) / fine grained soils with PI >= 7 exhibit clay-like behavior; if a soil plots as CL-ML, the PI criterion may be reduced to PI >= 5 and still be consistent with the available data.
According the the Caltrans Geotechnical Manual - Soils that should be considered potententially liquefiable are sands, low plasticity silts (PI<7), and, in unusual cases, gravel. Rock and most clay soil are not liquefiable.
Undrained Residual Shear Strength - Kramer, S. and Wang, C.H., (2015), "Empirical Model for Esimation of the Residual Strength of Liquefied Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE
as cited in Caltrans (2017), Memo To Designers (MTD) 20-15, Lateral Spreading Analysis for New and Existing Bridges.
SPT N - Where applicable the Sampler Size Conversion to SPT N-value is in accoradance with Caltrans Geotechnical Manual March 2021.
Seismic Settlement of Dry Layers - Pradel, Daniel, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, April 1998, pages 364 - 368.
Seismic Settlement of Saturated Layers - Lee, C.Y., Asian Research Publishing Network Journal of Engineering and Applied Sciences, 2006-2007. This approach approximates the Volumetric Strain (ev) and is based on the Tokimatsu and Seed (1987) procedure.

Summary of Liquefaction Analysis

Boring: B2
North Abutment
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
File No.: **23-948.9**
Date: **04/18/24**
Boring: **B2**
Support: **North Abutment**
By: **KBH**

Boring Elevation: 18.0 feet
Analysis Ground Water Depth = 15.0 feet
Analysis Ground Water Elevation = 3.0 feet

Hammer Energy (ER) = 80 %

Mw = 8.63
PGA = 1.03 g
Distance to Fault (R) = 9.2 miles

Liquefaction Factor of Safety (FS_L): 1.00

NA = Not Applicable - Soil layer above groundwater
NL = Non-Liquefiable
Note that soils with $(N1)_{60cs}$ > 30 are not considered susceptible to liquefaction irrespective of the other criteria or conditions.

Soil Layer	Depth to Bottom of Layer Below Ground Surface (feet)	Elevation to Bottom of Layer Below Ground Surface (feet)	Layer Thickness: Total Layer Thickness (feet)	Layer Thickness: Unsaturated Layer Thickness (feet)	Layer Thickness: Saturated Layer Thickness (feet)	Soil Type (USCS)	SPT N (blows/foot)	$(N1)_{60}$ (blows/foot)	$(N1)_{60cs}$ (blows/foot)	Soil Total Unit Weight (pcf)	Plasticity Index (PI)	Fines Content (%)	Factor of Safety Against Liquefaction FS_L	Potential Liquifiable Soil Layer FS_L < 1.00	Seismic Settlement: Dry Layers (inches)	Seismic Settlement: Saturated Layers (inches)	Seismic Settlement: Total (inches)	Undrained Residual Shear Strength Sr (psf)
1	10.0	8.0	10.0	10.0	0.0	SM	37	70	76	130		15	NA	NA	0.02		0.02	
2	15.0	3.0	5.0	5.0	0.0	SW	7	9	9	124		5	NA	NA	0.32		0.32	
3	20.0	-2.0	5.0	0.0	5.0	GW	8	11	11	125		3	0.13	x		1.41	1.41	331
4	25.0	-7.0	5.0	0.0	5.0	GW	9	10	10	125		3	0.10	x		1.47	1.47	328
5	30.0	-12.0	5.0	0.0	5.0	GW	39	53	53	132		3	NL	NL				
6	35.0	-17.0	5.0	0.0	5.0	GW	13	15	15	132		3	0.12	x		1.20	1.20	601
7	40.0	-22.0	5.0	0.0	5.0	CL	8	10	17	126	26	90	NL	NL				
8	45.0	-27.0	5.0	0.0	5.0	ML	4	4	10	123		60	0.09	x		1.51	1.51	213
9	50.0	-32.0	5.0	0.0	5.0	ML	5	6	12	123		60	0.10	x		1.34	1.34	268
10	60.0	-42.0	10.0	0.0	10.0	ML	6	5	12	123		50	0.10	x		2.77	2.77	270
11	70.0	-52.0	10.0	0.0	10.0	SP-SM	22	23	24	125		8	0.20	x		1.79	1.79	2026
														SUM:	0.34	11.49	11.83	

Liquefaction Analysis - Youd et. Al., (2001): Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils October, 2001.
Fines Correction - Seed and Idris formula (1997).
Fines Susceptibility - Boulanger and Idriss (2006) / fine grained soils with PI >= 7 exhibit clay-like behavior; if a soil plots as CL-ML, the PI criterion may be reduced to PI >= 5 and still be consistent with the available data.
According the the Caltrans Geotechnical Manual - Soils that should be considered potententially liquefiable are sands, low plasticity silts (PI<7), and, in unusual cases, gravel. Rock and most clay soil are not liquefiable.
Undrained Residual Shear Strength - Kramer, S. and Wang, C.H., (2015), "Empirical Model for Esimation of the Residual Strength of Liquefied Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE as cited in Caltrans (2017), Memo To Designers (MTD) 20-15, Lateral Spreading Analysis for New and Existing Bridges.
SPT N - Where applicable the Sampler Size Conversion to SPT N-value is in accoradance with Caltrans Geotechnical Manual March 2021.
Seismic Settlement of Dry Layers - Pradel, Daniel, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, April 1998, pages 364 - 368.
Seismic Settlement of Saturated Layers - Lee, C.Y., Asian Research Publishing Network Journal of Engineering and Applied Sciences, 2006-2007. This approach approximates the Volumetric Strain (ev) and is based on the Tokimatsu and Seed (1987) procedure.

Summary of Liquefaction Analysis

Boring: B3
North Pier
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
File No.: **23-948.9**
Date: **04/18/24**
Boring: **B3**
Support: **North Pier**
By: **KBH**

Boring Elevation: 0.0 feet (at Channel Bottom)
Analysis Ground Water Depth = 0.0 feet
Analysis Ground Water Elevation = 0.0 feet
Hammer Energy (ER) = 80 %

Mw = 8.63
PGA = 1.03 g
Distance to Fault (R) = 9.2 miles
Liquefaction Factor of Safety (FS_L): 1.00

NA = Not Applicable - Soil layer above groundwater
NL = Non-Liquefiable
Note that soils with $(N1)_{60cs}$ > 30 are not considered susceptible to liquefaction irrespective of the other criteria or conditions.

Soil Layer	Depth to Bottom of Layer Below Ground Surface (feet)	Elevation to Bottom of Layer Below Ground Surface (feet)	Layer Thickness: Total Layer Thickness (feet)	Layer Thickness: Unsaturated Layer Thickness (feet)	Layer Thickness: Saturated Layer Thickness (feet)	Soil Type (USCS)	SPT N (blows/foot)	$(N1)_{60}$ (blows/foot)	$(N1)_{60cs}$ (blows/foot)	Soil Total Unit Weight (pcf)	Plasticity Index (PI)	Fines Content (%)	Factor of Safety Against Liquefaction FS_L	Potential Liquifiable Soil Layer FS_L < 1.00	Seismic Settlement: Dry Layers (inches)	Seismic Settlement: Saturated Layers (inches)	Seismic Settlement: Total (inches)	Undrained Residual Shear Strength Sr (psf)
1	23.0	-23.0	23.0	0.0	23.0	ML	4	9	16	125		50	0.09	x		5.19	5.19	157
2	28.0	-28.0	5.0	0.0	5.0	ML	5	7	14	125		97	0.08	x		1.24	1.24	188
3	33.0	-33.0	5.0	0.0	5.0	SM	7	11	17	125		30	0.10	x		1.10	1.10	293
4	38.0	-38.0	5.0	0.0	5.0	SM	15	24	34	123		44	NL	NL				
5	43.0	-43.0	5.0	0.0	5.0	SM	6	7	14	123		44	0.09	x		1.26	1.26	235
7	53.0	-53.0	10.0	0.0	10.0	ML	14	19	27	125		92	0.20	x		1.65	1.65	905
8	63.0	-63.0	10.0	0.0	10.0	ML	8	10	17	125		62	0.12	x		2.18	2.18	399
9	68.0	-68.0	5.0	0.0	5.0	SP-SM	36	42	43	130		8	NL	NL				
10	70.0	-70.0	2.0	0.0	2.0	SP	40	37	37	130		5	NL	NL				
														SUM:	0	12.62	12.62	

Liquefaction Analysis - Youd et. Al., (2001): Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils October, 2001.
Fines Correction - Seed and Idris formula (1997).
Fines Susceptibility - Boulanger and Idriss (2006) / fine grained soils with PI >= 7 exhibit clay-like behavior; if a soil plots as CL-ML, the PI criterion may be reduced to PI >= 5 and still be consistent with the available data.
According the the Caltrans Geotechnical Manual - Soils that should be considered potententially liquefiable are sands, low plasticity silts (PI<7), and, in unusual cases, gravel. Rock and most clay soil are not liquefiable.
Undrained Residual Shear Strength - Kramer, S. and Wang, C.H., (2015), "Empirical Model for Esimation of the Residual Strength of Liquefied Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE as cited in Caltrans (2017), Memo To Designers (MTD) 20-15, Lateral Spreading Analysis for New and Existing Bridges.
SPT N - Where applicable the Sampler Size Conversion to SPT N-value is in accoradance with Caltrans Geotechnical Manual March 2021.
Seismic Settlement of Dry Layers - Pradel, Daniel, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, April 1998, pages 364 - 368.
Seismic Settlement of Saturated Layers - Lee, C.Y., Asian Research Publishing Network Journal of Engineering and Applied Sciences, 2006-2007. This approach approximates the Volumetric Strain (ev) and is based on the Tokimatsu and Seed (1987) procedure.

Summary of Liquefaction Analysis

Boring: B4
South Pier
Hammond Trail Pedestrian Bridge

Project: **Hammond Trail Pedestrian Bridge**
File No.: **23-948.9**
Date: **04/18/24**
Boring: **B4**
Support: **South Pier**
By: **KBH**

Boring Elevation: 2.0 feet (at Channel Bottom)
Analysis Ground Water Depth = 0.0 feet
Analysis Ground Water Elevation = 2.0 feet
Hammer Energy (ER) = 80 %

Mw = 8.63
PGA = 1.03 g
Distance to Fault (R) = 9.2 miles
Liquefaction Factor of Safety (FS_L): 1.00

NA = Not Applicable - Soil layer above groundwater
NL = Non-Liquefiable
Note that soils with $(N1)_{60cs}$ > 30 are not considered susceptible to liquefaction irrespective of the other criteria or conditions.

Soil Layer	Depth to Bottom of Layer Below Ground Surface (feet)	Elevation to Bottom of Layer Below Ground Surface (feet)	Layer Thickness: Total Layer Thickness (feet)	Layer Thickness: Unsaturated Layer Thickness (feet)	Layer Thickness: Saturated Layer Thickness (feet)	Soil Type (USCS)	SPT N (blows/foot)	$(N1)_{60}$ (blows/foot)	$(N1)_{60cs}$ (blows/foot)	Soil Total Unit Weight (pcf)	Plasticity Index (PI)	Fines Content (%)	Factor of Safety Against Liquefaction FS_L	Potential Liquifiable Soil Layer FS_L < 1.00	Seismic Settlement: Dry Layers (inches)	Seismic Settlement: Saturated Layers (inches)	Seismic Settlement: Total (inches)	Undrained Residual Shear Strength Sr (psf)
1	18.0	-16.0	18.0	0.0	18.0	GP	4	7	7	125			0.05	x		6.66	6.66	110
2	28.0	-26.0	10.0	0.0	10.0	CL	9	16	24	125		65	NL	NL				
4	43.0	-41.0	15.0	0.0	15.0	ML	14	22	32	125		90	NL	NL				
5	53.0	-51.0	10.0	0.0	10.0	ML	15	21	30	125		71	NL	NL				
6	63.0	-61.0	10.0	0.0	10.0	ML	15	19	28	125		96	0.22	x		1.64	1.64	1044
7	70.0	-68.0	7.0	0.0	7.0	GM	43	50	53	130		12	NL	NL				
																SUM:			
														SUM:	0	8.3	8.30	

Liquefaction Analysis - Youd et. Al., (2001): Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils October, 2001.
Fines Correction - Seed and Idris formula (1997).
Fines Susceptibility - Boulanger and Idriss (2006) / fine grained soils with PI >= 7 exhibit clay-like behavior; if a soil plots as CL-ML, the PI criterion may be reduced to PI >= 5 and still be consistent with the available data.
According the the Caltrans Geotechnical Manual - Soils that should be considered potententially liquefiable are sands, low plasticity silts (PI<7), and, in unusual cases, gravel. Rock and most clay soil are not liquefiable.
Undrained Residual Shear Strength - Kramer, S. and Wang, C.H., (2015), "Empirical Model for Esimation of the Residual Strength of Liquefied Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE
as cited in Caltrans (2017), Memo To Designers (MTD) 20-15, Lateral Spreading Analysis for New and Existing Bridges.
SPT N - Where applicable the Sampler Size Conversion to SPT N-value is in accoradance with Caltrans Geotechnical Manual March 2021.
Seismic Settlement of Dry Layers - Pradel, Daniel, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, April 1998, pages 364 - 368.
Seismic Settlement of Saturated Layers - Lee, C.Y., Asian Research Publishing Network Journal of Engineering and Applied Sciences, 2006-2007. This approach approximates the Volumetric Strain (ev) and is based on the Tokimatsu and Seed (1987) procedure.

APPENDIX D

Preliminary Geotechnical Parameters

Table 1: Idealized Geotechnical Parameters – SHN B-1 (South Abutment)

Elevation (ft)	Soil Description	N_{60}	Unit Weight (lb/ft^3)	Static Soil Condition			Liquified Soil Condition		
				L-Pile Soil Type	Friction Angle (degrees)	p-y Modulus, k (lb/in^3)	L-Pile Soil Type	Cohesion (psf)	Strain Factor, E50 (dim.)
18 to 10.5	Silt with Sand (ML) Poorly-graded Sand (SP)	8	116	Sand (Reese)	30	40	--	--	--
10.5 to 3	Poorly-graded Sand and Silt (SP-SM)	12	116	Sand (Reese)	30	60	--	--	--
3 to -2	Poorly-graded Sand and Silt (SP-SM)	4	54	Sand (Reese)	28	20	Soft Clay (Matlock)[1]	250	0.02
-2 to -9.5	Well-graded Gravel with Sand (GW)	12	68	Sand (Reese)	30	30	Soft Clay (Matlock)[1]	250	0.02
-9.5 to -17	Silt (ML)	7	53	Sand (Reese)	29	20	--	--	--
-17 to -32	Silt (ML) and Sandy Silt (ML)	11	53	Sand (Reese)	30	25	Soft Clay (Matlock)[1]	350	0.02
-32 to -62	Silty Sand (SM) and Clayey Sand with Gravel (GC)	20	63	Sand (Reese)	32	50	Soft Clay (Matlock)[1]	650	0.01
-62 to -101.5	Well-graded Sand with Silt and Gravel (SW-SM), Well-graded Sand with Gravel (SW), and Poorly-graded Sand with Clay (SP-SC)	55	68	Sand (Reese)	37	125	--	--	--

Notes: Elevations are based on NAVD88.
Buoyant unit weight used/shown below design groundwater (elev. 3 feet).
For design scour consideration, no soil/rock support is available above the scour elevation.
[1] Model using residual shear strength value due to liquefaction.

Table 2: Idealized Geotechnical Parameters – SHN B-2 (North Abutment)

Elevation (ft)	Soil Description	N_{60}	Unit Weight (lb/ft^3)	Static Soil Condition			Liquified Soil Condition		
				L-Pile Soil Type	Friction Angle (degrees)	p-y Modulus, k (lb/in^3)	L-Pile Soil Type	Cohesion (psf)	Strain Factor, E50 (dim.)
18 to 8	Gravel FILL and Silty Sand (SM)	49	130	Sand (Reese)	38	225	--	--	--
8 to 3	Well-graded Sand (SW)	9	124	Sand (Reese)	30	40	--	--	--
3 to -17	Well-graded Sand (SW) and Well-graded Gravel (GW)	23	68	Sand (Reese)	33	60	Soft Clay (Matlock)[1]	250	0.02
-17 to -42	Lean Clay (CL), Silt (ML) and Silty Sand/Sandy Silt (SM/ML)	8	63	Sand (Reese)	28	20	Soft Clay (Matlock)[1]	225	0.02
-42 to -62	Poorly-graded Sand with Silt (SP-SM) and Silty Sand (SM)	31	63	Sand (Reese)	33	60	Soft Clay (Matlock)[1]	1900	0.007
-62 to -83.5	Well-graded Sand with Gravel (GW)	68	68	Sand (Reese)	40	125	--	--	--

Notes: Elevations are based on NAVD88.
Buoyant unit weight used/shown below design groundwater (elev. 3 feet).
For design scour consideration, no soil/rock support is available above the scour elevation.
[1] Model using residual shear strength value due to liquefaction.

Table 3: Idealized Geotechnical Parameters – SHN B-3 (North Channel)

Elevation (ft)	Soil Description	N_{60}	Unit Weight (lb/ft^3)	Static Soil Condition			Liquified Soil Condition		
				L-Pile Soil Type	Friction Angle (degrees)	p-y Modulus, k (lb/in^3)	L-Pile Soil Type	Cohesion (psf)	Strain Factor, E50 (dim.)
0 to -11	Poorly-graded Gravel (GP)	6	68	Sand (Reese)	29	20	Soft Clay (Matlock)[1]	100	0.02
-11 to -28	Silt (ML)	7	63	Sand (Reese)	29	20	Soft Clay (Matlock)[1]	150	0.02
-28 to -63	Silty Sand (SM), Sandy Silt (ML) and Silt (ML)	14	63	Sand (Reese)	30	40	Soft Clay (Matlock)[1]	300	0.02
-63 to -73	Poorly-graded Sand with Silt (SP-SM)	50	68	Sand (Reese)	38	125	--	--	--
-73 to -194	Silty Sand with Gravel (SM), Poorly-graded Gravel (GP), Poorly-graded Gravel with Silt Sand (GP-GM), Poorly-graded Gravel with Sand (GP), and Poorly-graded Sand (SP)	>70	68	Sand (Reese)	40	125	--	--	--

Notes: Elevations are based on NAVD88.
Buoyant unit weight used/shown below design groundwater (elev. 3 feet).
For design scour consideration, no soil/rock support is available above the scour elevation.
[1] - Model using residual shear strength value due to liquefaction.

Table 4: Idealized Geotechnical Parameters – SHN B-4 (South Channel)

Elevation (ft)	Soil Description	N_{60}	Unit Weight (lb/ft^3)	Static Soil Condition			Liquified Soil Condition		
				L-Pile Soil Type	Friction Angle (degrees)	p-y Modulus, k (lb/in^3)	L-Pile Soil Type	Cohesion (psf)	Strain Factor, E50 (dim.)
2 to -16	Poorly-graded Gravel (GP)	5	68	Sand (Reese)	29	20	Soft Clay (Matlock)[1]	100	0.02
-16 to -51	Lean Clay with Gravel (CL), Elastic Silt with Sand (MH), Silt (ML)	17	63	Sand (Reese)	33	40	--	--	--
-51 to -61	Silt (ML)	20	63	Sand (Reese)	32	50	Soft Clay (Matlock)[1]	950	0.01
-61 to -201	Silty Gravel (GM), Silty Gravel with Sand (GM), Well-graded Gravel with Sand (GW), Poorly-graded Gravel (GP), Well-graded Sand with Gravel (SW), and Silty Sand with Gravel (SM)	>70	68	Sand (Reese)	40	125	--	--	--

Notes: Elevations are based on NAVD88.
Buoyant unit weight used/shown below design groundwater (elev. 3 feet).
For design scour consideration, no soil/rock support is available above the scour elevation.
[1] - Model as soft clay using residual shear strength value due to liquefaction.

APPENDIX E

Preliminary Axial Capacity – 72-inch CIDH Pier (Northern Channel)

PRELIMINARY AXIAL CAPACITY ANALYSIS OF 72-INCH CIDH PILES

At the request of Mark Thomas, preliminary axial analysis of 72-inch cast-in-drilled-hole (CIDH) pile foundations was completed for the proposed pier supports within the channel. The analysis was based on data obtained in SHN's northern channel boring (B-3). Refer to Section 12 of the report for additional discussion of this alternative. The loading data provided by Mark Thomas and the estimated tip elevations for the strength and extreme loading cases are summarized below in Table 1.

Table1: Preliminary Pier Foundation Design Recommendations

Support Location	Pile Type	Cut-off Elev. (ft)	Service-I Limit State Load Per Support (kips)	Nominal Resistance (kips)					Design Tip Elev. (ft)	Prelim. Specified Tip Elev. (ft)
				Strength/Const.		Extreme				
				Comp. $\varphi = 0.7$	Tens. $\varphi = 0.7$	Comp. $\varphi = 1.0$	Comp. with D.D.	Tens. $\varphi = 1.0$		
North Channel	72" CIDH	0	900	1,720	N/A	780	2,510	N/A	-69 (a-I) -98 (a-II) -153 (a-II)	-153

Notes:
1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (a-III) Compression (Extreme Event – Downdrag).
2) The piles will be embedded adequately into dense soil layers, and the pile design accounts for downdrag loads in the Extreme Event; therefore, a detailed assessment of the pile group settlement is not considered warranted.

Crawford did not complete a lateral pile analysis. Appendix D contains preliminary geotechnical and L-Pile program parameters for others to use to complete lateral analysis, as needed.

This analysis will be superseded by future analysis completed for the Foundation Report and based on additional field data (per Section 12.3).

COMPRESSIVE RESISTANCE

The side (compressive) resistance for the CIDH pile foundations for the 72-inch CIDH pile foundations was evaluated using Load and Resistance Factor Design (LRFD) method and factors from AASHTO LRFD Bridge Design Specifications (BDS), 8th Edition, with current Caltrans amendments. The computer program Shaft v.2017.8.11 developed by Ensoft, Inc. was used to determine the side (compressive) resistance for the proposed piles. No significant long-term pile settlement is anticipated at the site; however, negative skin friction due to liquefaction is anticipated and accounted for in the pile analysis.

The bottom length of pile equivalent to the shaft diameter is excluded from contributing to geotechnical capacity. Tip resistance in axial compression was neglected in consideration of slurry installation method, consistent with current Caltrans guidelines for CIDH pile design.

To determine the required nominal resistance of the CIDH piles for the controlling limit state we used the preliminary foundation data provided by Mark Thomas and compared the factored strength limit and factored extreme limit loads by dividing each by their respective geotechnical resistance factor for side resistance (φ_{qs} = 0.7 for strength limit; φ_{qs} = 1.0 for extreme limit) consistent with current AASHTO LRFD BDS and California amendments.

The Shaft program outputs with Nominal Resistance are attached.

TENSION (UPLIFT) RESISTANCE

No tension demands are indicated by Mark Thomas based on preliminary data.

NEGATIVE SKIN FRICTION

Drag load (negative skin friction) develops along the pile shaft from excessive soil settlement which occurs after installation. For bridge structures, “static” negative skin friction is typically associated with long-term consolidation settlement of soft ground due to approach fill loading, and therefore typically affects abutment piles only. Seismically-induced negative skin friction is generally associated with liquefaction-induced settlement of soil along pile shafts. Only the seismically-induced negative skin friction is present at this site.

Liquefiable soil layers are present at this site and liquefaction induced ground settlement could occur shortly following the design earthquake event. As indicated in Section 11.4.3 of this report, liquefaction induced settlements on the order of 8.3 to 12.6 inches may occur at the site shortly following the design earthquake event. The preliminary analysis for the 72-inch CIDH pile considers downdrag and follows the procedures outlined in “Liquefaction-Induced Downdrag” (Caltrans, January 2020). Based on the results of the downdrag analysis, the location of the maximum downdrag load (Line AA’) is estimated at elevation 59 feet. The nominal downdrag loads due to liquefaction-induced settlement from the soil layers located above line AA’ were calculated based on the shear strengths of the resettled liquefied soil. The resulting maximum downdrag loads (DD_{max}) are estimated to be 1,410 kips.

Downdrag calculation results are attached.

SETTLEMENT

Significant long-term (consolidation) settlement is not anticipated at this site.

Liquefaction-Induced Downdrag Analysis

Hammond Trail Pedestrian Bridge	File # 23-948.9
7/18/24	By KBH
North Channel (B-3)	Checked KL/WEN

Reference: Caltrans Geotechnical Manual - Liquefaction-Induced Downdrag (January 2020)

Evaluate Potential for Surface Manifestation of Liquefaction

Layers 1-8 — From elev. 0 (ground surface) to -63 ft is considered liquifiable. While layer 4 has low potential for liquefaction (N160cs =31) using an assumed ETR of 80%, we conservatively estimate that the lower layers would manifest through this thin (<6') layer with 33' of liquefiable soils above it.

Determine Pile DTE for Compression

All pile nominal resistance from the bottom elev. of liquefied soil layer and the overlying soil layers are ignored (lowest elevation = -63 ft)

Determine Pile Design Tip Elevation (DTE) for Extreme Event-I (Compression)

TOP LENGTH REMOVED FROM SHAFT FOR SCOUR: 15

Pile Cut-off Elev. (ft) =	0	*** Assumed cut-off at GSE*
Groundwater Elev. (ft) =	0	
Top Elevation of Liquefied Soil Layer (ft) =	0	
Bottom Elevation of Liquefied Soil Layer (ft) =	-63	***includes top 15 ft not modeled in SHAFT*
Zero-Friction Length Below Cut-off (ft) =	63	= Pile E30Cut-off Elev. - Bottom Elev. of Liquefied Soil Layer
Prelim Pile Length (ft) =	84.5	***includes top 15 ft not modeled in SHAFT*
DTE for Extreme Event-I (Compression) (ft) =	-85	= Pile Cut-off Elev. - Prelim Pile Length
Factored Design Load (kips) =	780	
Unit Weight of Pile (lb per cubic foot) =	150	
Effective Weight of Pile (kips) =	209	
Revised Factored Design Load (kips) =	989	= Factored Design Load + Effective Weight of Pile
Pile Diameter (ft) =	6	Area = 28.274 ft^2
Revised Pile Tip Elevation (ft) =	-93	*including effective weight of pile (w/ top 15 ft)* — Specified Tip Elevation (Extreme-I Limit) (ft) = -98
Embedment Depth into Bearing Strata (ft) =	30	= Bottom Elev. of Liquefiable Soil Layer - Revised Pile Tip Elev.
Ratio of Pile Embedment Depth =	5	= Embedment Depth into Bearing Strata/Pile Diameter (if greater than 1.5, no pile tip reduction)

Select the Preliminary Pile Tip Elevation for Downdrag Load Analysis

Controlling Limit State =	Extreme-I	
Design Tip Elevation (ft) =	-98	
Pile Length (ft) =	98	= Pile Cut-off Elev. - Design Tip Elev.

Calculate Pile Settlements at the Onset of Liquefaction-Induced Ground Settlement

Max Permanent Load per Pile (kips) =	730	
round up to nearest 10 kips =	730	
Estimated Pile Top Settlement (inches) =	0.071	Estimated from Pile Settlement vs
Estimated Pile Tip Settlement (inches) =	0.033	Axial Load Graph in Shaft

Calculate Liquefaction-Induced Ground Settlements

From B-3 liq spreadsheet, Lique. Settlement (in) =	12.62	*Includes settlement of scourable soil*

Determine Location of the Maximum Downdrag Load (DDmax)

Zmax/D =	0.01	
zmax (in) =	0.72	
Pile/Ground Settlement at point 0, δo (in) =	0.050	
Critical Ground Settlement (δc-Liq) (in) =	0.770	
AA' Line elevation (ft) =	59	*Does not include 15' of pile at top*

Calculate Maximum Downdrag Load (Ddmax)

Side Resistance from Cut-off to AA' Line (kips) =	1406	Estimated from Accumulated Skin Friction Graph in SHAFT
round up to nearest ten kips =	1410	*Includes top 15' of soil*

Calculate Pile Nominal Resistance in Compression below AA' Line

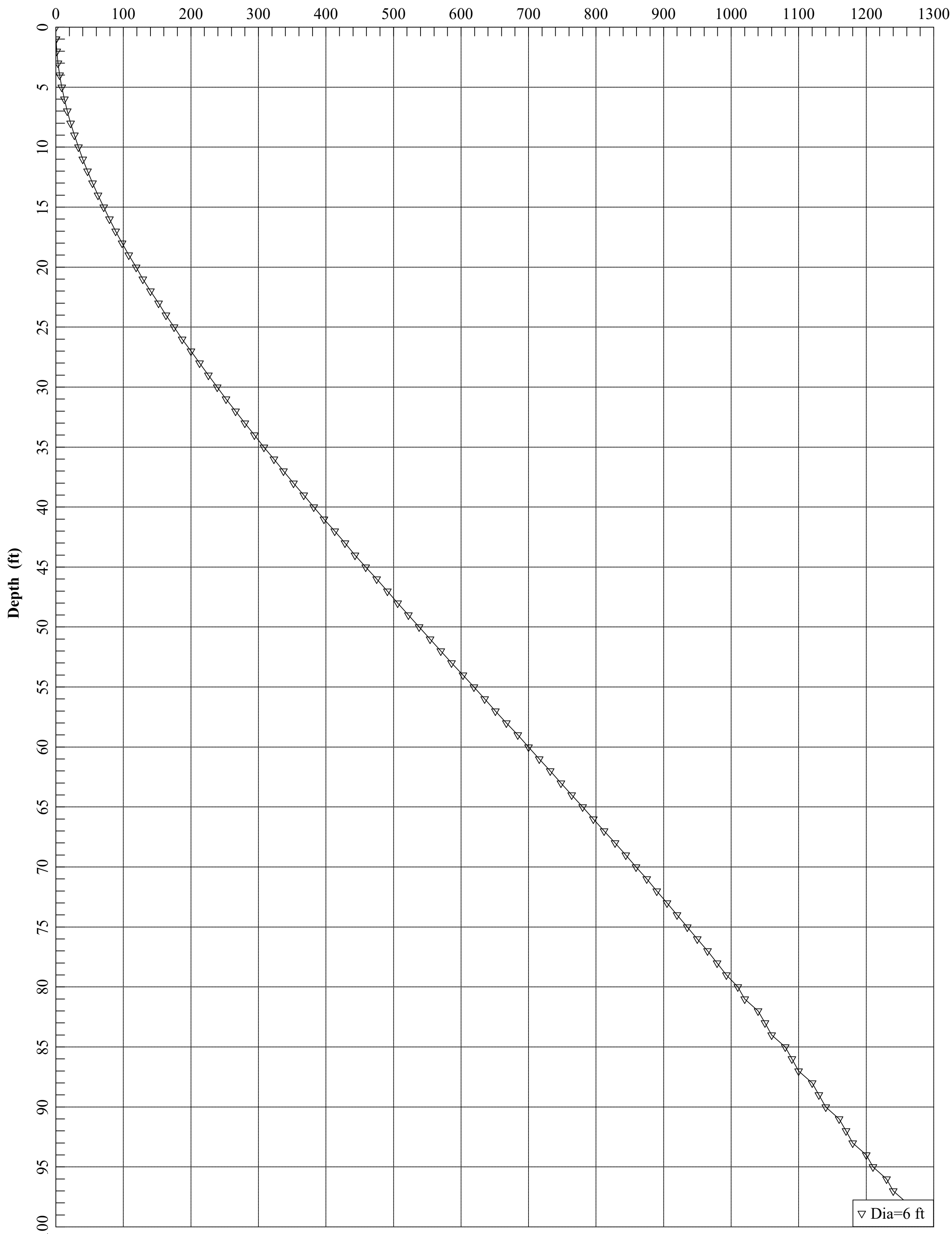
Calculated using SHAFT. See graph output.

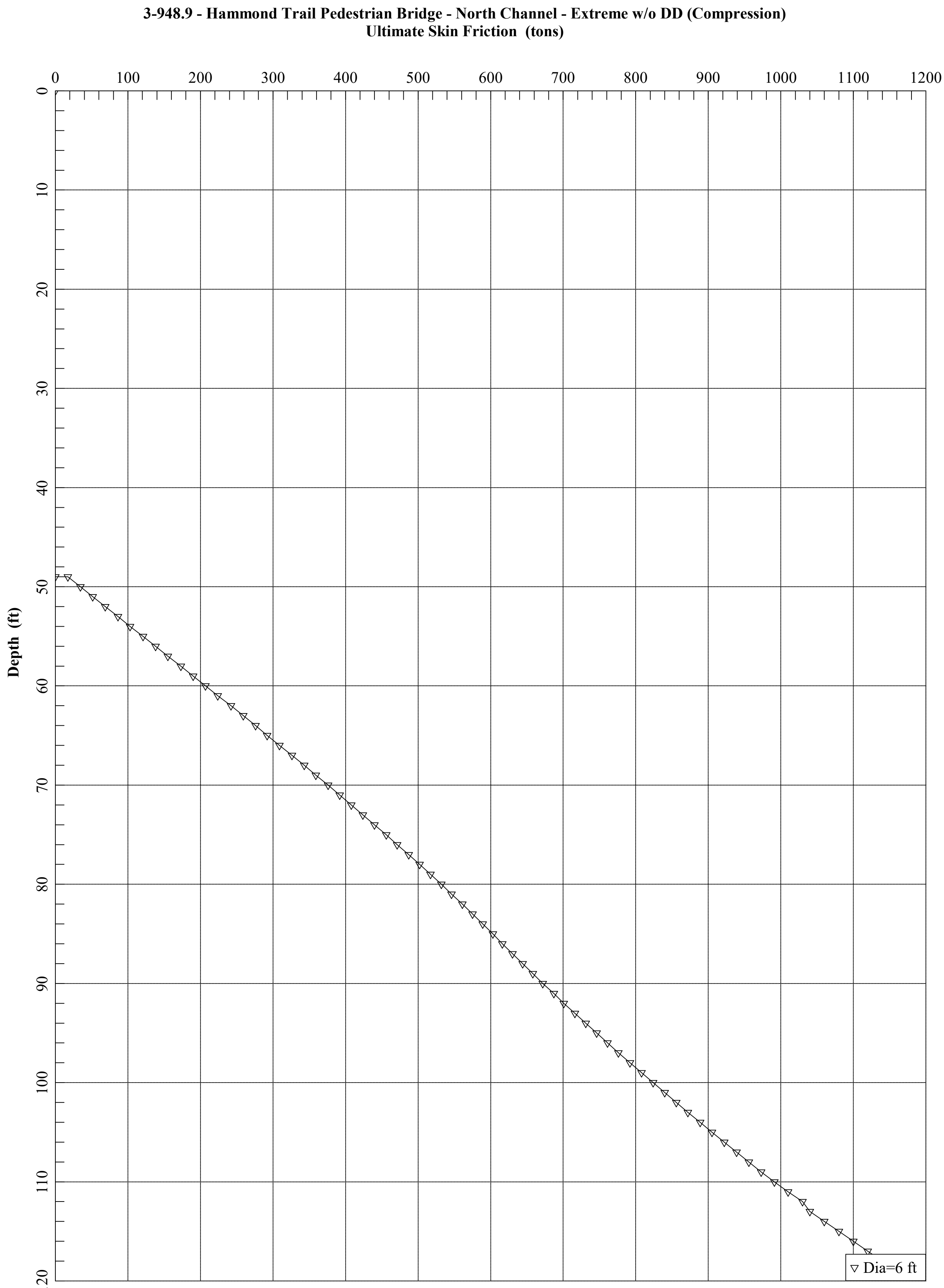
Determin Pile DTE for Compression (Extreme Event-I Downdrag)

Factored Total Seismic Design Load per Pile = (Factored Max. Permanent Load per Pile) + (Factored DDmax per Pile) + (Factored Effective Pile Weight of the Pile)

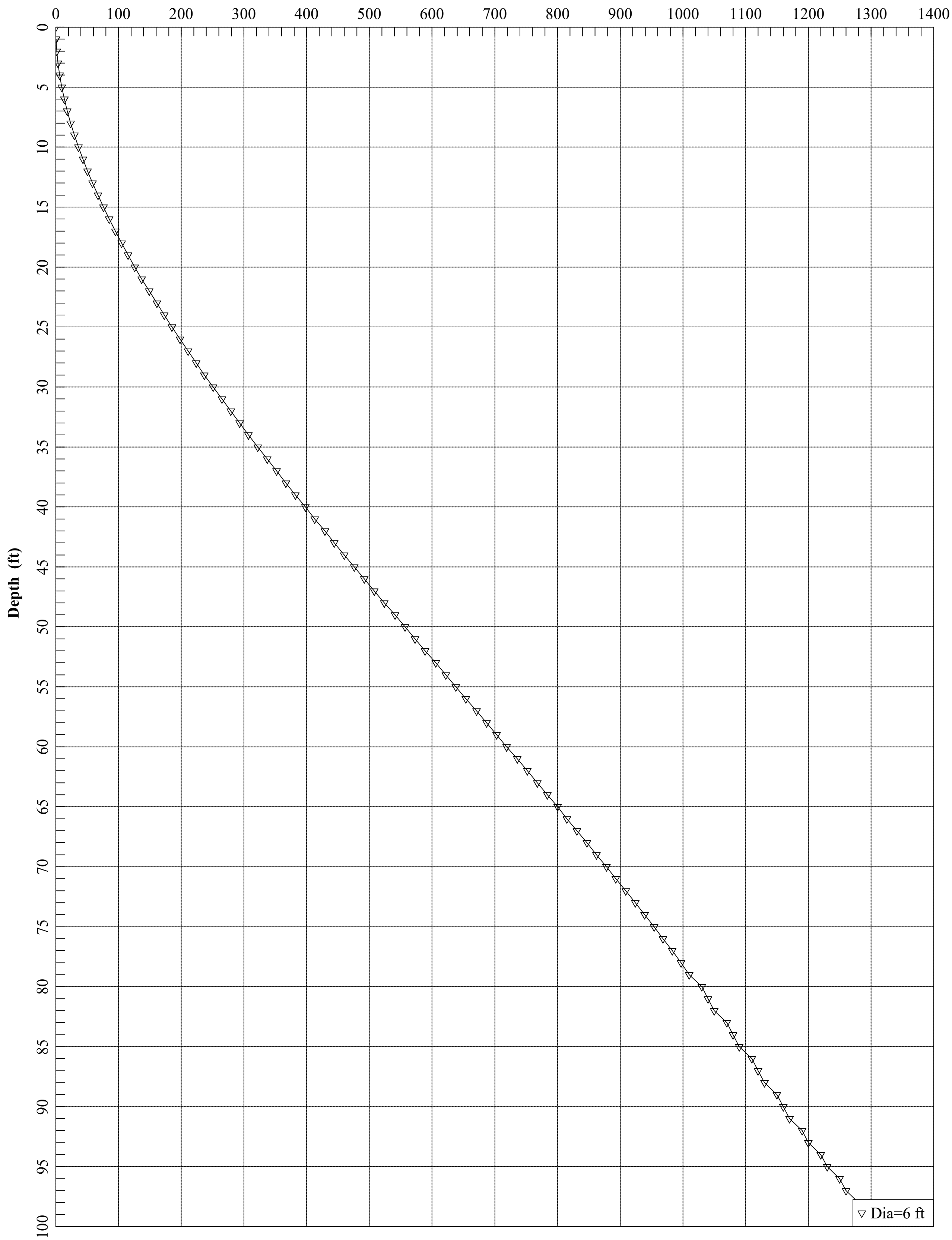
Factored Max. Permanent Load per Pile (kips) =	730	
Factored DDmax per Pile (kips) =	1406	
Factored Design Load without Pile Weight (kips) =	2136	
Prelim Pile Length (ft) =	139.5	Prelim Pile Elevation (ft) = -139.5 — *Includes top 15' of pile*
Total Effective Pile Weigth (kips) =	346	
Factored Total Seismic Design Load per Pile =	2482	= Factored Design Load without Pile Weigth + Total Effective Pile Weight
round up to nearest kip =	2490	
Revised Pile Length (ft) =	148	Revised Pile Elevation (ft) -148
Total Effective Pile Weigth (kips) =	367	
Factored Total Seismic Design Load per Pile =	2503	
round up to ten kips =	2510	
Available Factored Side Resistance (kips) =	2490	
Available Factored Tip Resistance (kips) =	1382	
5% of Available Factored Tip Resistance (kips) =	69	
Available Total Factored Resistance (kips) =	2559	
Design Tip Elevation (ft) =	-148	Specified Tip Elevation (Extreme-I Downdrag) (ft) = -153

3-948.9 - Hammond Trail Pedestrian Bridge - North Channel - Strength (Compression)
Ultimate Skin Friction (tons)

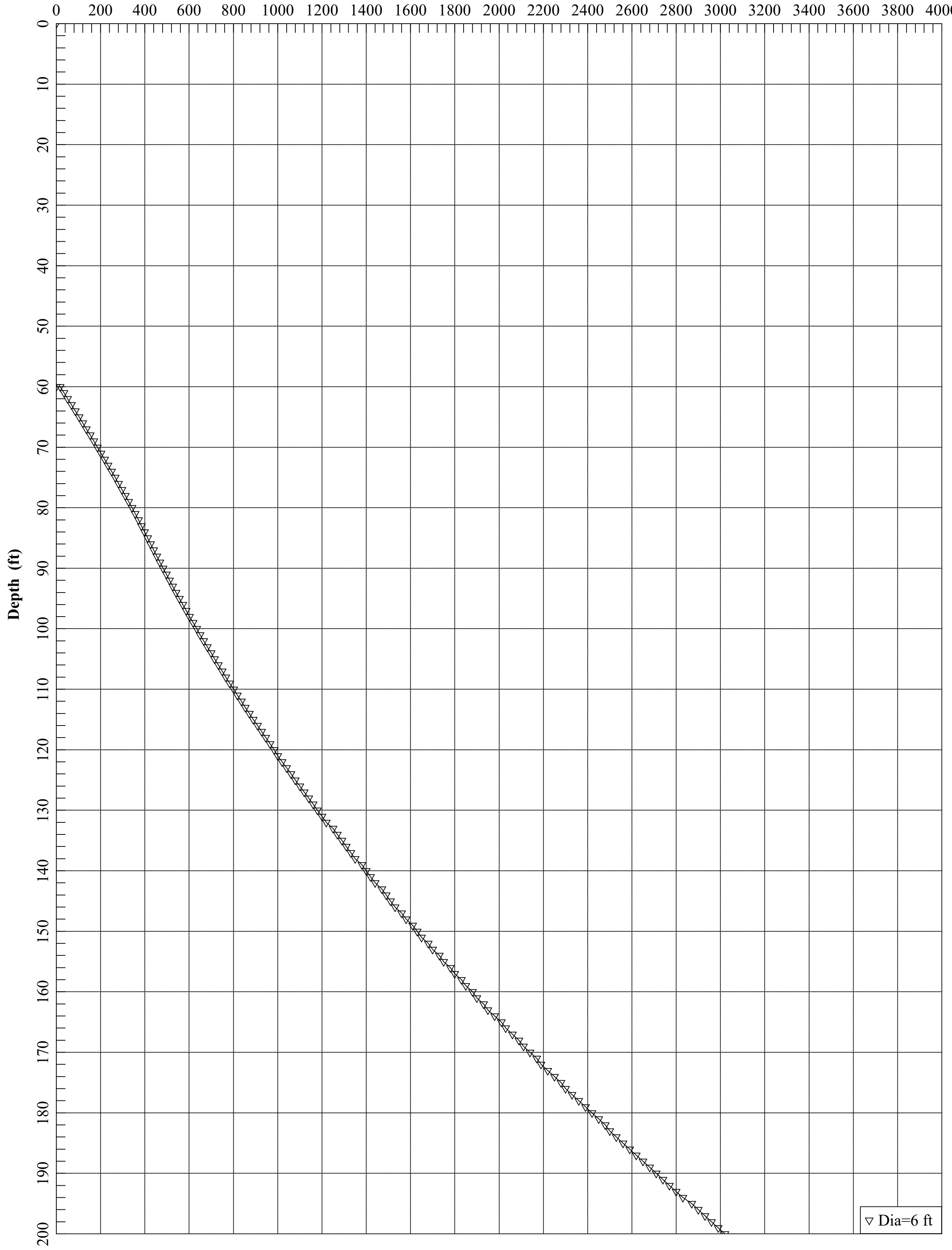



3-948.9 - Hammond Trail Pedestrian Bridge - North Channel - Extreme w/o DD (Compression)
Ultimate Skin Friction (tons)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Dia=6 ft

3-948.9 - Hammond Trail Pedestrian Bridge - North Channel - DDMax (Compression)
Ultimate Skin Friction (tons)



3-948.9 - Hammond Trail Pedestrian Bridge - North Channel - Extreme w/ DD (Compression)
Ultimate Skin Friction (tons)



ATTACHMENT D

Preliminary Hydraulics Report

TECHNICAL REPORT ◦ NOVEMBER 2024

Hammond Trail Bridge Replacement: Preliminary Hydraulic Report

PREPARED FOR

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Stillwater Sciences

Suggested citation:
Stillwater Sciences. 2024. Hammond Trail Bridge Replacement: Hydraulic Studies to Determine 100-year Water Surface Elevations. Prepared by Stillwater Sciences, Arcata, California for Mark Thomas, San Jose, California.

Cover photos: Hammond Trail Bridge over the Mad River, Humboldt County, California, October 2024, photo taken by Ian Pryor, Stillwater Sciences.

I hereby certify that all work described in this report follows accepted engineering practices and was completed under my direction. Future use of the information presented herein should consider the limitations of this analyses including inherent uncertainties associated with sediment transport modeling results that provided input data for future conditions hydraulic modeling and the coarse nature of the hydraulic modeling approach.

11/7/2024

Joel Monschke P.E. Date
Civil Engineer
Stillwater Sciences

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Figure 1. Study area map showing the extent of hydraulic modeling and locations of the Hammond Trail Bridge, USGS gaging station, and public boat launch 2

Appendices

Appendix A. Floodplain Maps Showing Average Expected Inundation and Water Surface Elevations During 100-year Storm Event for the FEMA Dynamic Water Level Scenario
Appendix B. Hammond Trail Bridge Cross-section Showing Modeled 1% AEP Water Surface Elevations

1 INTRODUCTION

This study summarizes hydraulic modeling conducted by Stillwater Sciences to support design alternatives for replacement of the Hammond Trail Bridge over the Mad River near Arcata, California. This report provides a summary of anticipated 100-year, or 1% annual exceedance probability (AEP) flood water surface elevations (WSEs) for the Study Area based on hydraulic model simulations. The Study area encompasses the lower Mad River and its floodplain, extending downstream from Highway 101 bridge crossing downstream to near the county boat ramp in the Mad River estuary and southwestward toward Arcata Bay (Figure 1). Most of the Study area is within Federal Emergency Management Authority (FEMA) flood Zone A which typically indicates detailed flood analyses have not been performed and 100-year WSEs are not available. This hydraulics report will be included with the Structure Type Selection Report being prepared by Mark Thomas for the Humboldt County Department of Public Works.

The Mad River drains approximately 500 square miles of the northern California Coast Range in Trinity and Humboldt counties. The river follows a predominately northwesterly course from its headwaters at an elevation of 5,300 feet to sea level where it drains into the Pacific Ocean near the community of McKinleyville, California. The Hammond Trail Bridge is located approximately 3.75 miles upstream of the present-day Mad River mouth and connects the northern and southern sections of the Hammond Trail. Originally constructed as a railroad bridge for timber operations in 1941, the 540-foot-long structure has served as a public pedestrian and bicycle crossing since 1983 (CH2MHILL, 1998).

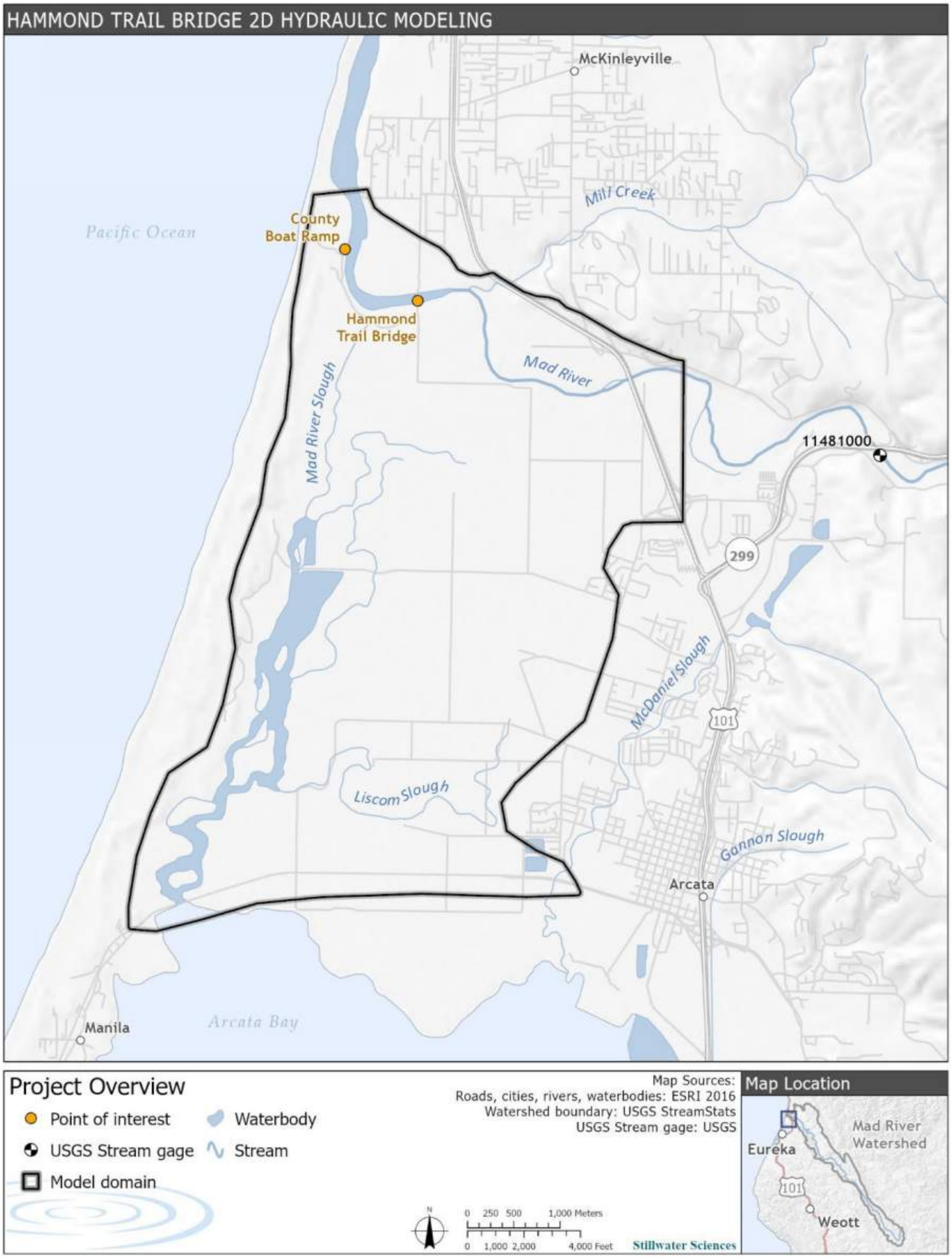

Figure 1. Study area map showing the extent of hydraulic modeling and locations of the Hammond Trail Bridge, USGS gaging station, and public boat launch.

2 EXISTING CONDITIONS HYDRAULIC MODELING

To better understand flow dynamics in the Study Area, hydraulics were modeled in the US Army Corps of Engineers' Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 6.1.0 (2023). HEC-RAS describes the physical properties of streams and rivers by performing two-dimensional (2-D) hydrodynamic routing with unsteady flow. HEC-RAS utilizes a user-defined computational mesh to represent the terrain data. The mesh is composed of 3- to 8-sided elements built via breaklines, with prescribed node spacing and important grade breaks identified in the terrain. Tighter node spacing yields smaller elements (and thus more nodes within the mesh) which can better represent complex terrain or hydraulically sensitive features.

2.1 Digital Terrain Model Development

The hydraulic modeling is based on a Digital Terrain Model (DTM) that combines aerial Light Detection and Ranging (LiDAR) point cloud data collected for the City of Eureka in 2019 (OCM Partners 2019) and survey data collected in 2007 (Stillwater Sciences 2008) and 2024 (this study). The primary objectives of the field survey were to fill in LiDAR data gaps which occur in the portion of the channel that was wetted during LiDAR acquisition, within areas of dense vegetation, and under bridges. Additional survey shots were collected on stable paved surfaces to check for vertical bias of the LiDAR. The final DTM was generated to capture existing conditions topographic and bathymetric detail to ensure adequate channel and floodplain flow conveyance capacity for 100-year flood modeling and may be unsuitable for purposes beyond those limitations.

Stillwater Sciences staff conducted topographic and bathymetric surveys between June–September 2024 utilizing robotic total station (RTS), real-time kinematic Global Navigation Satellite System (RTK GNSS), and single beam sonar survey equipment. The RTK GNSS was used to establish temporary survey control networks for RTS surveys underneath the Highway 101 and Hammond Trail Bridge locations and for topographic fill-in along shallow channel margins and banks throughout the survey reach. Deeper channel bathymetry data was collected with a survey grade single beam echo sounder integrated with RTK GNSS and mounted to an inflatable survey vessel. Field surveys focused on filling in channel bathymetry from immediately upstream of the Highway 101 bridge to approximately 1,300 feet downstream of the Hammond Trail Bridge near the upstream extent of 2007 surveys (Stillwater Sciences 2008). Top of bank, channel toe, and grade break points were opportunistically captured within the survey reach to generate breaklines that help maintain surface continuity and control elevation and slope breaks in the surface model. A channel longitudinal profile was surveyed from approximately 1,300 feet downstream of Hammond Trail Bridge to the Mad River boat launch to help validate channel bed elevations from the 2007 surveys.

The 2024 survey data was processed using an RTK GNSS base station position correction from the National Geodetic Survey (NGS) Online Positioning User Service (OPUS) referenced to North American Datum of 1983 (NAD83) at epoch 2010.00 horizontal datum and North American Vertical Datum of 1988 (NAVD88). GNSS ellipsoid heights were converted to orthometric elevations using GEOID18 hybrid geoid model. Project coordinates are reported in California State Plane Zone II, US survey feet units.

Additional RTK GNSS points were collected on paved surfaces at various locations near the Mad River boat launch and along Mad River Road for LiDAR vertical bias correction. A vertical shift (+0.2 feet) for the 2019 LiDAR data was determined by comparing the survey elevations collected along paved surfaces to the point cloud in GeoCue LP360 software. The adjusted LiDAR point cloud was integrated with the 2024 and 2007 topographic and bathymetric surveys. The final DTM was exported to a Digital Elevation Model (DEM) with 3-foot cell size spacing and imported into HEC-RAS RAS mapper.

2.2 Existing Infrastructure

The Hammond Trail Bridge superstructure is represented in the model 2D mesh based on the 2019 LiDAR data as described in Section 3.1. The shape and dimensions of the bridge abutments and piers are represented in the model as an internal connection bridge structure. The bridge deck's low and high cord are represented in the model. FEMA 100-year discharge resulted in WSEs approximated 6 ft below the middle low chord elevation, so pressure flow and weir overtopping were not evaluated during these simulations.

2.3 Hydrology

The USGS stream gage on the Mad River (No. 11481000) recorded flows intermittently from 1911 to 1913 and continuously from 1951 to the present. Positioned approximately 12 miles upstream of the river's mouth and about 4 miles upstream of the Hammond Trail Bridge, this gage provides valuable long-term data for hydrologic analysis. Recorded annual peak discharges range from 3,360 cubic feet per second (cfs) on March 7, 1977, to a maximum of 81,000 cfs during a historic flood event on December 22, 1964.

Hydrologic analysis was conducted to generate essential streamflow inputs for the hydraulic model. By examining peak streamflow and mean daily flow records, flood frequency estimates were calculated to inform potential flood risks.

2.3.1 Peak flows

Peak flow estimates from a flood frequency analysis have specific recurrence intervals, or frequencies (e.g., a 100-year peak flow has a 1% chance of occurring any year, or once in 100 years, on average). A flood frequency analysis (FFA) was performed on annual peaks recorded at USGS gage 11481000 in accordance with USGS Bulletin 17C (USGS 2019) using the Hydrologic Engineering Center's statistical software package (HEC-SSP) (USACE 2019) for Water Years 1911-1913, and 1951-2022 (76-year period of record). Previous hydrologic analysis conducted for the Humboldt County Department of Public Works reported a regional skew coefficient of -0.425 with a mean square error of 0.134 for USGS gage 11481000 (Northern Hydrology & Engineering, 2013). These values were adopted for the FFA analysis and combined with station skew to apply a weighted regional skew for the FFA analysis.

Additional peak flow data were sourced from the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for the Mad River (FEMA, 2018) and published USGS peak-flood estimates (Gotvald and others, 2012), as shown in Table 1. Peak flow estimates from this study and the FEMA FIS are generally lower than the USGS results. The FEMA estimates were developed using Bulletin 17B methods based on peak flow records from the USGS Mad River gage near Arcata, representing a 64-year period of annual peak discharges.

The difference in peak discharge estimates between the USGS analysis and FEMA's FIS values likely arises from methodological differences, including FEMA's use of Bulletin 17B guidelines and regional scaling by drainage area. This approach contrasts with the site-specific hydrologic data and Bulletin 17C guidelines used in the USGS analysis. Differences between the published USGS estimates and peak flood estimates computed for this study likely arise from differences in the period of record analyzed, adopted regional skew coefficients, and perception thresholds applied to the periods of missing record. The USGS study utilized data through Water Year 2006, a 56-year period of record whereas the computed peak flood estimates for this study utilized the entire 76-year record. Skew values and perception thresholds are not reported for USGS gage 11481000.

Table 1. Mad River peak flows at USGS gage.

Return period (year)	Peak flood flows (cfs)		
	FEMA FIS	**USGS**	**This Study**
2		36,400	18,112
5		54,800	33,229
10	58,360	66,700	44,532
20		81,200	56,019
50	81,270	91,700	71,590
100	90,960	102,000	83,677

Based on available data and model objectives, the HEC-RAS model utilized peak flows from both the FEMA and USGS estimates. The inclusion of both datasets allowed for comparative analysis of flood risk scenarios, providing a conservative estimate of water surface elevations across recurrence intervals.

2.4 Model Setup

A terrain mesh was created for the Study Area to characterize main channel and off-channel (overbank) geometries by importing a digital elevation model (DEM) into HEC-RAS derived from the DTM described in Section *2.1 Digital Terrain Model development*. Mesh cells were set to a consistent 10 feet throughout the model domain, with breakline refinement to align mesh elements. The terrain surface and corresponding mesh elevations were compared to ensure the terrain was captured as accurately as possible.

2.4.1 Channel roughness

The channel bed and floodplain surface roughness characteristics for the model were defined in HEC-RAS with spatially discrete areas, which are a plan view of Manning's roughness coefficients for the model domain. Manning's roughness coefficients were assigned based on standard references (Chow 1959), field observations, and aerial imagery (Table 2).

Table 2. Manning's roughness coefficients.

Land cover	Manning's roughness coefficient
Barren land	0.03
Open water cobbles	0.045
Gravel bars	0.065
Channel	0.032
Channel banks	0.055
Overbank	0.07
Mixed forest	0.07
Floodplain	0.05–0.08

2.4.2 Boundary and initial conditions

Boundary conditions define model behavior at the limits of the model domain, providing essential inputs for computations. They differ between the upstream and downstream boundaries to account for site-specific hydraulic influences. To minimize boundary effects in the HEC-RAS 2D model, the upstream and downstream boundary conditions were extended beyond the limits of the available survey and DEM data through extrapolation, ensuring smoother flow transitions and reducing potential inaccuracies at the model extents.

The downstream boundary for the Mad River was set using a WSE to represent tidal and backwater effects. At the downstream extent of the model domain, the Pacific Ocean boundary was assigned a static WSE of 7.63 feet, reflecting the Mean Higher-High Water (MHHW) level from the National Oceanic and Atmospheric Administration (NOAA) Station 9418865 in Arcata, CA (NOAA 2024). This static approach models typical high-water levels without incorporating tidal fluctuations.

To assess a conservative scenario, the FEMA dynamic water level (DWL) was also applied, using a WSE of 18.4 feet to represent potential storm surge or extreme tidal influences. FEMA's DWL accounts for variable downstream water elevations impacted by tides, storm surges, and transient riverine or coastal events. This boundary condition models the interaction of upstream flows with downstream tidal fluctuations, providing a precautionary perspective on flood hazards when water levels are affected by such dynamic factors.

For the Arcata Bay boundary condition, two scenarios were modeled. The first used a normal depth boundary condition with a slope of 0.002, applied to the MHHW scenario. For the DWL scenario, a FEMA-provided WSE of 13.4 feet was assigned using a stage hydrograph boundary condition. This approach ensured a comprehensive assessment of flood behavior under both likely and extreme conditions.

2.4.3 Simulation

Computational time-steps were set to Courant controlled and ranged from 0.06 to 2 seconds. After several initial runs, the computational mesh was refined to enhance resolution and strategically orient cell centers, reducing fragmentation of inundation areas and improving model continuity. In HEC-RAS 2D, floodplain mapping relies on the detailed terrain model, so wetted areas are defined by site-specific topography rather than by the size of the computational mesh cells. As a result, cells are accurately represented as partially wet or dry based on the terrain data, and mapping outputs reflect nuanced topographic details rather than binary wet-dry states of the computational cells.

2.5 Existing Conditions HEC-RAS Modeling

Two-dimensional hydraulic modeling was conducted based on the input data described above.

2.5.1 Model calibration and validation

Model calibration was based on observed high-water marks and corresponding gage discharge from the water year (WY) 2024. Manning's roughness coefficients were adjusted to ensure that modeled design flows aligned with observed conditions. Sensitivity analysis indicated that within reasonable roughness ranges, adjustments to Manning's roughness coefficients had minimal impact on key hydraulic parameters, demonstrating model stability.

The Mad River flood of record at the gage occurred on December 22nd, 1964, with a measured peak discharge of 81,000 cfs and an estimated recurrence interval of 50-years. The USACE collected and tabulated elevations of high-water marks from the 1964 flood on the Mad River. The calibration results shown in Table 3 demonstrate a close alignment between observed and simulated water surface elevations, with differences of less than a tenth of a foot, indicating a high level of accuracy in the model's ability to replicate historical flood conditions for the December 22, 1964 event. The locations and elevations of some of these marks relevant to this study are shown on Table 3.

Table 3. December 22, 1964, event calibration results.

Observation point	Observed WSE (ft)	Simulated WSE (ft)	Difference (ft)
On south bank of Mad River, downstream of U.S. Highway 101 bridge, near Canal School	25.62	25.56	0.06
On pier of downstream face of southbound U.S. Highway 101 bridge	32.67	32.71	-0.04

Observations of the May 5th, 2024, storm event by Stillwater staff provided additional calibration data. During this, 53,900 cfs, approximately 5- to 10-year, storm event, staff documented high-flow debris lines shortly after peak flows. These observed high-water marks aligned closely with the simulated extent and elevation of inundation during a modeled discharge, validating model accuracy to within tenths of a foot of observed water surface elevations (Table 4).

Table 4. May 5, 2024, event calibration results.

Observation point	Observed WSE (ft)	Simulated WSE (ft)	Difference (ft)
A	15.297	15.713	-0.416
B	16.525	16.384	0.131
C	16.518	16.295	0.174
D	16.618	16.396	-0.363
E	31.000	30.770	0.230

2.5.2 Model scenarios

Following the calibration runs, modeling was conducted for peak flow scenarios as described in Table 5 to assess anticipated WSEs at the Hammond Trail bridge. MHHW and DWL downstream boundary conditions were analyzed to understand tidal impacts on the WSE at Hammond Trail Bridge. The elevations in Table 5 reference the North American Vertical Datum of 1988 (NAVD 88).

Table 5. 100-year WSE at Hammond Trail Bridge.

Scenario	Discharge (cfs)	Mad river stage at downstream boundary condition (ft)	Arcata Bay boundary condition	WSE at Hammond Trail Bridge (ft)
Flood of Record	81,000	7.63	Normal Depth	19.84
FEMA 50-yr	81,270	7.63	Normal Depth	19.85
FEMA 100-yr (MHHW)	90,960	7.63	Normal Depth	19.94
FEMA 100-yr (DWL)	90,960	18.4	DWL	20.18
USGS 100-yr (MHHW)	101,000	7.63	Normal Depth	20.02
USGS 100-yr (DWL)	101,000	18.4	DWL	20.25

2.5.3 Freeboard

The hydraulic design of the bridge should follow the California Department of Transportation's (Caltrans) criteria (Caltrans 2020). The bridge freeboard requirements applicable to the Study Area are based on the Caltrans criterion for the hydraulic design of bridges is that they be designed to pass the 2-percent-probability-of-annual-exceedance flow (50-year design discharge) or the flood of record, whichever is greater, with adequate freeboard to pass anticipated debris. Two feet of freeboard is commonly used in bridge designs. The bridge should also be designed to pass the 1-percent-probability-of-annual-exceedance flow (100-year design discharge, or base flood). No freeboard is added to the base flood.

In summary, the Hammond Trail Bridge must maintain a minimum clearance above the 100-year WSE, 20.25 feet, with no additional freeboard requirements, as per Caltrans guidelines.. However, it is recommended to consider additional freeboard to accommodate floating debris, as well as to account for potential impacts from sea-level rise and climate change, which may increase the frequency and severity of storm events.

3 HYDRAULIC MODEL RESULTS

Two-dimensional hydraulic modeling predicts a 100-year flood WSE of 20.25 feet (NAVD88) at the Hammond Trail Bridge location. This WSE corresponds to the worst-case tidal scenario, as shown in Table 5, where both the USGS 100-year discharge and the DWL boundary conditions are applied. Specifically, the Mad River downstream boundary condition uses a DWL of 18.4 feet, while the Arcata Bay boundary condition applies a DWL of 13.4 feet, resulting in the highest modeled WSE at the bridge.

3.1 Uncertainty and Risk Assessment

The hydraulic modeling conducted for the Hammond Trail Bridge replacement project identifies several uncertainties, particularly related to flow dynamics and WSEs under various flood scenarios. This uncertainty stems from factors such as model boundary conditions, channel roughness variations, and the interaction between upstream and downstream flow conditions influenced by tidal and riverine effects. A proactive risk management approach is essential to effectively monitor and address potential changes in hydraulic behavior post-construction.

Key Sources of Uncertainty:

1. **Boundary Conditions**: The model utilizes downstream boundary conditions with static and dynamic water levels MHHW and DWL, representing the interactions with tidal and coastal influences from the Pacific Ocean. The static nature of these boundary conditions provides a conservative scenario but does not fully capture the range of potential fluctuations due to tidal effects, extreme weather, or storm surges, which could impact WSEs at the bridge location. This introduces uncertainty in predicted WSEs during high-flow events.
2. **Channel Roughness and Terrain Variability**: Variability in channel roughness is a significant factor, as changes in Manning's roughness coefficients can alter modeled WSEs. Field observations, aerial imagery, and standard references inform these roughness values. Although sensitivity analyses show limited impacts on WSEs within typical roughness ranges, there remains an inherent uncertainty due to variations in bedform, vegetation, and sediment deposits that may affect local hydraulics during different flow conditions.
3. **Topographic Data and Digital Terrain Model (DTM)**: The terrain model integrates recent LiDAR data (2019) and survey data (2024 and 2007) to establish a comprehensive topographic and bathymetric profile. However, limitations in the spatial resolution of the DTM, especially within complex channel zones, areas of dense vegetation cover, and under-bridge areas, may reduce the accuracy of modeled floodplain interactions. These potential inaccuracies, particularly in flood conveyance areas, introduce uncertainty into flood mapping and WSE predictions, especially during extreme events.
4. **Hydrologic Input and Recurrence Intervals**: The hydraulic model incorporates flood recurrence data from various sources, including FEMA Flood Insurance Study (FIS) and USGS gage records, providing discharge values for 50-year and 100-year flood events. However, differences between FEMA and USGS values for certain segments suggest a degree of uncertainty in discharge projections,

Preliminary Analysis of Uncertainty and Risk

Based on the factors outlined, we provide the following specific recommendations for the 2D hydraulic modeling results detailed in Table 5:

1. **Upstream of the Hammond Trail Bridge Crossing:** The 100-year WSEs in Table 5 and Table 6 carry a recurrence probability of approximately 1 in 100. This is based on stable hydrologic conditions observed upstream, consistent with historical records. Consequently, significant deviations in WSEs upstream are unlikely, as this segment aligns with past study results, indicating low variance in modeled hydraulic conditions.
2. **Downstream of the Hammond Trail Bridge Crossing:** Beyond the immediate bridge location, the likelihood of modeled 100-year WSEs occurring in a given year is expected to be significantly less than 1 in 100. This lower frequency reflects the limited probability of extreme tidal influences coinciding with a 100-year riverine flow event. Additionally,

the downstream extent provides an opportunity for adaptive management strategies to mitigate risks effectively if unanticipated flow behaviors arise.

3.2 Preliminary Scour Analysis

To evaluate potential scour at the Hammond Trail Bridge, including pier, bank, and toe scour, a preliminary scour analysis was conducted using both 1D and 2D hydraulic models. The flow conveyed through the bridge section between the approach embankments during the FEMA 100-year flood event (total discharge of 90,960 cfs) is estimated to be 28,000 cfs, based on results from the current 2D model. The remaining 62,960 cfs flows overbank, with a significant portion directed toward Arcata Bay, which helps to alleviate scour potential within the main channel at the bridge crossing. Preliminary scour analysis was analyzed using an initial DEM generated solely from the 2019 LiDAR dataset which does not include bathymetry and assumes no tidal influence on the downstream boundary to avoid reducing scour impacts associated with backwater conditions.

It should be noted that this conveyance discharge may increase, potentially resulting in more severe scour than currently estimated, and should be considered preliminary to guide bridge pier embedment needs. As additional data is collected, bed gradation and scour predictions may change accordingly.

Hydraulic modeling to assess bridge scour was conducted for existing conditions and three potential bridge replacement options (design concepts provided by Mark Thomas). A 1D model in HEC-18 Hydraulic Toolbox Bridge Scour Design was utilized extending approximately five channel widths upstream and downstream of the Hammond Trail Bridge, with increased cross-section density near the bridge to enhance model accuracy. The resulting depths of both pier-only and combined (contraction and pier) scour for each scenario are summarized in Table 6. The piers are identified as Left, Middle, and Right, looking in the downstream direction.

Table 6. Predicted Scour Depths at Hammond Trail Bridge

Scenario	# of Piers	Scour Result	Left Pier Scour Depth (ft)	Middle Pier Scour Depth (ft)	Right Pier Scour Depth (ft)
Existing Conditions	2	Pier Only	15.8		167
		Combined	16.7		17.6
Option 1	2	Pier Only	12.9		12.2
		Combined	13.5		12.8
Option 2	1	Pier Only		12.8	
		Combined		13.2	
Option 3	3	Pier Only	12.9	12.8	12.9
		Combined	13.7	13.7	13.7

To evaluate bank scour and potential toe erosion that could undermine the proposed riprap revetment, Stillwater utilized the USDA-ARS Bank Stability and Toe Erosion Model (BSTEM) within HEC-RAS. The 2015 SHN geotechnical report for the Hammond Trail Bridge, along with additional reference documents, provided essential data on bank material stratigraphy and composition. Although some assumptions were necessary to run the simulation, various equations and calculation options were iterated to establish a range of potential scour values along the abutment slopes. Using the FEMA 100-year flood storm event, with an estimated discharge of 28,000 cfs conveyed through the bridge section between the approach embankments, potential bank scour was estimated to be 4 to 6 feet of lateral retreat.

Additional parameters, such as toe scour and riprap sizing were calculated using HEC-RAS hydraulic design functions. Anticipated toe scour depths based on three applicable equations, ranged from 9 to 12 feet below the existing channel elevation, suggesting a risk of undermining the riprap revetment. For additional stability, either a keyed-in riprap toe or a launchable toe design is recommended. Riprap sizing calculations for the left and right banks beneath the bridge suggest a recommended D50 size of 24 inches (D30 = 15 inches and D100 = 33 inches) with a minimum layer thickness of 33 inches. Hydraulic analysis of the design cross-section yielded an average channel velocity of 6.4 ft/s, with right and left bank velocities at 3.0 ft/s and 2.6 ft/s, respectively.

3.3 Floodplain Impacts and Regulatory Requirements

The proposed replacement of the Hammond Trail Bridge incorporates design modifications that may influence its interaction with the floodplain, primarily due to the addition of extra piers within the river channel and the relocation of abutments closer to the banks. These changes have the potential to impact WSEs and alter floodplain dynamics, necessitating evaluation to ensure compliance with FEMA's floodplain management standards.

The addition of piers within the river could increase localized hydraulic resistance, potentially raising WSE around the piers during high-flow events. However, the bridge is situated within a relatively large floodplain, which helps distribute floodwaters across a broad area, likely minimizing any WSE increase. Preliminary modeling suggests that changes in WSE will be minimal and generally within FEMA's allowable limits for floodplain encroachments.

Relocating the abutments closer to the banks may slightly reduce the effective channel width, potentially increasing flow velocities near the abutments and concentrating flow toward the center of the channel. Despite this adjustment, the expansive floodplain and its conveyance capacity should dissipate these effects without significantly impacting floodplain storage or flow patterns.

FEMA regulations stipulate that new bridge designs must not increase the 100-year WSE by more than 1 foot to prevent adverse impacts on surrounding properties and infrastructure. Preliminary analysis indicates that if the proposed bridge's low chord elevation matches or exceeds that of the existing bridge, substantial changes to the WSE are unlikely. However, the closer proximity of the abutments to the top of the bank could result in localized WSE increases that approach or exceed the 1-foot limit. Final hydraulic modeling will be conducted to verify that the design remains compliant with FEMA regulations. Furthermore, the bridge design will incorporate scour protection measures, such as riprap around abutments and piers, to safeguard against erosion during flood events, thus maintaining structural stability and floodplain function.

Overall, due to the expansive nature of the floodplain, any WSE increase resulting from the new bridge is expected to be minor and within FEMA's regulatory thresholds. As the design progresses, refined modeling will ensure that floodplain impacts remain minimal and that the bridge design meets all FEMA floodplain management requirements.

Next Steps for Risk Management

This preliminary uncertainty and risk assessment provides a high-level framework for evaluating hydraulic responses relevant to the Hammond Trail Bridge replacement. It is recommended that site-specific design activities include more detailed assessments to quantify and address localized risks, particularly those associated with boundary condition variations and channel morphology changes. Such refined analyses will support engineering designs that are robust, resilient, and capable of adapting to potential hydraulic variations across different flood events.

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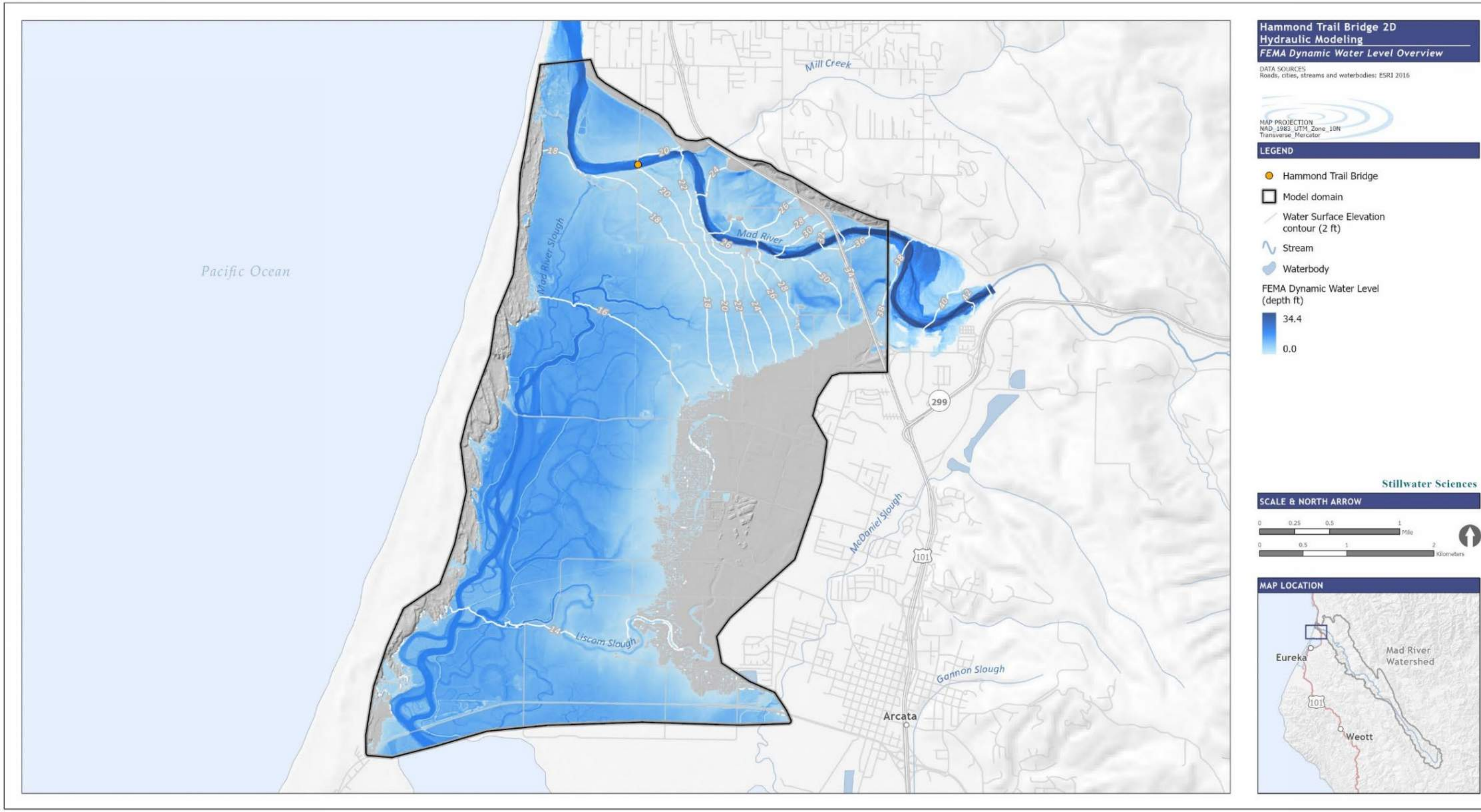
Appendices

Appendix A

Floodplain Maps Showing Average Expected Inundation and Water Surface Elevations During 100-year Storm Event for the FEMA Dynamic Water Level Scenario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FEMA Dynamic Water Level Overview
DATA SOURCES
Roads, cities, streams and waterbodies: ESRI 2016
MAP PROJECTION
NAD_1983_UTM_Zone_10N
Transverse_Mercator
LEGEND
Hammond Trail Bridge
Model domain
Water Surface Elevation contour (2 ft)
Stream
Waterbody
FEMA Dynamic Water Level (depth ft)
34.4
0.0
Stillwater Sciences
SCALE & NORTH ARROW
MAP LOCATION
Eureka
Mad River Watershed
Weott
Pacific Ocean
Mill Creek
Mad River
Mad River Slough
McDaniel Slough
Liscom Slough
Gannon Slough
Arcata
299
101


Hammond Trail Bridge 2D
Hydraulic Modeling
FEMA Dynamic Water Level Page 1 of 3
DATA SOURCES
Roads, cities, streams and waterbodies: ESRI 2016
Imagery: City of Eureka 2019
Underlying imagery: NAIP 2022
MAP PROJECTION
NAD_1983_UTM_Zone_10N
Transverse_Mercator
LEGEND
Model domain
Water Surface Elevation contour (0.5 ft)
Adjacent tile
FEMA Dynamic Water Level (depth ft)
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0.0
Stillwater Sciences
SCALE & NORTH ARROW
0 150 300 600 US Feet
0 50 100 200 Meters
MAP LOCATION
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Hammond Trail
Mad River
19
19.5
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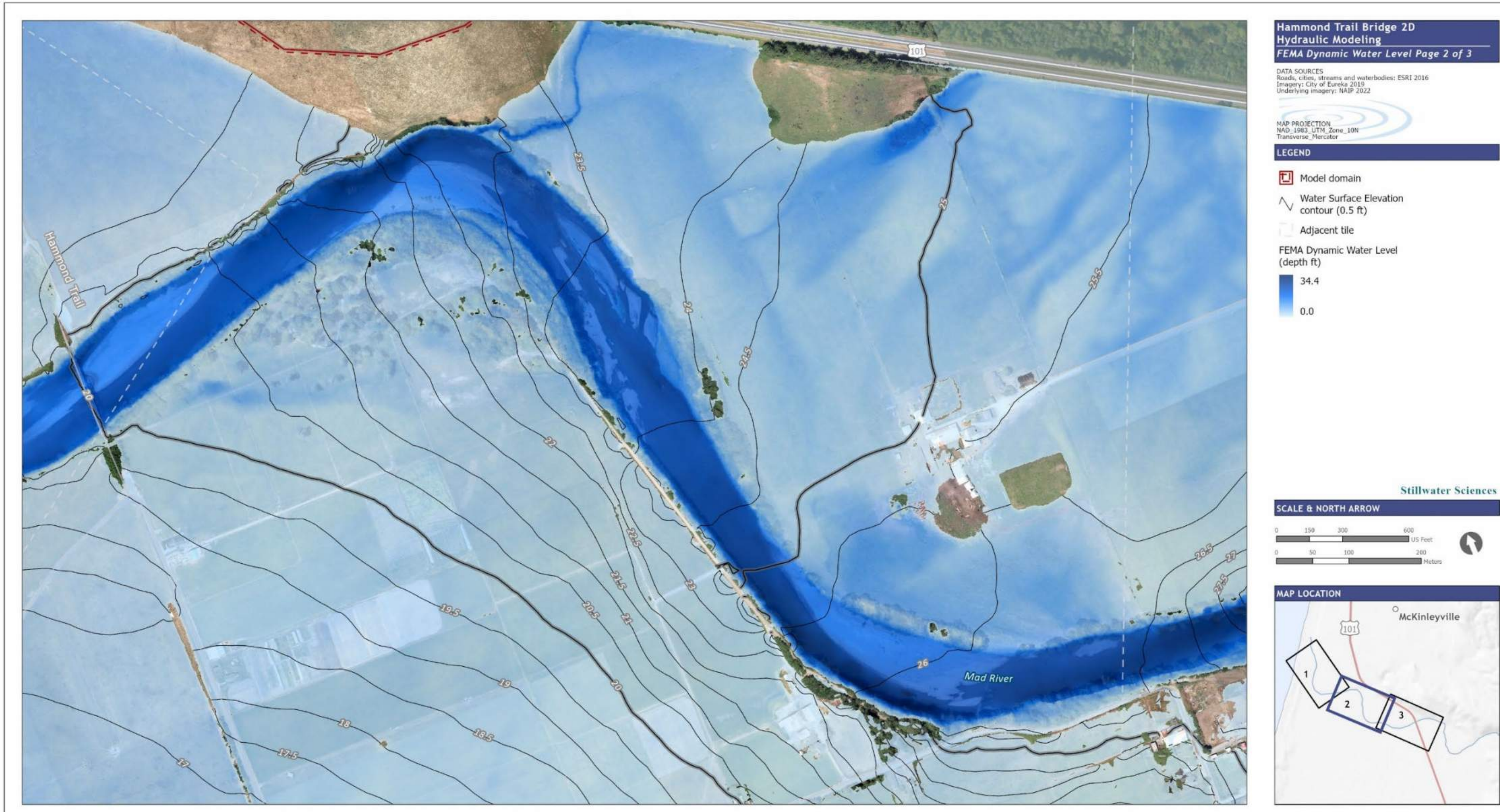
17.5
17
18.5
18


Hammond Trail Bridge 2D Hydraulic Modeling
FEMA Dynamic Water Level Page 2 of 3
DATA SOURCES
Roads, cities, streams and waterbodies: ESRI 2016
Imagery: City of Eureka 2019
Underlying imagery: NAIP 2022
MAP PROJECTION
NAD_1983_UTM_Zone_10N
Transverse_Mercator
LEGEND
Model domain
Water Surface Elevation contour (0.5 ft)
Adjacent tile
FEMA Dynamic Water Level (depth ft)
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Stillwater Sciences
SCALE & NORTH ARROW
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McKinleyville
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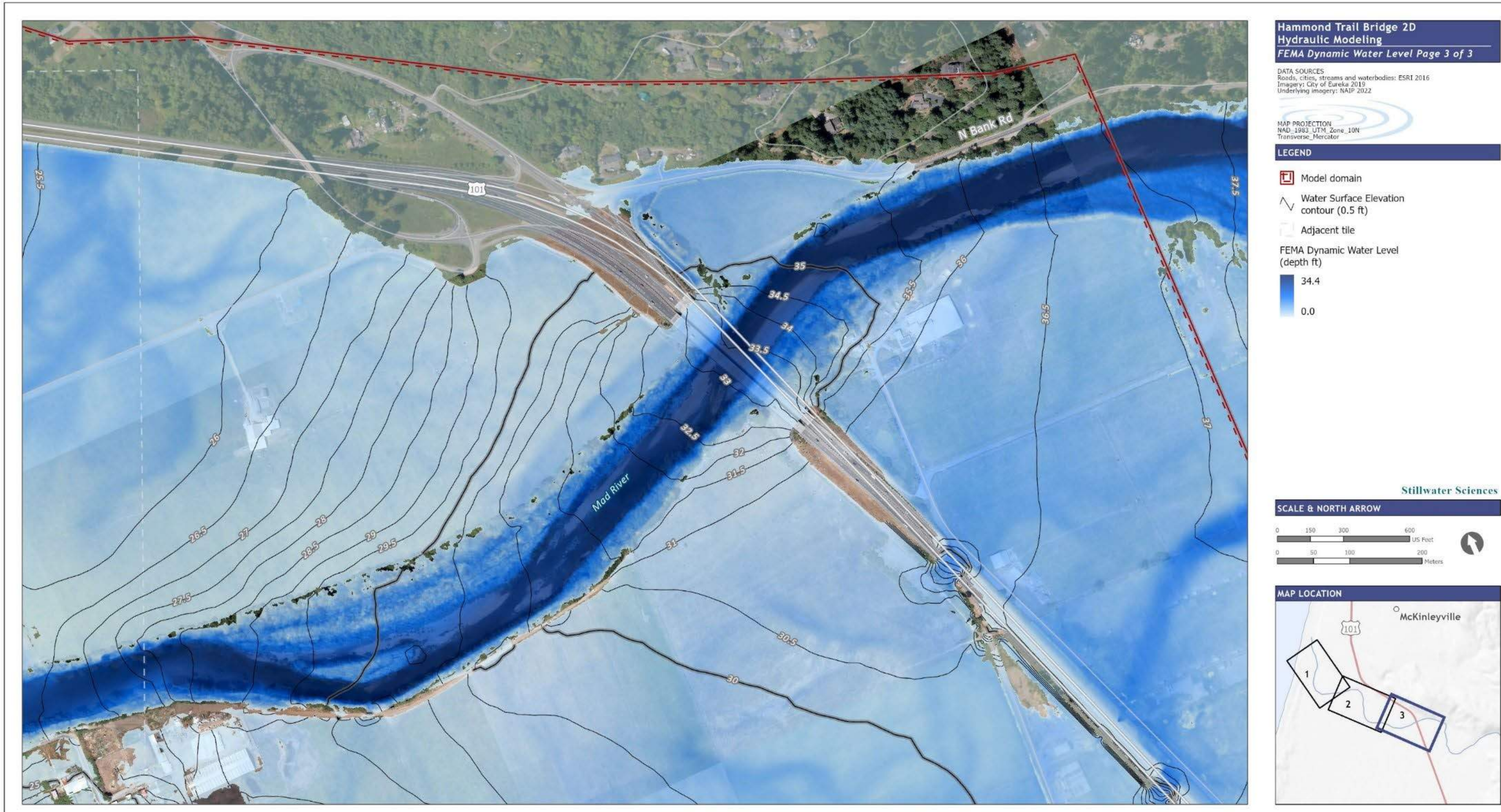
Hammond Trail Bridge 2D Hydraulic Modeling
FEMA Dynamic Water Level Page 3 of 3
DATA SOURCES
Roads, cities, streams and waterbodies: ESRI 2016
Imagery: City of Eureka 2019
Underlying imagery: NAIP 2022
MAP PROJECTION
NAD_1983_UTM_Zone_10N
Transverse_Mercator
LEGEND
Model domain
Water Surface Elevation contour (0.5 ft)
Adjacent tile
FEMA Dynamic Water Level (depth ft)
34.4
0.0
Stillwater Sciences
SCALE & NORTH ARROW
0 150 300 600 US Feet
0 50 100 200 Meters
MAP LOCATION
McKinleyville
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N Bank Rd
Mad River
101
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33.5
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36
36.5
37
37.5
25

Appendix B

Hammond Trail Bridge Cross-section Showing Modeled 1% AEP Water Surface Elevations

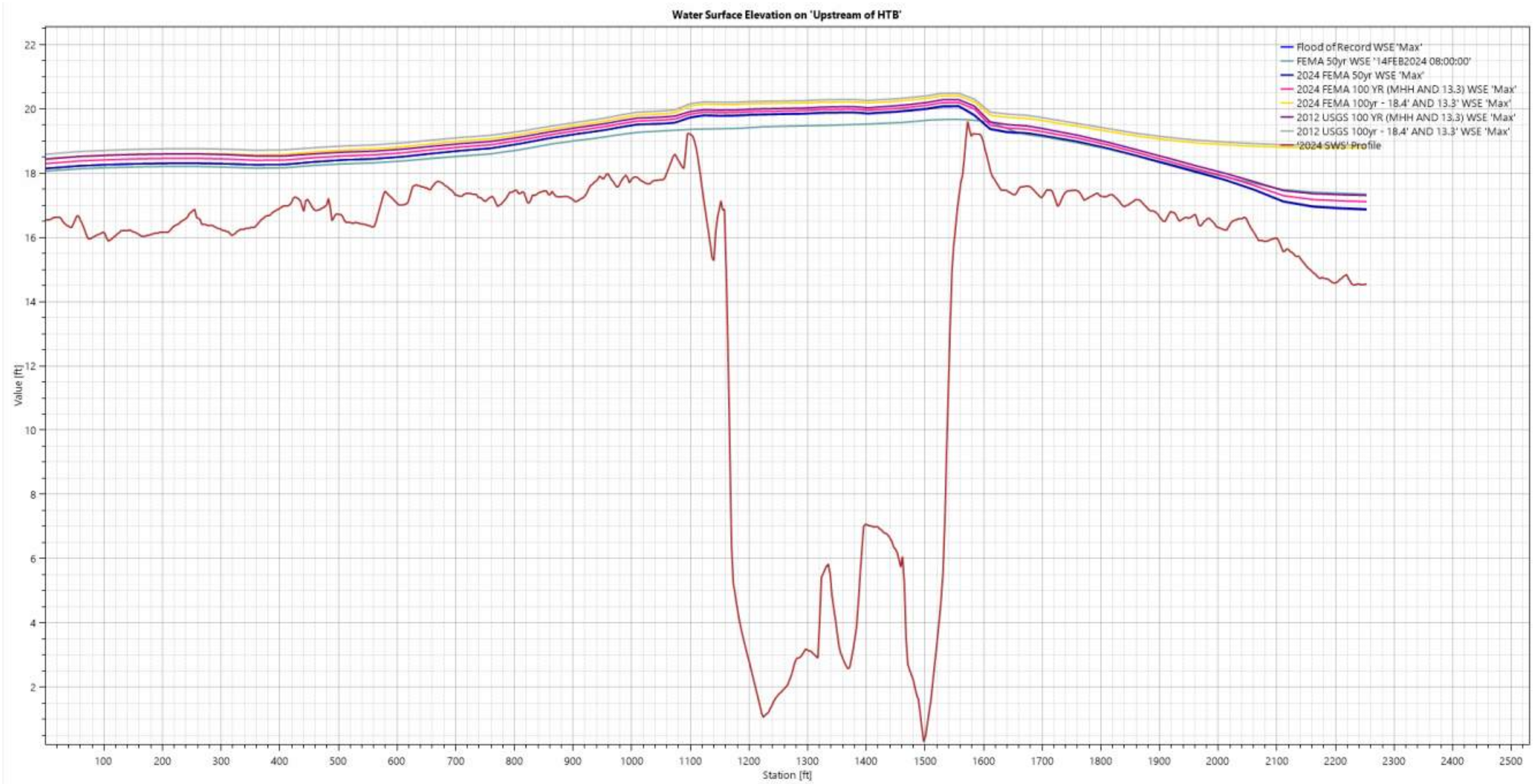

Figure 1. Cross-Section upstream of Hammond Trail Bridge. 2024 SWS is the combined terrain discussed in Section 2.1 Digital Terrain Model Development.

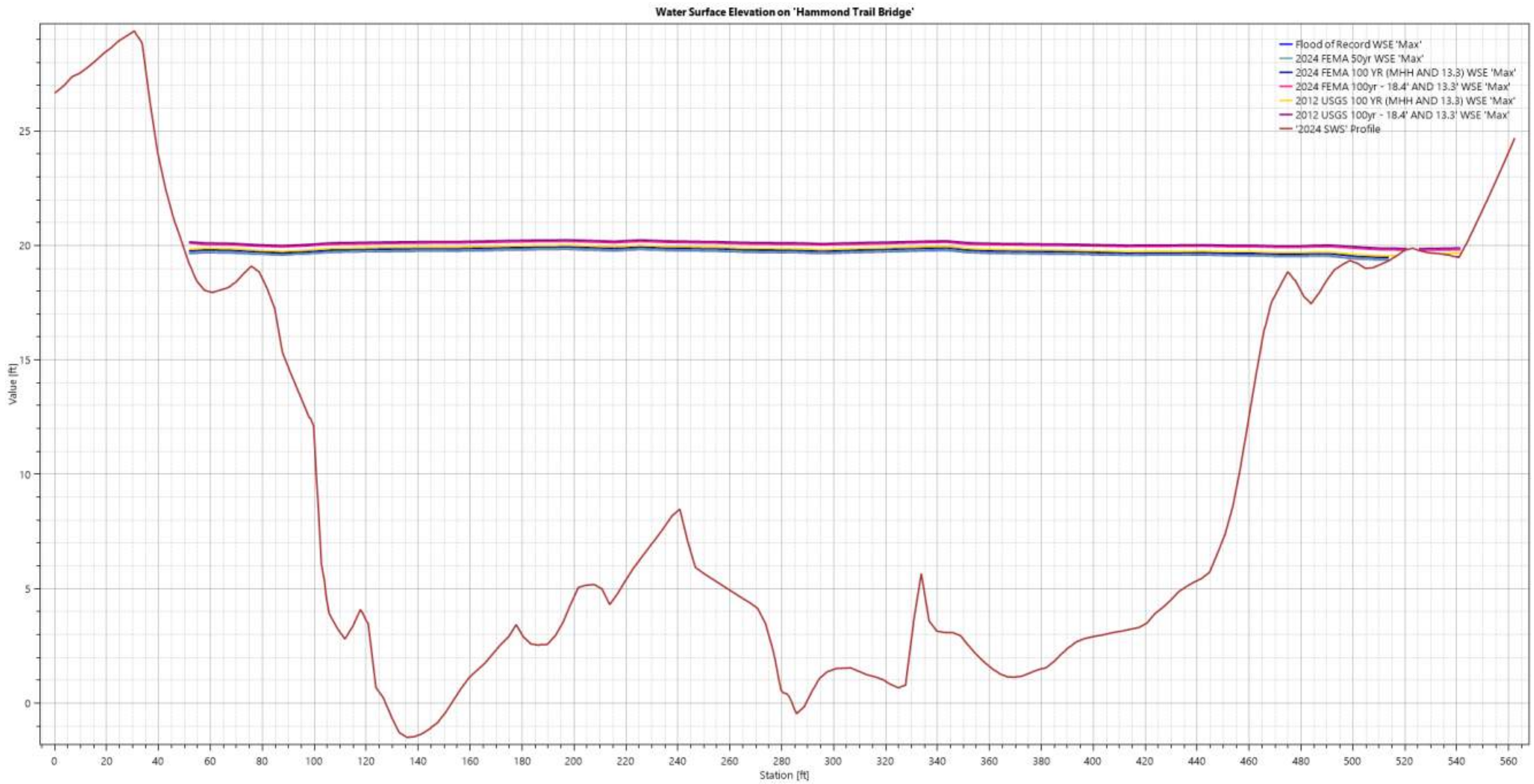

Figure 2. Cross-Section at Hammond Trail Bridge between the approach embankments. 2024 SWS is the combined terrain discussed in Section 2.1 Digital Terrain Model Development.

Figure 3. Cross-Section downstream of Hammond Trail Bridge. 2024 SWS is the combined terrain discussed in Section 2.1 Digital Terrain Model Development.