Cowling Creek Culverts Replacement at Miller Bay Road NE

Feasibility Evaluation

Project No. E316301200

Cardno
Shaping the Future

Document Information

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Table of Contents

Appendices

Tables

Figures

1 Introduction

The Cowling Creek watershed is located near Suquamish, Washington, on the Kitsap Peninsula. The watershed covers an area of 1.9 square miles and contains approximately 12 miles of streams, 5.5 miles of which are fish bearing (Wild Fish Conservancy 2010). Cowling Creek crosses under Miller Bay Road NE, approximately 350 feet upstream of its outlet into Miller Bay (Figure 1). Currently, the crossing consists of two 36-inch-diameter concrete culverts that are covered by approximately 40 feet of fill by the Miller Bay Road NE embankment. A culvert assessment conducted by Washington State Department of Fish and Wildlife (WDFW) in 2014 concluded that the existing twin 36-inch culverts are a fish passage barrier at all flows.

Mid Puget Sound Fisheries Enhancement Group (MSFEG) received funding from the Salmon Recovery Funding Board (SRFB) to conduct a feasibility study for replacement of the existing culverts at the Miller Bay Road NE crossing. Due to the lack of bypass alternatives for the road, Kitsap County is requiring that at least one lane of the road remain open at all times during construction of a replacement crossing. To develop feasible alternatives for a crossing structure that allows for fish passage under Miller Bay Road NE, Cardno has teamed with Shannon & Wilson and Wilson Engineering to develop feasibility level design alternatives. Cardno is contributing project management, fish passage design, hydrology, hydraulics, geomorphology, and stream channel design elements. Shannon & Wilson is furnishing geotechnical expertise, and Wilson Engineering is providing topographic surveying services.

Project objectives for this phase of work include evaluation of the existing conditions at the crossing, development of conceptual design alternatives and costs, selection of a preferred alternative, and development of conceptual plans, cost estimate, and design report for the preferred alternative. The conceptual plans and design report will aid MSFEG in obtaining future funding for the project to move forward with design and construction.

This document provides the relevant site background information that was used in developing the conceptual alternatives, description of the alternatives, and a discussion of feasibility, advantages, and risks associated with each alternative. The expected outcome from this document is the selection of a preferred alternative to be advanced to 15% design and used to support future funding requests.

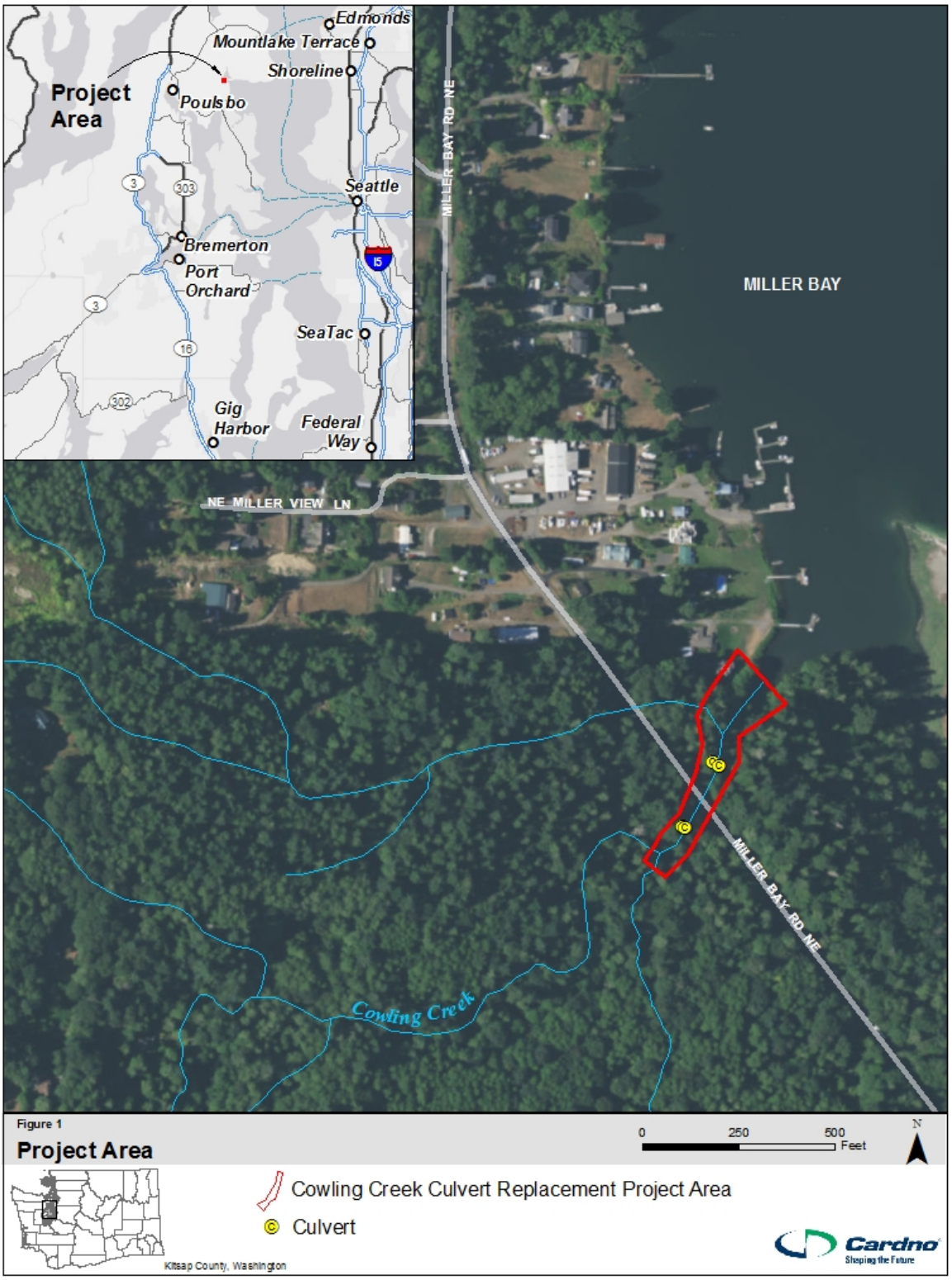

Figure 1. Project and vicinity map.

2 Site Reconnaissance

On January 10, 2017, Cardno and Shannon & Wilson staff performed a site reconnaissance of the Cowling Creek stream channel and the Miller Bay Road NE embankment to evaluate the existing conditions of the stream crossing and explore alternatives for culvert replacement. Findings from the geotechnical investigation are provided in Appendix A, Concept-Level Geotechnical Engineering Report. Wilson Engineering was also onsite during this time to perform the topographic survey of the stream and roadway embankment for the development of a topographic base map.

At the Miller Bay Road crossing, the existing twin 36-inch-diameter concrete culverts are covered by approximately 40 feet of fill. The road embankment extends approximately 65 feet southwest to the toe of the slope where the culvert inlet is located. The embankment extends 80 feet northeast to the toe of the slope where the culvert outlet is located. The existing culverts are 180 feet in length and set at a slope of 3.2%. The roadway embankment slopes range from 1.3 H:1 V to 1.7:1 V and are covered with small trees and shrubs.

Downstream of the Miller Bay Road NE crossing, Cowling Creek displays a wider cross section as it flows into Miller Bay (Figure 2). Cowling Creek downstream of the culvert outlets is tidally influenced, and substrate consists primarily of gravels and sands. During lower tides, flow from the culverts travels downstream until it reaches Miller Bay approximately 300 feet downstream. During tides of mean higher high water (MHHW) and above, the culvert outlets are completely submerged and flow in the culverts is backwatered. A riprap apron surrounds the outlet of the culverts. Due to the tidal influence in the downstream section of Cowling Creek, representative bankfull widths were not measured but an approximate ordinary high water width of 39 feet was measured. Approximately 150 feet downstream from the culvert outlets, a smaller, unnamed tributary joins Cowling Creek.

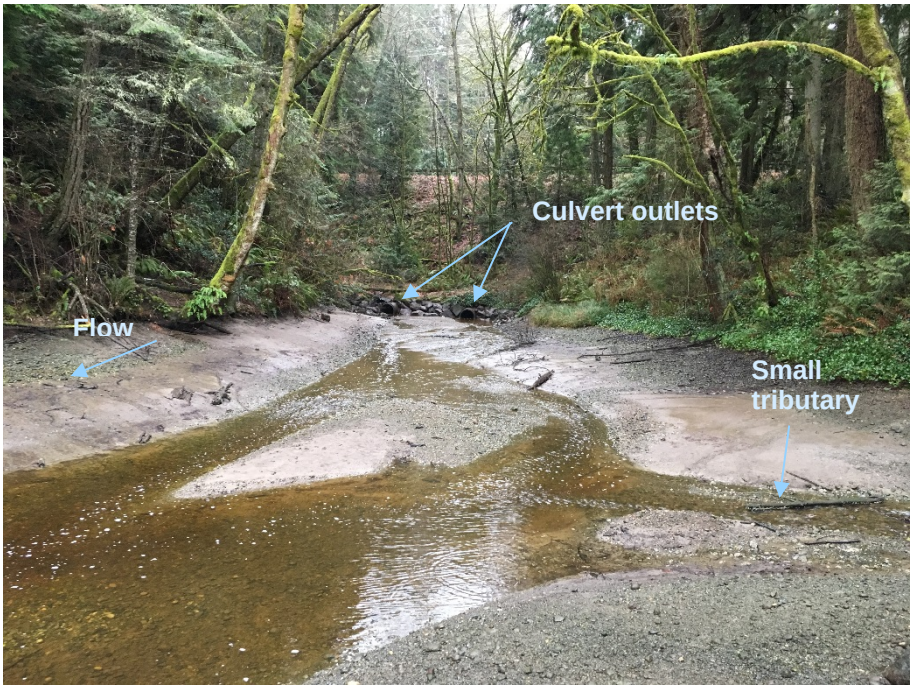

Figure 2. Downstream end of Miller Bay Road NE stream crossing during low tide. Looking upstream.

For approximately 100 feet upstream of the Miller Bay Road NE crossing, Cowling Creek flows northeast through a narrow, confined section of the creek (Figures 3 and 4). The channel in this section is confined

by near-vertical banks on the left bank of the creek and a steep embankment on the right bank. Channel substrate in the reach consists of fine to coarse gravels and is influenced by backwater hydraulics from the undersized crossing at Miller Bay Road NE. Portions of the channel bed and banks reveal exposed glaciolacustrine deposits, suggesting that the channel downcutting may be limited by a basement of more resistant material. Bankfull widths measured in this reach averaged 11.5 feet, with an average bankfull depth of 2 feet.

Figure 3. Confined section of Cowling Creek immediately upstream of culvert inlets. Looking upstream.

Figure 4. Bankfull width measurement of 10 feet in confined section. Looking downstream.

Upstream of the confined reach, Cowling Creek flows through the historical Cowling Creek Hatchery area, which has significantly modified the creek channel and sediment transport characteristics of the reach. The hatchery facilities consist of a concrete headworks at the upstream end, small rearing and acclimation ponds adjacent to the channel in the middle of the facility, and a short concrete flume that flows through a gravel road embankment at the downstream end (Figures 5 and 6). The hatchery headworks and downstream flume have created large, vertical, grade-control structures within Cowling Creek, which have resulted in storage of large volumes of sediment both upstream of the hatchery reach and within the hatchery reach. The hatchery facility has reduced the natural transport of sediment through the reach, resulting in a large amount of aggradation both immediately upstream of the headworks and upstream of the concrete flume near the downstream end of the hatchery. Currently, fish passage through the hatchery facility is aided by a temporary fish ladder at the headworks. If future restoration within the watershed includes removal of the hatchery facilities, it is likely that large volumes of this stored sediment would be mobilized and transported through the reach and downstream.

Figure 5. Channel in hatchery area. Looking upstream.

Figure 6. Headworks at upstream end of hatchery. Note large deposits of coarse sediment upstream of headworks. Looking downstream.

Upstream of the hatchery infrastructure, Cowling Creek returns to a more natural setting with a meandering planform set within a wider valley flood and a lower slope than the reach immediately upstream of the existing culverts (Figure 7). Channel substrate in the reach consists of medium and coarse gravels with cobbles and is more representative of the native sediment supply than either of the downstream reaches. Bankfull width measurements taken in the reach averaged 13 feet and bankfull depth averaged 1.8 feet. Channel banks are well-vegetated with some areas of 1 to 2 feet of near-vertical bank erosion.

Figure 7. Bankfull width measurement of 12.5 feet in reach upstream from hatchery. Looking downstream.

3 Hydrology and Hydraulics

3.1 Hydrology

Peak flow hydrology was developed for the Cowling Creek crossing location to evaluate existing and proposed hydraulics at the crossing site. The Cowling Creek watershed does not have a permanent stream gage installed, but the Suquamish Tribe has collected periodic flow measurements on Cowling Creek between 1976 and 2007 (Figure 8). Measurements collected by the Suquamish Tribe show flows ranging from 1 to 12 cubic feet per second (cfs) with November through April having the highest flows and August through October having the lowest flows (Suquamish Tribe 2017). No peak flow measurements have been conducted by the Suquamish Tribe. Due to the lack of peak flow records for Cowling Creek, peak flow hydrology was evaluated using two methods: regional regression equations and a comparative basin analysis on an adjacent watershed. The results of these two methods were evaluated and appropriate design flows were selected with which to model the existing and proposed hydraulics at the crossing.

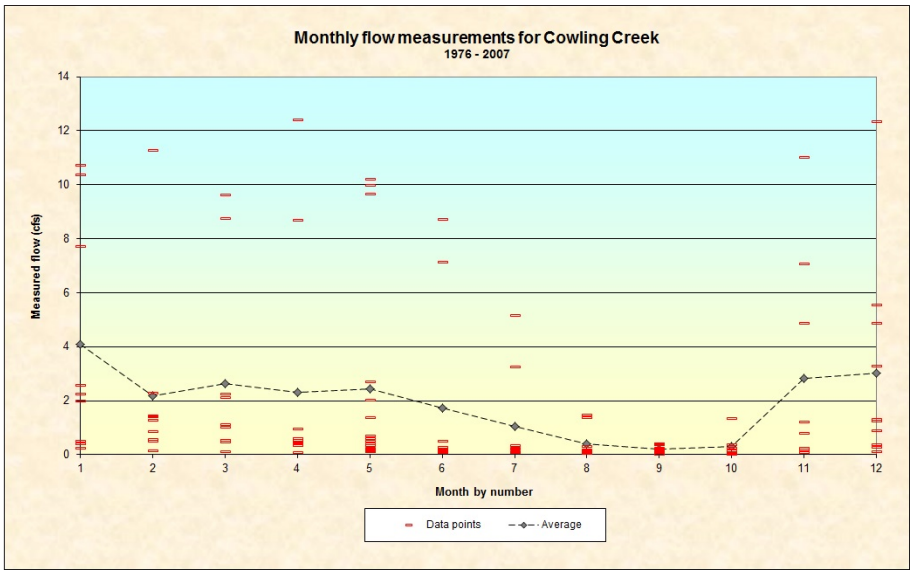

Figure 8. Monthly flow measurements for Cowling Creek (1976–2007).

Computation of peak flows from the regional regression equations is based upon basin area and average annual precipitation data for the watershed. The U.S. Geological Survey (USGS) online tool StreamStats was used to determine both basin area and annual precipitation at the crossing location. In reviewing the Cowling Creek watershed in the StreamStats tool, a disconnection between the “upper” and “lower reaches of Cowling Creek was identified. Based on data collected by Wild Fish Conservancy during their water typing assessment of Cowling Creek, it was determined that the disconnection seen in the StreamStats data was an error and that these two reaches should be connected (Wild Fish Conservancy 2010). The total basin area was determined by combining the basin areas for the “upper” and “lower” reaches identified in StreamStats.

Peak flows were computed from the regression equations for Region 2 using the total basin area and average annual precipitation described above (Knowles and Sumioka 2001). For a comparative analysis, recurrence flows from the Dogfish Creek USGS gage (USGS 12070000) were scaled to the Cowling Creek crossing location based on drainage areas. Dogfish Creek is a basin located to the west of Cowling Creek on the Kitsap Peninsula. The USGS gage on Dogfish Creek has a peak flow record of 28 years and a drainage area of 5 square miles. Table 1 below shows the estimated peak flows from these two analyses.

Table 1. Peak Stream Flows Obtained from Regional Regression Equation and Comparative Basin

Recurrence Interval (years)	Regional Regression Flow (cfs, standard error)	Regional Regression Flow Plus Standard Error (cfs)	Dogfish Creek at USGS 1207000 (cfs)	Dogfish Creek Scaled to Cowling Creek Crossing Location (cfs)
2	30 (56%)	**47**	137	49
10	53 (53%)	**81**	243	87
25	65 (53%)	**100**	303	109
50	76 (53%)	**116**	349	125
100	85 (54%)	**131**	398	143

The peak flow estimates determined from the comparative basin analysis of Dogfish Creek are very similar to the regression equation flows when the standard error is added to the regression flows. Therefore, we determined that the regional regression equations with the standard error included were the most appropriate peak flow values to apply to the evaluation of existing and proposed culvert hydraulics. Utilizing this approach, the site is estimated to have a 2-year flood recurrence of 47 cfs and a 100-year flood recurrence of 131 cfs.

3.2 Hydraulics

3.2.1 Model Development

The evaluation of existing hydraulics within the stream channel and existing culverts was completed using the one-dimensional hydraulic model HEC-RAS. HEC-RAS is a backwater analysis model that was developed by the US Army Corps of Engineers to evaluate channel and culvert hydraulics under both steady and unsteady conditions. For the purposes of this study, HEC-RAS was run under steady state conditions using the peak flow values shown in Table 1 (regional regression plus standard error).

Model geometry was developed from the topographic survey conducted by Wilson Engineering and channel and floodplain roughness observations made by the field reconnaissance team. The model extends 280 feet upstream of Miller Bay Road NE and 275 feet downstream, covering a total of 555 feet of the channel and existing culverts. Channel and floodplain roughness was set based on observations made by the field reconnaissance team and standard hydraulic references. In addition to the existing twin 36-inch-diameter culverts at Miller Bay Road NE, the model also included the concrete flume and overflow pipe associated with the upstream hatchery infrastructure.

Model boundary conditions were set using design flows and tidal data to develop various hydraulic scenarios to evaluate the culvert and channel hydraulics. The upstream boundary of the model was set to the 2- and 100-year recurrence interval flows determined above and the average upstream channel slope. The 2-year flow was modeled to understand the hydraulic conditions that occur during the more frequent channel-forming discharges, whereas the 100-year flows were modeled to evaluate culvert capacity and freeboard. At the downstream end of the model, the boundary was set to a steady state water surface elevation to correspond with variable tidal conditions. For the purposes of evaluating the range of tidal conditions, both MHHW and mean lower water (MLW) were evaluated. The MHHW tidal boundary was used to simulate the effect of frequent tidal backwatering on the culvert hydraulics, whereas the MLW tidal boundary was used to represent the conditions in Cowling Creek when not controlled by a tidal backwater. Table 2 below shows the four model scenarios that were developed for evaluation of the channel and culvert hydraulics.

Table 2. Hydraulic Modeling Scenarios

Model Scenario	Upstream Boundary Condition	Downstream Boundary Condition	Purpose
1	2-year flow (47 cfs)	MHHW (8.95 ft NAVD 88)	Evaluation of channel-forming flow during tidal backwater conditions
2	2-year flow (47 cfs)	MLW (0.53 ft NAVD 88)	Evaluation of channel-forming flow during free-flowing conditions (not tidally backwatered)
3	100-year flow (131 cfs)	MHHW (8.95 ft NAVD 88)	Evaluation of culvert capacity during tidal backwater conditions
4	100-year flow (131 cfs)	MLW (0.53 ft NAVD 88)	Evaluation of culvert capacity during free-flowing conditions (not tidally backwatered)

3.2.2 Existing Conditions Model Results

The existing twin 36-inch-diameter concrete culverts at Miller Bay Road NE were evaluated for the four hydraulic scenarios presented in Table 2. Figures 9 and 10 below show the resulting water surface profiles for these four scenarios under existing conditions. At both the 2- and 100-year flows during a MHHW tide, the downstream outlet of the existing culverts is submerged, creating a backwater effect inside the culvert. For the 2-year flow, this backwater effect only extends part of the way up the culvert, whereas during the 100-year event the entire culvert is backwatered and this backwater effect continues upstream to the flume. During the free-flowing event at an MLW tide, both the 2- and 100-year flows are not backwatered at the outlet. However, the 100-year flow does create a backwater condition at the culvert inlet and extends upstream to the flume, indicating that the existing culvert capacity is insufficient to pass the 100-year flood.

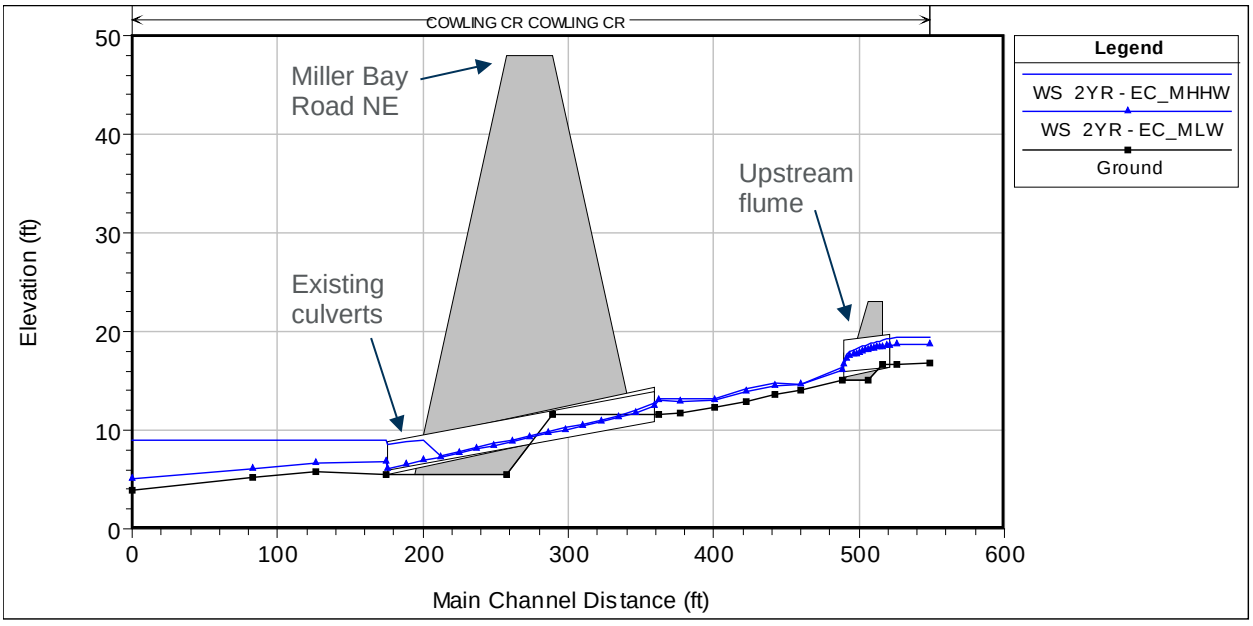

Figure 9. Existing culverts at 2-year flow at tidally backwatered and free-flowing conditions.

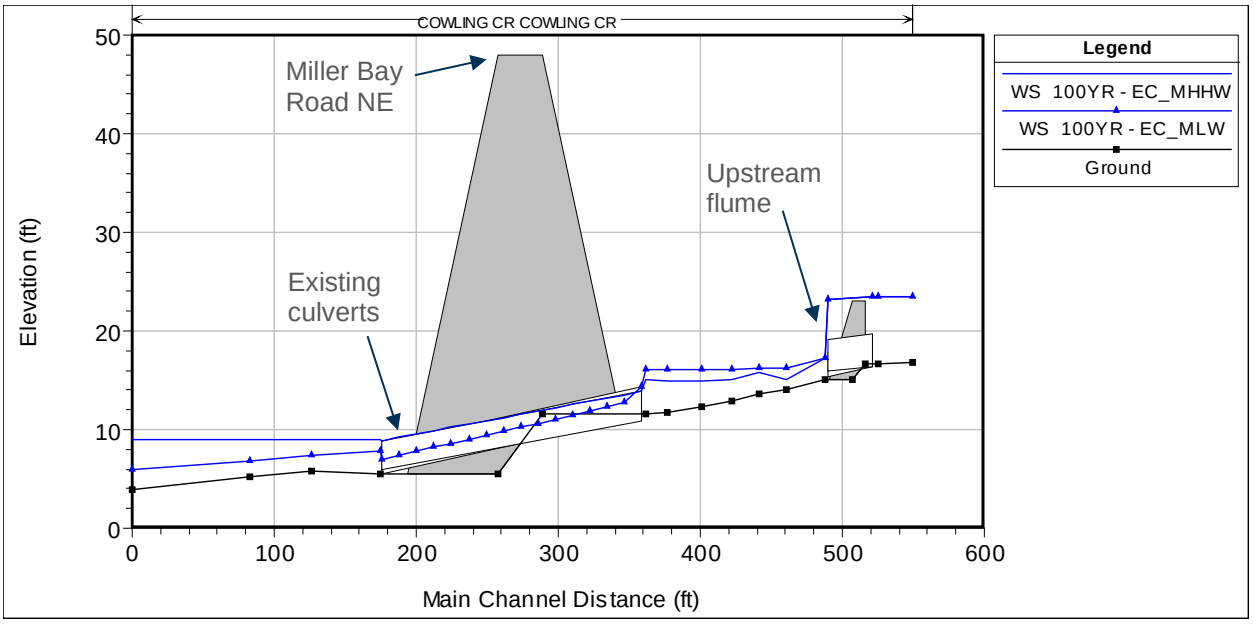

Figure 10. Existing culverts at 100-year flow at tidally backwatered and free-flowing conditions.

4 Fish Passage Design

Concepts for fish passage design of the Cowling Creek crossing under Miller Bay Road NE followed guidance developed by WDFW in the 2013 Water Crossing Design Guidelines (WCDG) (Barnard et al. 2013). Crossing design followed the general channel geometry requirements outlined in the Stream Simulation Culvert Design Option in the WCDG, which is based on sizing the minimum width of the crossing structure based on representative bankfull channel widths; the exception to this is the jack/bore option, which does not meet the WCDG criteria. Stream Simulation can be applied to channels with a bankfull width of less than 15 feet and a wide range of channel slopes. Channel crossing structures are also designed to have a streambed material mixture that is representative of the existing material in the creek and that takes into consideration sediment supply and future sediment transport dynamics following the installation of the larger/replacement crossing structure.

4.1 Minimum Crossing Structure Width

The minimum crossing structure/channel width was determined based on the average bankfull width measured in the reach between the culvert inlet and the upstream hatchery of 11.5 feet. The minimum crossing width can be determined by Equation 3.2 in the WDCG:

$$W_{culvert\ bed} = 1.2W_{channel} + 2\ (feet)$$

Based on the average bankfull width of 11.5 feet, this results in a minimum crossing structure width of 15.8 feet.

4.2 Existing Streambed Sediment Distribution

Streambed sediment for the proposed crossing shall be designed to mimic the natural streambed sediment in the creek while also taking into account sediment mobility and available sediment supply. Streambed sediment downstream and upstream from the culvert was characterized during the field reconnaissance by completing Wolman pebble counts in each of these areas. Table 3 below shows the resulting sediment distributions for each location.

Table 3. Pebble Count Results

Diameter Class	Downstream of Miller Bay Road NE (inches, classification)	Upstream of Miller Bay Road NE below Hatchery (inches, classification)	Upstream of Hatchery (inches, classification)
D_{16}	0.2 (fine gravel)	0.2 (fine gravel)	0.3 (fine gravel)
D_{50}	0.5 (medium gravel)	0.6 (medium gravel)	0.8 (coarse gravel)
D_{84}	0.9 (coarse gravel)	1.6 (coarse gravel)	1.9 (coarse gravel)
D_{100}	10.1 (cobble)	5.0 (cobble)	7.1 (cobble)

The results of the pebble counts in the three reaches of Cowling Creek indicate that substrate material sizes decrease traveling downstream in the system. Sediment distribution within the reaches surrounding the Miller Bay Road NE crossing have been affected by both the undersized crossing and the historical hatchery infrastructure. Both of these features have functioned as grade structures that have limited the transport of larger-sized sediment downstream, resulting in the lack of larger sediment sizes seen in the pebble count data. It is expected that the replacement of the existing undersized culverts with a larger crossing structure and the potential future removal of the hatchery infrastructure would reestablish natural sediment transport through the system.

5 Conceptual Culvert Replacement Options

Cardno and Shannon & Wilson developed three conceptual culvert replacement alternatives intended to offer a range of potential actions deemed feasible according to the geotechnical, geomorphic, and hydraulic considerations and based on experience with similar projects. Estimated conceptual construction costs for each alternative were developed based on past project experience, correspondence with contractors, and estimated material quantities and unit costs.

To develop and evaluate these alternatives, the following design objectives were considered:

> Provide fish passage through the Miller Bay Road NE crossing of Cowling Creek at a range of flows;

> Maintain a minimum of one lane of public access along Miller Bay Road NE through project construction;

> Meet WDFW fish passage design criteria for the crossing design;

> Improve instream habitat and geomorphic conditions within the project reach;

> Provide crossing designs that will be resilient to expected future changes upstream of the project reach such as removal of existing hatchery infrastructure and the expected channel response to those changes; and

> Minimize construction costs.

5.1 Option 1: 16-foot Arch Culvert Installed via Tunneling

To meet the WDFW Stream Simulation crossing width requirements, a 16-foot-wide arch culvert will be installed by tunneling through the Miller Bay NE road embankment. Tunneling will be performed by installing an arch soil support curtain consisting of either pressure grouted pipes or ground freezing above the proposed culvert. Once the soil support curtain is installed and the embankment soils are stabilized, the tunnel will be excavated using traditional earthwork equipment and the arch culvert incrementally advanced using bolted plate sections. The arch culvert will sit on top of a concrete stem wall, which allows for an increased depth of streambed material and maximum channel width within the culvert. The proposed culvert length will be 140 feet, which is approximately 40 feet shorter than the existing culverts. The new channel would be constructed upstream and downstream of the culvert to tie into the existing stream beyond the embankment limits.

Temporary access roads would need to be installed on either side of the roadway down to the toe of the roadway embankment. On the upstream end, the existing gravel road could be utilized for access and staging, whereas on the downstream end a new access and staging area would need to be created. During construction, Cowling Creek would be bypassed through the existing northern 36-inch culvert. Both lanes of Miller Bay Road NE would remain open throughout construction, and clearing and grading of the adjacent roadway embankment would be minimized.

Design concept figures are presented in Appendix B, Conceptual Culvert Replacement Options Figures.

Estimated Cost: $5.4 Million

Miller Bay Road NE Access: Two-lanes throughout construction

5.2 Option 2: 50-foot-Long Bridge on H-Piles

The second culvert replacement option is to construct a 50-foot-long bridge supported by H-piles and construction of 180 feet of new stream channel. The bridge could be a pre-fabricated bridge, consisting of multiple prestressed, hollow-core bridge beams with a paved surface, or be constructed using prestressed concrete beams with concrete decking. At each bridge approach, a soldier pile retaining wall

will be constructed below the bridge. This wall will wrap around parallel to each side of the road to retain embankment fill. Each end of the bridge will bear on pile caps constructed on top of the soldier pile (H-pile) foundations. Below the bridge, the H-piles will serve both as bridge foundations and as retaining wall elements.

Bridge and channel construction will be completed in stages. To reduce traffic impacts, the bridge will be constructed first, then excavation of embankment from below the bridge will occur. During construction one lane of Miller Bay Road NE will be closed as each half of the bridge is constructed. Soldier piles will be installed first. Temporary relocation or de-energizing of overhead power will be required during soldier pile installation. Where soldier piles are installed through the roadway, they will be cut off below top of pavement elevation. After soldier pile installation, shaft caps will be constructed over the soldier piles at each abutment, the embankment excavated down to just below the bottom of bridge beam elevation, and the first half of the bridge, constituting one lane, installed. Following construction of the first half of the bridge, traffic will be shifted onto the one lane of new bridge while the other half of the bridge is constructed using a similar construction sequence. After finishing construction of the bridge, both lanes will be open to traffic.

Following completion of the bridge construction, excavation of the existing embankment below the bridge and stream grading will be performed by equipment working within the ravine and on the embankment slopes. Timber lagging will be placed between soldier piles and tiebacks installed as the excavation advances down. The stream will continue to flow through the existing culverts until embankment excavation is near completion. Final phases of stream construction will include diverting the stream through temporary channels and possibly through bypass pipes. Final construction will include constructing permanent concrete fascia on the retaining walls, installing driven soil nails to improve stability of the steep slope, and vegetating the slope.

Design concept figures are presented in Appendix B, Conceptual Culvert Replacement Options Figures.

Estimated Cost: $3.5 Million

Miller Bay Road NE Access: Temporary one-lane closure to install each side of bridge. Two-lanes open once bridge construction is completed.

5.3 Option 3: 8-foot Circular Culvert Installed via Jack and Bore

To improve existing fish passage at the crossing location and minimize construction costs, a smaller 8-foot-diameter culvert, which could be installed via jack and bore construction, was evaluated. While this option does not meet Stream Simulation crossing width requirements, it would offer a significant improvement to existing fish passage at the crossing location at a substantially lower construction cost. Within the 8-foot culvert a roughened streambed mixture would be placed to increase hydraulic roughness and decrease culvert velocities, thereby increasing the range of flows passable for fish. The 8-foot culvert would be of a similar length to the existing culverts and would have minimal channel grading at the culvert inlet and outlet.

Temporary access roads would need to be installed on either side of the roadway down to the toe of the roadway embankment. On the upstream end the existing gravel road could be used for access and staging, whereas on the downstream end a new access and staging area would need to be created. At the inlet and outlet of the proposed 8-foot culvert, a staging area would be required for boring and receiving equipment. During construction Cowling Creek would be bypassed through the existing northern 36-inch culvert. Both lanes of Miller Bay Road NE would remain open throughout construction, and clearing and grading of the adjacent roadway embankment would be minimized.

Design concept figures are presented in Appendix B, Conceptual Culvert Replacement Options Figures.

Estimated Cost: $1.4 Million

Miller Bay Road NE Access: Two-lanes throughout construction

5.4 Technical and Stakeholder Review

On May 15, 2017, the project team met to discuss the conceptual culvert replacement options and select a preferred alternative. Participants included members of Mid Puget Sound Fisheries Enhancement Group, Suquamish Tribe, Kitsap County Public Works, Cardno, and Shannon and Wilson. Discussion of conceptual alternatives focused on construction costs, construction impacts to Miller Bay Road NE, fish passage and habitat benefits, and permitting constraints. Based on these criteria, Option 2: 50-foot-long Bridge on H-Piles was selected as the preferred alternative. This option was selected based on a lower estimated construction cost and overall construction complexity than Option 1: 16-foot Arch Culvert Installed via Tunneling. Option 3: 8-foot Circular Culvert Installed via Jack and Bore, although the least expensive of the options, was not selected due to not meeting fish passage requirements for structure width.

6 Preferred Conceptual Design

The preferred conceptual design was developed based on Conceptual Design Option 2 and feedback obtained during the technical and stakeholder review. The 15% conceptual design drawings are shown in Appendix C. Elements of the conceptual design are discussed in the following sections.

6.1 Crossing Structure

The crossing structure for Cowling Creek under Miller Bay Road NE will consist of a 50-foot-long bridge installed on H-piles. The 50-foot-long bridge will provide a 40-foot-wide opening for Cowling Creek to pass under Miller Bay Road NE. Bridge deck width will provide for two lanes of traffic and can be modified to meet future Kitsap County standards and incorporate optional roadway elements such as bike lanes or sidewalks. The bridge will be supported by vertical H-piles that will be installed in predrilled holes. To support the roadway embankment and bridge opening, a permanent soldier pile wall consisting of permanent horizontal lagging and tieback anchors will be installed between the H-piles. Additional details on crossing structure elements and geotechnical design considerations are provided in Appendix A, Concept-Level Geotechnical Engineering Report.

6.2 Construction Sequencing and Impacts

Construction of the 50-foot-long bridge will occur in phases to keep one lane of traffic along Miller Bay Road NE open throughout construction. Construction of the bridge will occur first with the excavation of the roadway embankment, and stream channel grading following the completion of the bridge. Bridge construction will require alternating lane closures of the road as each lane of the bridge is installed. Following the installation of the first lane of the bridge, traffic will be routed over the bridge as the piles and bridge are installed on the second lane. After finishing construction of the bridge, both lanes will be open to traffic.

Primary construction impacts include the following items and will be considered in greater detail during future design phases:

> Lane closure timing and traffic impacts to Miller Bay Road NE;

> Temporary or permanent relocation of utilities along Miller Bay Road NE; and

> Limited access and staging areas along existing road right-of-way;

6.3 Aquatic Habitat Benefits

The construction of the 50-foot-long bridge at the Miller Bay Road NE crossing will provide for fish passage at all flows along Cowling Creek. By removing the existing 183-foot-long twin culverts and replacing them with the bridge, approximately 140 feet of stream channel will be constructed, resulting in an increase in habitat within the project reach. The 40-foot-wide bridge opening will allow for the construction of floodplain benches outside of the active channel that will be engaged during large flow events and high tidal conditions. Other habitat elements, such as the placement of large wood structures and riparian planting, may be added as part of the streambed grading to provide additional habitat benefits within the project reach.

6.4 Conceptual-Level Construction Cost Estimate

A conceptual-level cost estimate was developed based on past project experience, correspondence with contractors, and material quantities and unit costs. A breakdown of the estimated conceptual level construction cost is provided in Appendix D, 15% Conceptual Construction Cost Estimate.

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Cowling Creek Culverts
Replacement at Miller Bay Road NE

APPENDIX

A

CONCEPT-LEVEL GEOTECHNICAL ENGINEERING REPORT

Concept-Level Geotechnical Engineering Report
Cowling Creek Culverts Replacement Feasibility Study
Kitsap County, Washington

July 6, 2017

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

CONCEPT-LEVEL GEOTECHNICAL ENGINEERING REPORT
COWLING CREEK CULVERTS REPLACEMENT FEASIBILITY STUDY
KITSAP COUNTY, WASHINGTON

1.0 INTRODUCTION

This report presents the results of our geologic site reconnaissance, geotechnical engineering analyses, and concept-level design recommendations for the Cowling Creek Culverts Replacement Feasibility Study. The purpose of the study was to review alternatives and select a preferred concept for the replacement of two salmon-blocking culverts on Cowling Creek under Miller Bay Road NE. Our services were performed in accordance with Cardno Task Order S&W-COW-2016-01 dated December 16, 2016.

2.0 SITE AND PROJECT DESCRIPTION

The project site is approximately 1¼ miles north of Suquamish, Washington, on the Kitsap Peninsula (Figure 1). Cowling Creek crosses underneath Miller Bay Road NE through two 36-inch-diameter concrete culverts and empties into the southern terminus of Miller Bay. The culverts were constructed in 1935 and are approximately 180 feet long and about 40 feet below existing road grade.

Mid Sound Fisheries Enhancement Group (MSFEG) received funding from the Salmon Recovery Funding Board to conduct a feasibility study of alternatives for the replacement of the existing culverts with a fish-passable structure. The roadway and culverts are owned and maintained by Kitsap County (County). The County requires that the roadway remain open to traffic during construction of the replacement alternative. The County also expressed interest in an alternative that would allow possible future widening of the alignment with a sidewalk and bike path.

3.0 GEOLOGIC SITE RECONNAISSANCE

Shannon & Wilson, Inc. (Shannon & Wilson) completed a geologic reconnaissance at the site on January 7, 2017. The roadway is a two-lane asphalt paved road with no sidewalks. The pavement and roadway safety barriers appear to be in good condition. The roadway fill embankment slopes vary between about 1.3 Horizontal to 1 Vertical (1.3H:1V) to 1.7H:1V and are covered with shrubs and small trees. A 10-inch-diameter corrugated metal pipe was observed in the west slope north of the culverts. Signs of deep-seated slope instability or soil

creep were not observed. Shallow erosion was locally observed on the east slope. Riprap was present at the toe of the east slope and around the culvert outlets.

Five shallow explorations (P-2, P-3, P-4, P-6, and P-7) were completed using hand auger and T-probe equipment as part of our geologic reconnaissance. The explorations extended about 2½ to 4½ feet below ground surface. We recorded preliminary classification and relative density of the soils in the explorations near the Cowling Creek stream bank and along the existing fill slopes on both the east and west sides of Miller Bay Road NE. The exploration locations are included in the Cardno Cowling Creek Topographic Survey in Appendix A. A description of the soils encountered is included in Section 4.2.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geologic Conditions

The Puget Sound area was subjected to six or more major glacial events over the past 100,000 years, each depositing new sediment and partially eroding previous sediments. During the intervening periods when glacial ice was not present, stream processes, wave action, weathering, and landsliding eroded and reworked some of the glacially derived sediment, further complicating the geologic setting.

During the most recent Vashon Stade of the Fraser Glaciation, the glacial ice is estimated to have been about 3,000 feet thick in the project area (Thorson, 1989). The weight of the glacial ice resulted in compaction and overconsolidation of the glacial and nonglacial sediments beneath the ice. As the Vashon ice retreated around 13,500 years ago, sand and gravel outwash and mixtures of sediments trapped in the ice were deposited. These recessional glacial deposits are overlain by softer and looser alluvial and colluvial deposits. Development and land use continues to modify the landscape in the Puget Sound.

4.2 Shallow Subsurface Conditions

In general, the surrounding hillslopes and stream bed are underlain by glacially overconsolidated silt, sand, and gravel. The existing fill embankment consists of loose to medium dense, silty, sandy gravel and sandy silt. Several exposures in the valley sidewalls and along the stream bed show gray, hard, gravelly, sandy silt that are consistent with mapped Quaternary Vashon Till (Qvt) and Quaternary Recessional Ice Contact deposits (Qvri). Haugerud and Troost (2011) mapped both deposits near the project area. Embankment fill appears to be reworked till and ice

contact deposits. Densities are based on T-probe measurements and should be considered approximate.

4.3 Groundwater Conditions

Auger explorations near the elevation of the creek (P-2 and P-6) show groundwater at or within a foot of ground surface. During our reconnaissance, stormwater runoff was observed flowing onto the east and west fill embankments from the roadway. Seepage or flowing water was not observed originating from within the fill embankment.

5.0 PRELIMINARY ALTERNATIVES ASSESSMENT

The project team evaluated the feasibility of three replacement alternatives for the site. The three replacement alternatives included:

- Sixteen- (16-) foot arch culvert installed by conventional tunneling methods.
- Fifty- (50-) foot-long single-span bridge supported by soldier pile and tieback wall abutments.
- Eight- (8-) foot-diameter circular culvert installed using jack-and-bore methods.

Shannon & Wilson developed concept-level geotechnical construction considerations and probable construction costs for the geotechnical aspects of each alternative. We provided our information to Cardno for inclusion in their Cowling Creek Culverts Replacement Feasibility Study report dated April 26, 2017. On May 15, 2017, Shannon & Wilson and Cardno presented our findings to MSFEG and the County. Based on subsequent discussions with MSFEG and the County, the bridge alternative was selected as the preferred construction option. The proposed bridge is approximately 50 feet long and 40 feet wide and supported at the abutments by a permanent soldier pile and tieback wall. Concept-level design details and construction sequencing is provided in the Cardno Feasibility Study report (2017).

6.0 CONCEPTUAL ENGINEERING ANALYSES AND RECOMMENDATIONS

We understand that the project will be designed in accordance with Washington State Department of Transportation (WSDOT) Bridge Design Manual (WSDOT, 2015a); the WSDOT Geotechnical Design Manual (GDM) (WSDOT, 2015b); and the American Association of State and Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2016). Construction of the new bridge requires seismic design parameters, deep foundation design, lateral earth pressures, and cut slope

recommendations. The following sections provide concept-level geotechnical design recommendations for the new bridge and permanent cut slopes.

6.1 Seismic Design Ground Motions

The computation of forces used for seismic design is based on seismological input and soil response factors. The seismological inputs are the peak ground acceleration, short-period spectral acceleration, and spectral acceleration at the 1-second period given in the table below. These inputs were determined for a ground motion level corresponding to a 7 percent probability of exceedance in 75 years, or about a 1,000-year return period.

The site soil response factors are based on the determination of the site class. The Washington Division of Geology and Earth Resources Site Class Map of Kitsap County, Washington (Palmer and others, 2004) shows that the site could be classified as Site Class C to D. Based on the near-surface soils encountered during our geologic reconnaissance, we recommend the site be considered Site Class C for conceptual design.

The design parameters listed in Table 1 below were calculated using the web-based tool provided by the U.S. Geological Survey (USGS, 2017).

TABLE 1
RECOMMENDED SEISMIC DESIGN PARAMETERS

Spectral Response Acceleration and Site Coefficients	
Peak Ground Acceleration, PGA	0.41g
Short-Period Spectral Acceleration, S_S	0.91g
Long-Period Spectral Acceleration, S_1	0.32g
Site Factor, F_{pga} *(Site Class C)*	1.00
Site Factor, F_a *(Site Class C)*	1.04
Site Factor, F_v *(Site Class C)*	1.48
Peak Design Spectral Acceleration, A_S *(Site Class C)*	0.41g
Short-Period Design Spectral Acceleration, S_{DS} *(Site Class C)*	0.95g
Long-Period Design Spectral Acceleration, S_{D1} *(Site Class C)*	0.47g

Subsurface explorations should be performed at the site to confirm the site class for final design. Earthquake-induced geologic hazards, including liquefaction and fault-related ground rupture, should be evaluated for final design.

6.2 Abutment Walls

The selected design alternative utilizes a permanent soldier pile and tieback wall that integrates deep foundation elements supporting the bridge deck with retaining walls to support the existing embankment fill. Due to the exposed height of the walls (approximately 40 feet), at least two rows of tiebacks will likely will be required to retain the embankment fill. The walls will provide excavation support while the existing fill below the bridge deck is excavated to the elevation of the new stream channel.

6.2.1 Lateral Earth Pressures

Lateral earth pressures against abutment walls depend on many factors, including retained soil, surcharges, drainage provisions, water pressure, construction method, and whether the wall can yield or deflect laterally or rotate at the top during or after excavation. For gravity retaining walls and cantilever soldier pile walls that are allowed to move at least 0.001 times the wall height, we recommend that a static, active lateral earth pressure be used. For gravity retaining walls and cantilever soldier pile walls that are not allowed to move 0.001 times the wall height, static, at-rest lateral earth pressures should be used. For soldier pile walls with tiebacks, apparent lateral earth pressures should be used.

For the seismic condition, the seismic lateral earth pressures provided are consistent with a pseudo-static analysis using the Mononobe-Okabe equation for lateral earth pressures. The bridge abutment walls are considered nonyielding in accordance with Section 11.6.5.4 of AASHTO (2016) and are based on a horizontal seismic coefficient equal to the site design spectral acceleration of 0.41g. For walls that are allowed to move at least 0.001 times the wall height during and after seismic shaking, the seismic lateral earth pressures provided are based on a horizontal seismic coefficient equal to one-half the site design spectral acceleration.

Figure 2 presents our recommended static and seismic lateral earth pressure equivalent fluid weights and distributions for the proposed soldier pile walls. The lateral earth pressure recommendations assume the ground surface behind the wall is level, the groundwater table is below the base of the wall, and that water in the retained embankment fill is free to escape from behind the wall.

Lateral earth pressures due to surcharge loads, such as traffic or construction equipment, should be added to the recommended lateral earth pressures. Static lateral pressures as a result of surcharge loads may be evaluated based on the diagrams presented in Figure 3. Seismic lateral

pressures as a result of surcharge loads should be applied as recommended in the AASHTO and the WSDOT Bridge Design Manual.

6.2.2 Soldier Piles

Vertical members for the soldier pile wall consist of steel sections placed in predrilled holes. The soldier piles should be designed to resist the total vertical component of the tieback anchor forces as well as the loads imposed by the bridge. Vertical soldier pile capacities below finished grade can be evaluated from the skin friction and end-bearing pressures given in Table 2. Penetration depths below finished grade should be adequate for kick-out resistance.

6.2.3 Tieback Anchors

The frictional resistance of an anchor depends on many factors, including the selected Contractor's method and care of installation. The length of production anchors should be based on a series of test anchors. For conceptual design, we recommend a preliminary nominal (unfactored) load transfer rate of 4 kips per lineal foot (klf) for a 6- to 8-inch-diameter borehole and a bond zone in the embankment fill. For anchor bond zones in the glacially overridden soil anticipated beneath the fill, we recommend a preliminary nominal (unfactored) load transfer rate of 10 klf. These load transfer rates are for a single-stage pressure-grouted anchor. AASHTO (2016) recommends using a resistance factor (RF) of 0.65 for tieback bond resistance in granular soil. AASHTO (2016) allows the RF to be increased to 1.0 if anchor proof tests are performed on every production anchor. We recommend tieback anchors be spaced a minimum of 3 diameters apart, measured from the edge of the borehole.

6.2.4 Lagging

Lagging for the soldier pile wall system should be installed as the excavation proceeds and, in general, not more than 4 feet (measured vertically) of unsupported excavation should be exposed at one time. The actual height of vertical, unsupported excavation may vary depending on results from a subsurface exploration program and the encountered soil conditions during excavation.

Due to arching between soldier piles, a reduced lateral earth pressure could be used for lagging design. We recommend lagging be designed following the guidelines presented in AASHTO (2016) Article C11.8.5.2. Weep holes, geocomposite drain boards, or other means should be incorporated into the permanent wall design to prevent the buildup of hydrostatic pressures behind the wall.

6.3 Drilled H-Piles

The proposed bridge will be supported by drilled H-Piles that serve as the structural elements for the soldier pile abutment wall. WSDOT (2015b) requires permanent soldier piles be installed using drilled methods. Soldier piles will be installed from the current roadway elevation, through the embankment fill, and terminate in the anticipated underlying glacial deposits. The borehole should be filled with structural concrete from its base to the bottom elevation of the wall. The remainder of the borehole should be filled with a lean mix concrete that can be chipped away during installation of lagging and tiebacks.

6.3.1 Axial Resistance

The drilled H-Pile elements will derive axial capacity from the anticipated underlying glacial soils (Qvt/Qvri). Axial resistance from the embankment fill should be ignored. The axial resistance will vary with foundation penetration and size, subsurface conditions, and installation techniques. We assumed a diameter of 4 feet for conceptual engineering analysis.

We performed axial analysis for the conceptual drilled H-Piles using our in-house spreadsheet. We estimated nominal unit skin friction and unit end bearing values for the glacial soils located below the creek elevation. For conceptual design, we assumed the glacial soils to be very dense (i.e., Standard Penetration Test value of 50 blows per foot). Our recommended deep foundation properties for conceptual bridge design are presented in Table 2.

TABLE 2
DEEP FOUNDATION DESIGN SOIL PROPERTY SUMMARY

Material	Unit Weight (lb/ft^3)	k tanδ	Q_u (tsf)
Qvt / Qvri	130	0.32	30

Notes:
lb/ft^3 = pounds per cubic foot
tsf = tons per square foot

Recommended axial resistance for individual drilled H-Piles are presented in Figure 4. These recommendations are presented as plots of estimated axial resistances versus depth below creek elevation. For the drilled H-Piles, estimated resistances are provided for the service limit state (settlement of 0.5 and 1 inch), the strength limit state, and the extreme limit state. Scour was not considered in our analysis.

The LRFD bridge design manual indicates that a group reduction factor should be applied to single element nominal resistance when drilled element spacing within a single row is closer than 3 diameters center to center. Soldier piles are typically spaced 4 to 8 feet on center. We

recommend that the appropriate group reduction factor be applied when final spacing is determined.

6.4 Permanent Cut Slopes

We recommend the surrounding stream channel slopes be designed and graded at a slope no steeper than 2H:1V. For cut slopes higher than 10 feet, WSDOT requires a stability analysis to account for soil characteristics, adjacent loads, and groundwater conditions. We recommend that a global stability analysis be performed to determine the impact of any alternations to the surrounding slopes.

If slopes steeper than 2H:1V are designed, we recommend using a slope stabilization method such as soil nails or Spiralnails to improve slope stability and reduce shallow failures. Based on our experience with similar anticipated soil types and the lack of established access to the slopes surrounding the project area, the Spiralnail system may be a cost effective and less intrusive solution.

Spiralnails are a proprietary system of driven soil nails developed by Hilfiker Retaining Walls. Spiralnails consist of 2-inch-diameter square steel tubing that has one spiral revolution every 4 feet. The nails are driven into place with an excavator-mounted hammer and locked for rotation with a steel mesh facing or steel “spider” hubs. The mesh facing or “spider” hubs help retain soil between the nails, reducing the potential for shallow failures. Permanent erosion control and vegetation should be used in combination with the mesh facing and hubs. Once locked and prevented from rotating, the nail-to-soil adhesion of Spiralnails is similar to conventional soil nails. Nail spacing depends on soil characteristics, but typically varies about 4 to 6 feet, measured center to center.

Spiralnail installation using an excavator can be done during grading of the slope and would not require a separate access road to be constructed. If this option is selected, we recommend a test nail program be conducted to determine the pullout capacity of the nails for local soil types prior to design.

7.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

Design and construction considerations for the bridge alternative will likely include:

- **Tieback Anchors:** Design of the abutment walls will likely require tiebacks at both the front and side faces of the wall. Tiebacks should be designed and installed so as not to interfere with the zone of influence for each anchor. Tiebacks installed on the

parallel wall segments may be drilled through the existing embankment fill and secured using walers installed on the face of each wall to reduce the number of anchors.

- **Creek Diversion:** A cofferdam or other water diversion method will be required to channel flow of the existing creek during construction. Work in and around the creek area will require special permits and be restricted to certain months for fish migration activities.
- **Utilities:** Temporary and permanent relocation of existing underground and overhead utilities may be required.
- **Access:** There is minimal public road access to Cowling Creek west of Miller Bay Road NE and no access to the east. Construction will require building temporary access roads and/or obtaining rights of access from private landowners.
- **Construction Sequencing:** The County's requirement that one lane of traffic be maintained during construction requires careful consideration of sequencing. A lack of available space adjacent to the project area limits equipment and material laydown and storage at the site. This may require bridge segments to be trucked separately and result in delays that may impact other aspects of construction.

8.0 LIMITATIONS

This report was prepared for the exclusive use of Cardno and MSFEG for specific application to the concept-level design for the project at this site as it relates to the geotechnical aspects discussed in this report. Our conclusions and recommendations are intended for concept-level design only and should not be used for final design or construction. This report should be relied on for factual data only. The interpretations, recommendations, and conclusions presented in this report should not be construed as a warranty of surficial or subsurface conditions, and should not be relied upon by prospective contractors.

Within the limitations of the scope, schedule, and budget, the interpretations, recommendations, and conclusions presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they existed during our site reconnaissance and further assume that the shallow explorations are representative of the subsurface conditions throughout the project site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the shallow explorations. Our conclusions and recommendations are based on our understanding of

the project as described in this report and the site conditions as interpreted from the geologic reconnaissance and shallow explorations.

If there is substantial lapse of time between the submission of this report and final design for the project, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations concerning the changed conditions or time lapse.

The scope of our services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or around the site. Shannon & Wilson has qualified personnel to assist you with these services should they be necessary. We have prepared Appendix B, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our report.

SHANNON & WILSON, INC.

Brian S. Reznick, PE
Associate

AJD:BSR:SRB/ajd

9.0 REFERENCES

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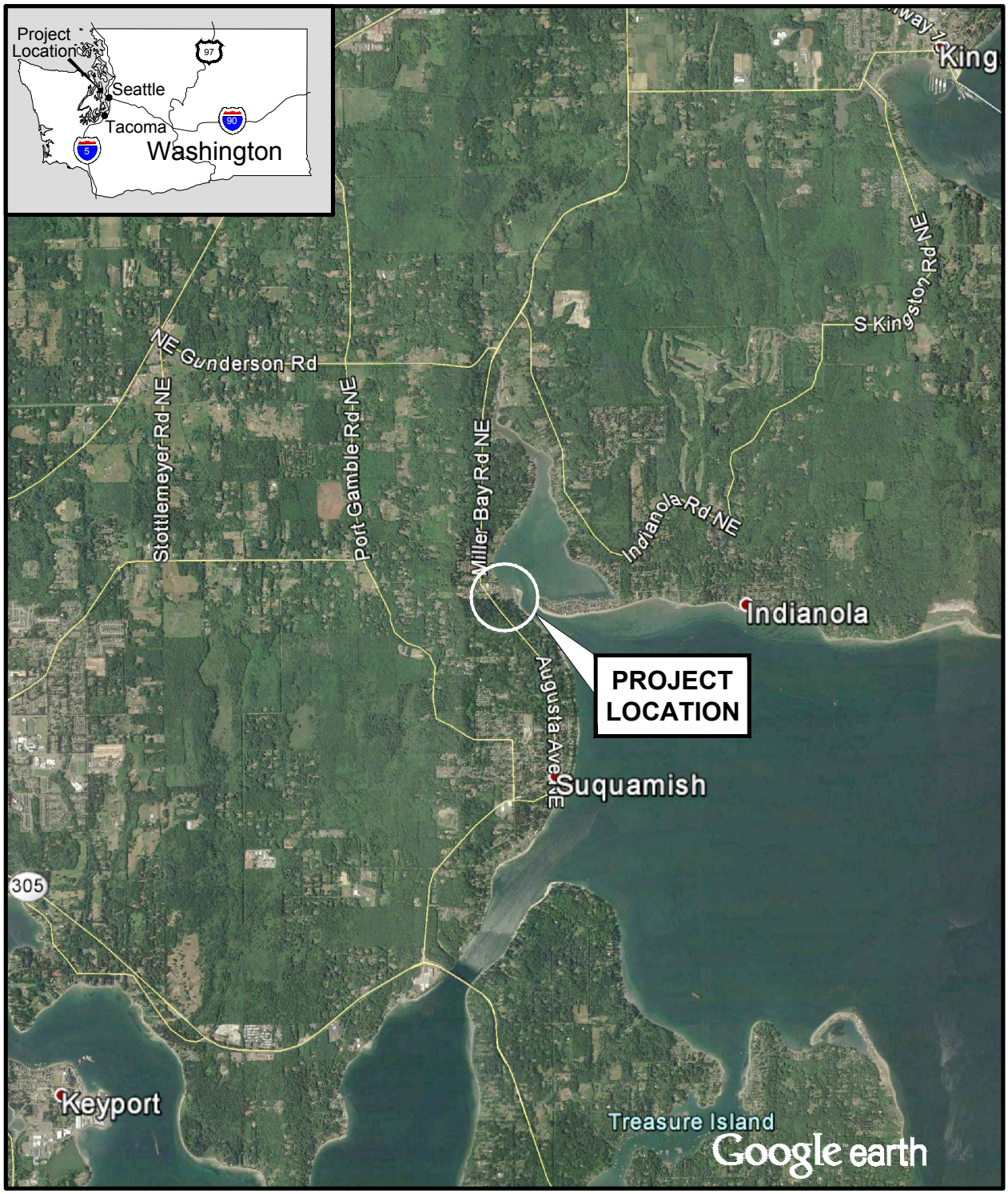

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

0 1 2

Approximate Scale in Miles

Cowling Creek Culverts Replacement
Feasibility Study
Kitsap County, WA

VICINITY MAP

July 2017 21-1-22317-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

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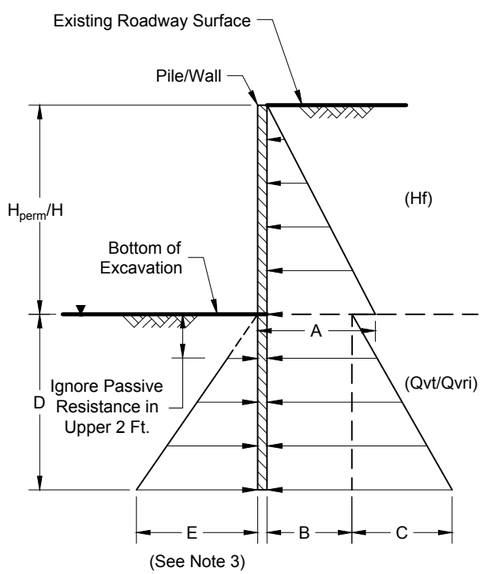
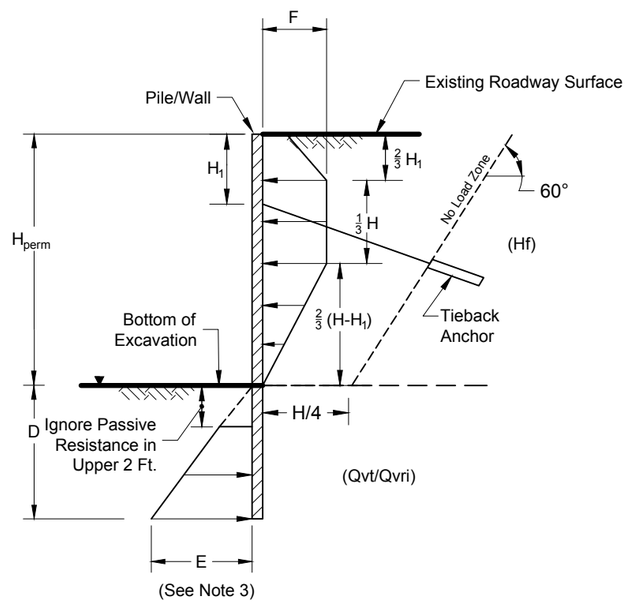
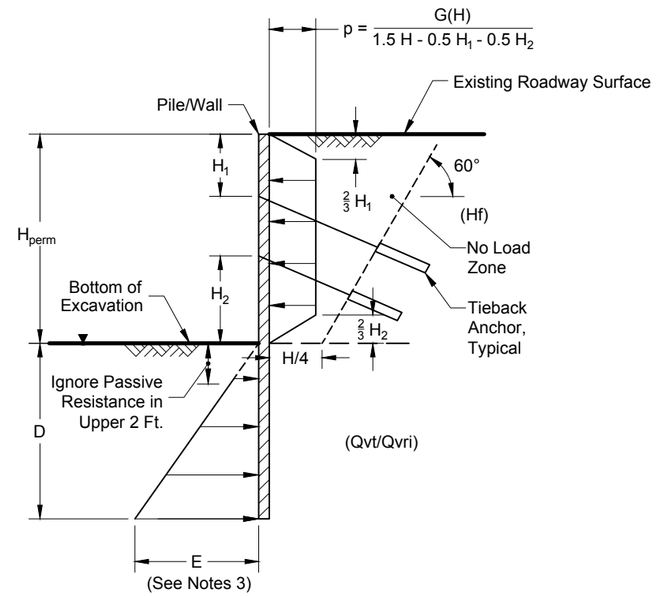
Recommended Earth Pressures for Cantilever Soldier Pile Walls and During Excavation Prior to Tieback Installation



Recommended Earth Pressures for Single Tieback Soldier Pile Walls



Recommended Earth Pressures for Multiple Tieback Soldier Pile Walls



NOTES

1. All earth pressures are in units of pounds per square foot per foot of wall width. The total earth pressure is the sum of either the cantilever condition, single tieback, or multiple tieback pressure, and the additional earth pressures due to surcharge loading as shown on Figure 3.

2. Wall Embedment (D) should consider kickout resistance. Embedment should be determined by satisfying horizontal equilibrium about the bottom of the pile. D should not be less than 8 feet.

3. A resistance factor of 0.75 should be used for passive resistance for static and seismic loading conditions (AASHTO LRFD Table 11.5.7-1).

4. Apply active, at-rest, and surcharge pressures over the width of the soldier piles below the base of the lagging and apply passive resistance over twice the width of the piles or the spacing of the piles, whichever is smaller.

5. Design lagging in accordance with the guidelines presented in AASHTO Article C11.8.5.2.

6. Estimated lateral earth pressures presented here are for conceptual design only, and based on geologic site reconnaissance and geologic maps. Subsurface explorations and further geotechnical evaluation is needed for final design.

7. The lateral earth pressures assume no scour or that the walls will be protected from scour. This should be reviewed for final design.

8. Lateral earth pressure parameters:

Soil Unit	Hf	Qvt/Qvri
K_a	0.30	0.23
K_o	0.50	0.40
γ_m(pcf)	120	-
γ_b(pcf)	-	67.6

pcf = pounds per cubic foot

9. Cantilever Soldier Pile Wall

Load Type	A	B	C	E
Static (active)	36H	28H	16D	-
Static (at-rest)	60H	48H	27D	-
Static (active) + Seismic	55H	28H	16D	-
Static (at-rest) + Seismic	102H	48H	27D	-
Static (passive)	-	-	-	320D
Static (passive) + Seismic[1]	-	-	-	270D
Static (passive) + Seismic[2]	-	-	-	220D

Notes: 1. Use with active condition.
2. Use with at-rest condition.

10. Single Tieback and Multiple Tieback Soldier Pile Wall

Load Type	F	G	E
Static	36H	36H	320D
Static + Seismic	Use Cantilever Soldier Pile Wall Figure and Static (active) + Seismic and Static (passive) + Seismic[1]		

11. Recommended ultimate load transfer for 6- to 8-inch diameter single-stage pressure grouted anchors is 10 kips/ft in dense glacially overridden soil, and 4 kips/ft for the existing embankment fill. A resistance factor of 0.65 should be applied to this preliminary ultimate load transfer rate. Anchor pullout resistance should be confirmed by field verification testing.

12. Temporary cantilever condition should be evaluated for pre-tieback installation condition.

13. Lateral earth pressures presented in this figure are applicable for walls that retain flat ground and have flat ground in front of them. Walls with backslopes sloping toward the wall will have higher lateral earth pressures. Soil that slopes downward away from the wall toe will offer lower passive earth resistance.

LEGEND

H = Temporary Wall Height During Construction (Ft.)

H_{perm} = Permanent Condition Wall Height (Ft.)

H_1 = Depth to Uppermost Tieback Level (Ft.)

H_2 = Distance from base of excavation to lowermost tieback level (Ft.)

D = Embedment Depth (Ft.)

γ_m = Moist soil unit weight

γ_b = Bouyant soil unit weight

Cowling Creek Culverts Replacement
Feasibility Study
Kitsap County, Washington

LATERAL EARTH PRESSURES FOR PERMANENT SOLDIER PILE WALLS

July 2017 21-1-22317-001

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2

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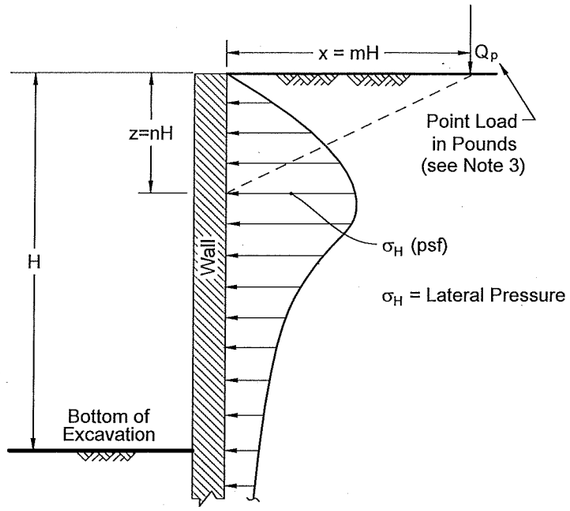
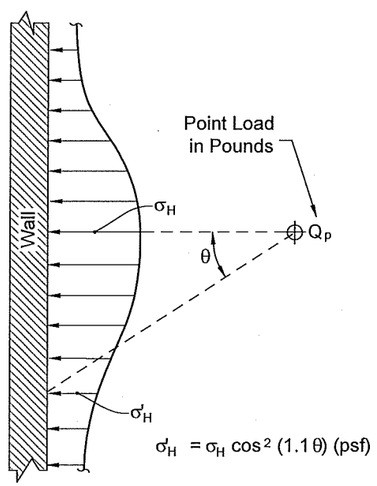


ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.28 \frac{Q_p}{H^2} \frac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

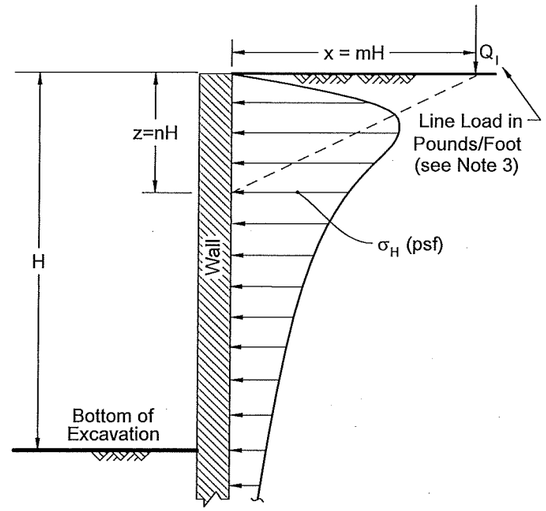
For $m > 0.4$: $\sigma_H = 1.77 \frac{Q_p}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)



PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



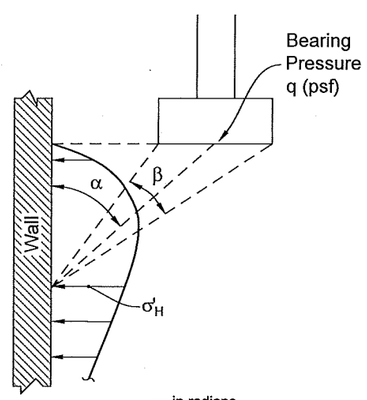
ELEVATION VIEW

For $m \le 0.4$: $\sigma_H = 0.20 \frac{Q_l}{H} \frac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.28 \frac{Q_l}{H} \frac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

(NAVFAC DM 7.2, 1986)

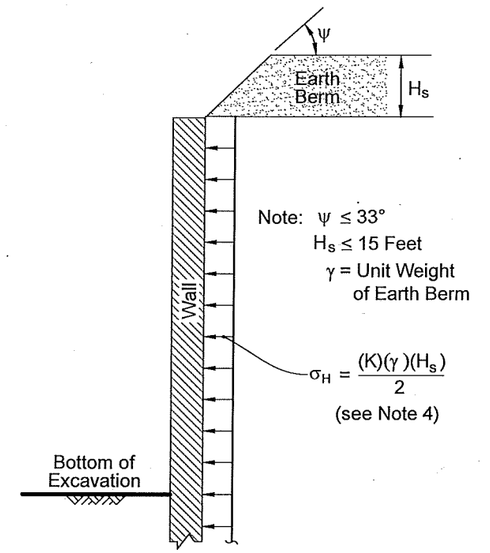


in radians

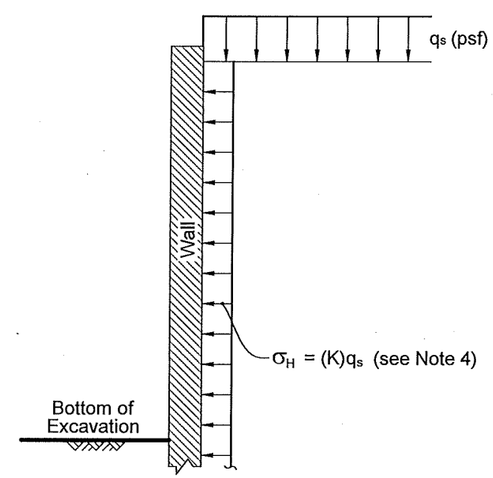
$\sigma_H = \frac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$ (psf)

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



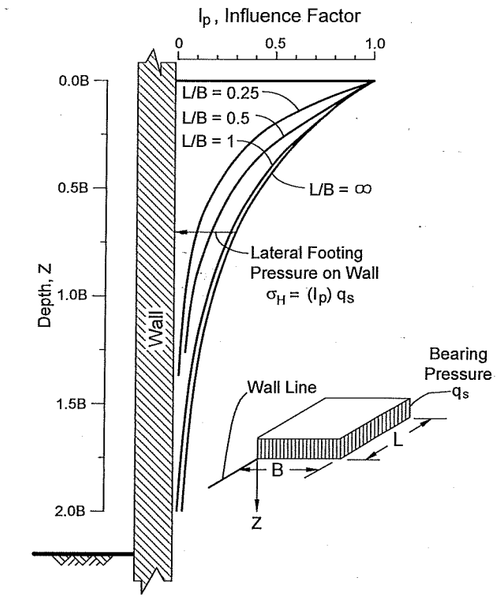
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulos and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT FOOTING

(derived from NAVFAC DM 7.2, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that $m \le 0.4$, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. See individual lateral earth pressure figures for recommended K values.

Cowling Creek Culverts Replacement
Feasibility Study
Kitsap County, Washington

RECOMMENDED SURCHARGE LOADING FOR PERMANENT WALLS

July 2017 | 21-1-22317-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

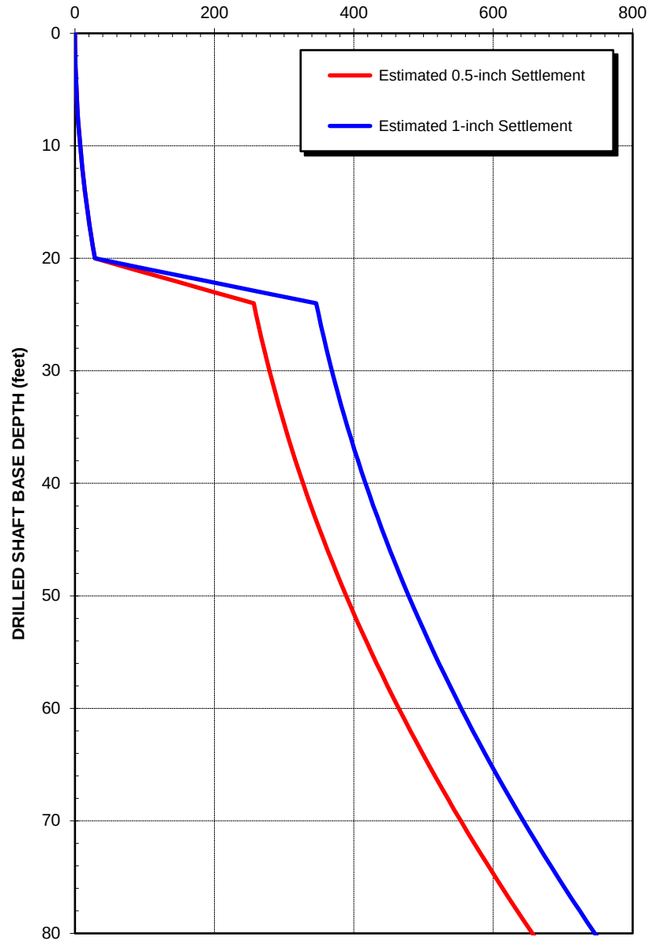
FIG. 3

File: J:\21\22317\21-1-22317-001 SL.dwg Date: 07-03-2017 Author: mjm

ASSUMED SUBSURFACE PROFILE

SERVICE LIMIT

NOMINAL RESISTANCE (tons)

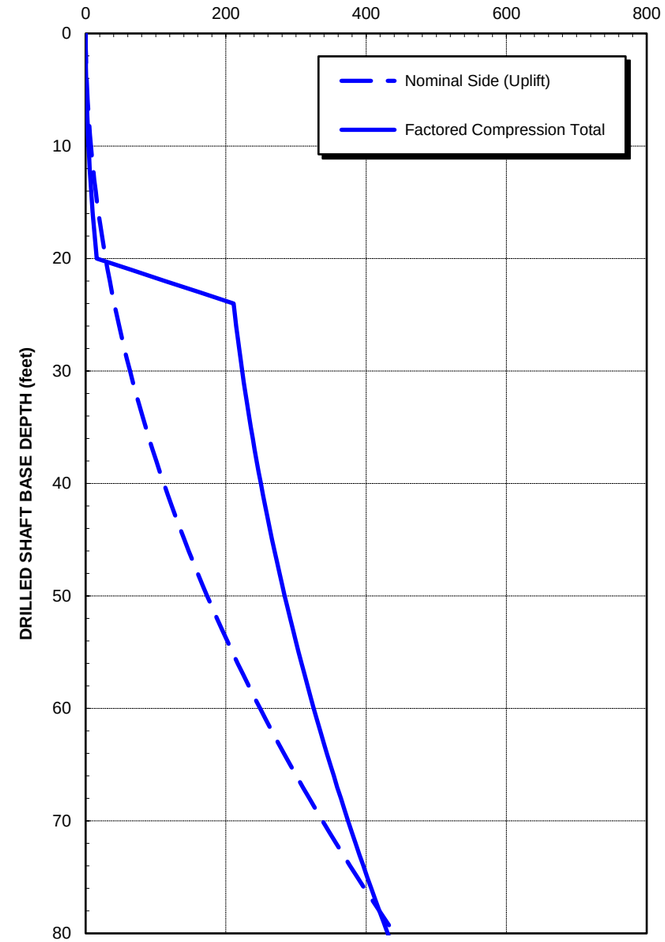


SERVICE LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

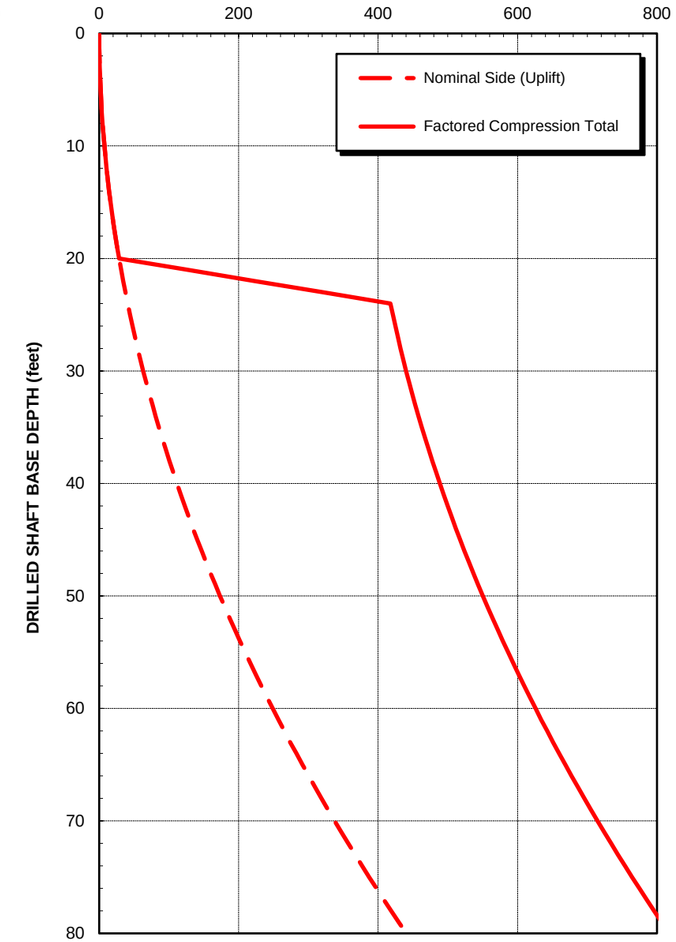


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per WSDOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per WSDOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per WSDOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. Estimated axial resistance presented here is for conceptual design only, and based on geologic site reconnaissance and geologic maps. Subsurface explorations and further geotechnical evaluation is needed for final design.
2. The analyses were performed based on guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
3. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
4. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.
5. The axial resistance plots assume no scour or that the foundations will be protected from scour. This assumption should be reviewed for final design.
6. Downdrag loads due to the potential for liquefaction or embankment settlement have not been analyzed, and should be evaluated for final design.

Cowling Creek Culverts Replacement
Feasibility Study
Kitsap County, WA

ESTIMATED AXIAL RESISTANCE FOR A 4-FOOT DIA. DRILLED H-PILE

July 2017 21-1-22317-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4

APPENDIX A

CARDNO TOPOGRAPHICAL SURVEY OF COWLING CREEK PROJECT SITE

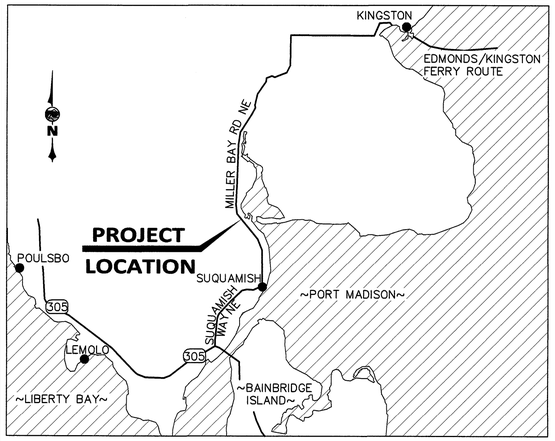
CARDNO

COWLING CREEK TOPO

DATUM: NAVD88

Call 811 two business days before you dig

VICINITY MAP - NOT TO SCALE

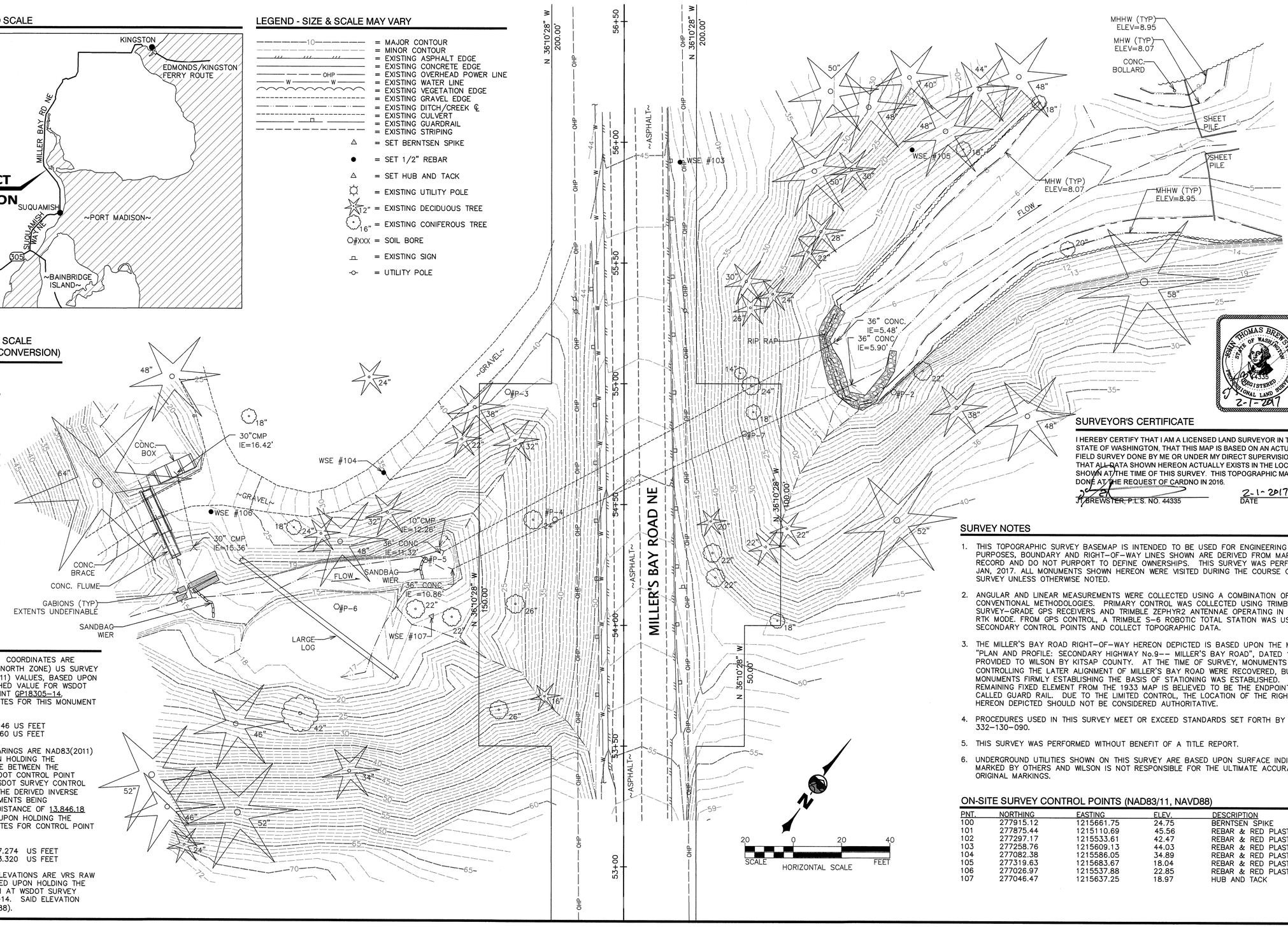


LEGEND - SIZE & SCALE MAY VARY

- = MAJOR CONTOUR
- = MINOR CONTOUR
- = EXISTING ASPHALT EDGE
- = EXISTING CONCRETE EDGE
- = EXISTING OVERHEAD POWER LINE
- = EXISTING WATER LINE
- = EXISTING VEGETATION EDGE
- = EXISTING GRAVEL EDGE
- = EXISTING DITCH/CREEK ℄
- = EXISTING CULVERT
- = EXISTING GUARDRAIL
- = EXISTING STRIPING
- = SET BERNTSEN SPIKE
- = SET 1/2" REBAR
- = SET HUB AND TACK
- = EXISTING UTILITY POLE
- = EXISTING DECIDUOUS TREE
- = EXISTING CONIFEROUS TREE
- #XXX = SOIL BORE
- = EXISTING SIGN
- = UTILITY POLE

DATUM CONVERSION SCALE (PER NOAA VDATUM CONVERSION)

mhhw =	+8.95'
mhw =	+8.07'
mtl =	+4.30'
NGVD29 =	+3.52'
mlw =	+0.53'
NAVD88 =	0.00'
mllw =	-2.28'
calc elw =	-6.78'



JOHN THOMAS BREWSTER STATE OF WASHINGTON 44335 REGISTERED PROFESSIONAL LAND SURVEYOR 2-1-2017

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT I AM A LICENSED LAND SURVEYOR IN THE STATE OF WASHINGTON, THAT THIS MAP IS BASED ON AN ACTUAL FIELD SURVEY DONE BY ME OR UNDER MY DIRECT SUPERVISION AND THAT ALL DATA SHOWN HEREON ACTUALLY EXISTS IN THE LOCATIONS SHOWN AT THE TIME OF THIS SURVEY. THIS TOPOGRAPHIC MAP WAS DONE AT THE REQUEST OF CARDNO IN 2016.

JT BREWSTER, P.L.S. NO. 44335 — DATE 2-1-2017

SURVEY NOTES

1. THIS TOPOGRAPHIC SURVEY BASEMAP IS INTENDED TO BE USED FOR ENGINEERING DESIGN PURPOSES, BOUNDARY AND RIGHT-OF-WAY LINES SHOWN ARE DERIVED FROM MAPS OF RECORD AND DO NOT PURPORT TO DEFINE OWNERSHIPS. THIS SURVEY WAS PERFORMED IN JAN, 2017. ALL MONUMENTS SHOWN HEREON WERE VISITED DURING THE COURSE OF THIS SURVEY UNLESS OTHERWISE NOTED.
2. ANGULAR AND LINEAR MEASUREMENTS WERE COLLECTED USING A COMBINATION OF GPS AND CONVENTIONAL METHODOLOGIES. PRIMARY CONTROL WAS COLLECTED USING TRIMBLE R7 SURVEY-GRADE GPS RECEIVERS AND TRIMBLE ZEPHYR2 ANTENNAE OPERATING IN NETWORKED RTK MODE. FROM GPS CONTROL, A TRIMBLE S-6 ROBOTIC TOTAL STATION WAS USED TO TIE SECONDARY CONTROL POINTS AND COLLECT TOPOGRAPHIC DATA.
3. THE MILLER'S BAY ROAD RIGHT-OF-WAY HEREON DEPICTED IS BASED UPON THE MAP OF THE "PLAN AND PROFILE: SECONDARY HIGHWAY No.9-- MILLER'S BAY ROAD", DATED 1933 AND PROVIDED TO WILSON BY KITSAP COUNTY. AT THE TIME OF SURVEY, MONUMENTS CONTROLLING THE LATER ALIGNMENT OF MILLER'S BAY ROAD WERE RECOVERED, BUT NO MONUMENTS FIRMLY ESTABLISHING THE BASIS OF STATIONING WAS ESTABLISHED. THE ONLY REMAINING FIXED ELEMENT FROM THE 1933 MAP IS BELIEVED TO BE THE ENDPOINTS OF THE CALLED GUARD RAIL. DUE TO THE LIMITED CONTROL, THE LOCATION OF THE RIGHT-OF-WAY HEREON DEPICTED SHOULD NOT BE CONSIDERED AUTHORITATIVE.
4. PROCEDURES USED IN THIS SURVEY MEET OR EXCEED STANDARDS SET FORTH BY WAC 332-130-090.
5. THIS SURVEY WAS PERFORMED WITHOUT BENEFIT OF A TITLE REPORT.
6. UNDERGROUND UTILITIES SHOWN ON THIS SURVEY ARE BASED UPON SURFACE INDICATORS MARKED BY OTHERS AND WILSON IS NOT RESPONSIBLE FOR THE ULTIMATE ACCURACY OF THE ORIGINAL MARKINGS.

CONTROL NOTES

BASIS OF COORDINATES: COORDINATES ARE WASHINGTON STATE (NORTH ZONE) US SURVEY FOOT GRID NAD83(2011) VALUES, BASED UPON HOLDING THE PUBLISHED VALUE FOR WSDOT SURVEY CONTROL POINT GP18305-14. PUBLISHED COORDINATES FOR THIS MONUMENT ARE AS FOLLOWS:

NORTHING = 265,316.946 US FEET
EASTING = 1,212,691.860 US FEET

BASIS OF BEARINGS: BEARINGS ARE NAD83(2011) VALUES, BASED UPON HOLDING THE GPS-DERIVED INVERSE BETWEEN THE AFOREMENTIONED WSDOT CONTROL POINT GP18305-14, AND WSDOT SURVEY CONTROL MONUMENT "SQU". THE DERIVED INVERSE BETWEEN SAID MONUMENTS BEING N 31° 08' 44" E, A DISTANCE OF 13,846.18 FEET (GRID), BASED UPON HOLDING THE FOLLOWING COORDINATES FOR CONTROL POINT "SQU":

NORTHING = 277,167.274 US FEET
EASTING = 1,219,853.320 US FEET

BASIS OF ELEVATIONS: ELEVATIONS ARE VRS RAW NAVD88 VALUES BASED UPON HOLDING THE PUBLISHED ELEVATION AT WSDOT SURVEY MONUMENT GP18305-14. SAID ELEVATION BEING: 75.77' (NAVD88).

ON-SITE SURVEY CONTROL POINTS (NAD83/11, NAVD88)

PNT.	NORTHING	EASTING	ELEV.	DESCRIPTION
100	277915.12	1215661.75	24.75	BERNTSEN SPIKE
101	277875.44	1215110.69	45.56	REBAR & RED PLASTIC CAP
102	277297.17	1215533.61	42.47	REBAR & RED PLASTIC CAP
103	277258.76	1215609.13	44.03	REBAR & RED PLASTIC CAP
104	277082.38	1215586.05	34.89	REBAR & RED PLASTIC CAP
105	277319.63	1215683.67	18.04	REBAR & RED PLASTIC CAP
106	277026.97	1215537.88	22.85	REBAR & RED PLASTIC CAP
107	277046.47	1215637.25	18.97	HUB AND TACK

WILSON ENGINEERING, LLC
805 DUPONT STREET
BELLINGHAM, WA 98225
(360) 733-6100 • FAX (360) 647-9061
www.wilsonengineering.com
Wilson SURVEY/ENGINEERING

DESIGNED BY
DRAWN BY CLM
CHECKED BY

CARDNO

COWLING CREEK TOPOGRAPHIC SURVEY

KITSAP COUNTY WASHINGTON

DATE 02-01-2017
SCALE AS SHOWN
JOB NUMBER 2016-150
SHEET 1
PAGE 1 OF 1

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Cowling Creek Culverts
Replacement at Miller Bay Road NE

APPENDIX B

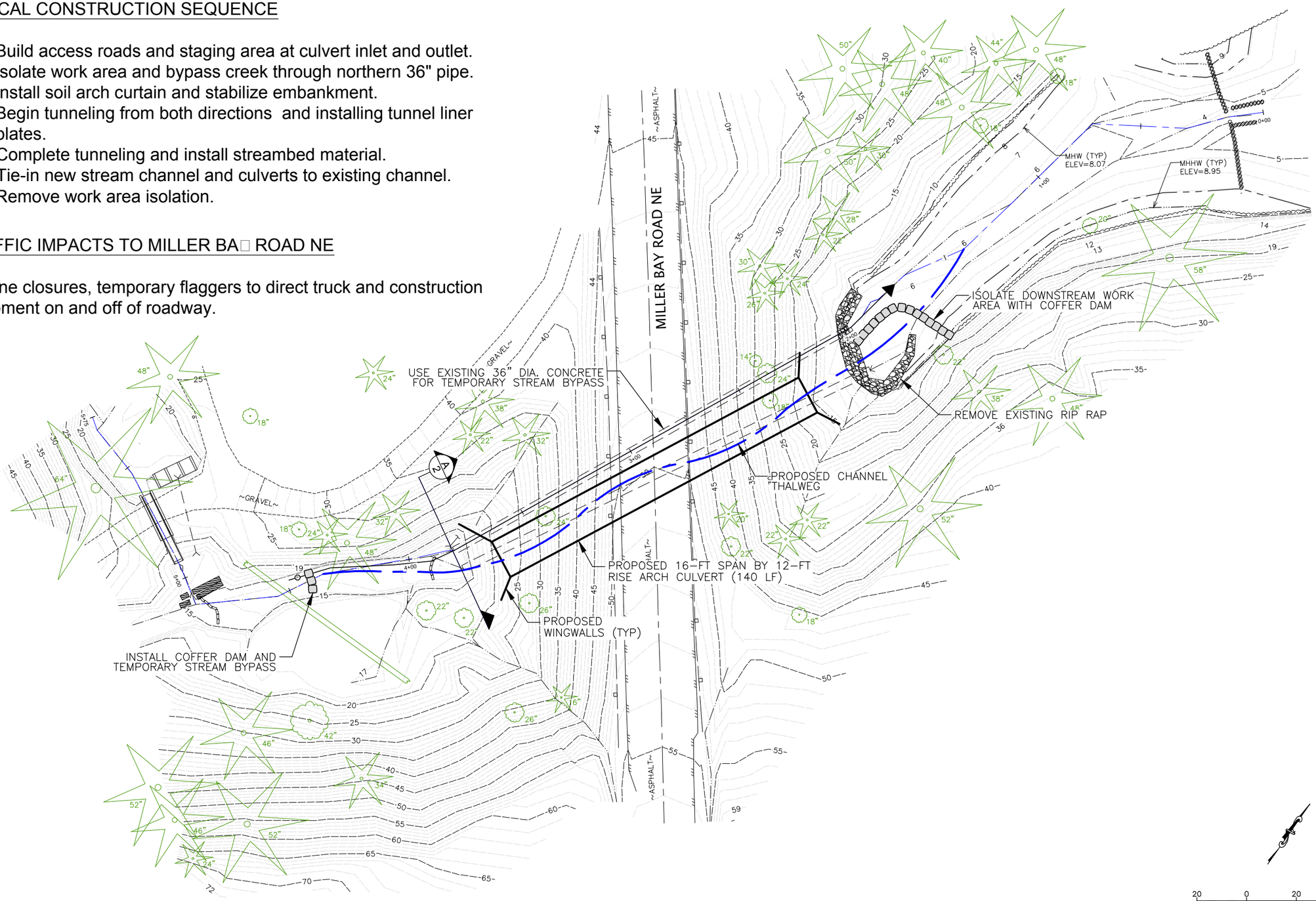
CONCEPTUAL CULVERT REPLACEMENT OPTIONS FIGURES

T□PICAL CONSTRUCTION SEQUENCE

1. Build access roads and staging area at culvert inlet and outlet.
3. Isolate work area and bypass creek through northern 36" pipe.
4. Install soil arch curtain and stabilize embankment.
5. Begin tunneling from both directions and installing tunnel liner plates.
6. Complete tunneling and install streambed material.
7. Tie-in new stream channel and culverts to existing channel.
8. Remove work area isolation.

TRAFFIC IMPACTS TO MILLER BA□ ROAD NE

No lane closures, temporary flaggers to direct truck and construction equipment on and off of roadway.



DATE	RE ISIONS	NO.

SEAL

Cardno
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801 SECOND A E, SUITE 700
SEATTLE, WA 98104
(206) 269-0104

MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

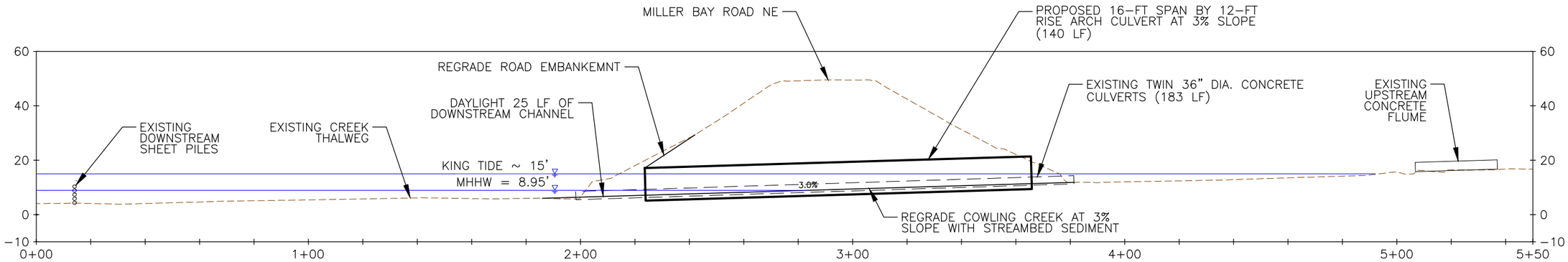
CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

OPTION 1 - 16-FT CUL ERT

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

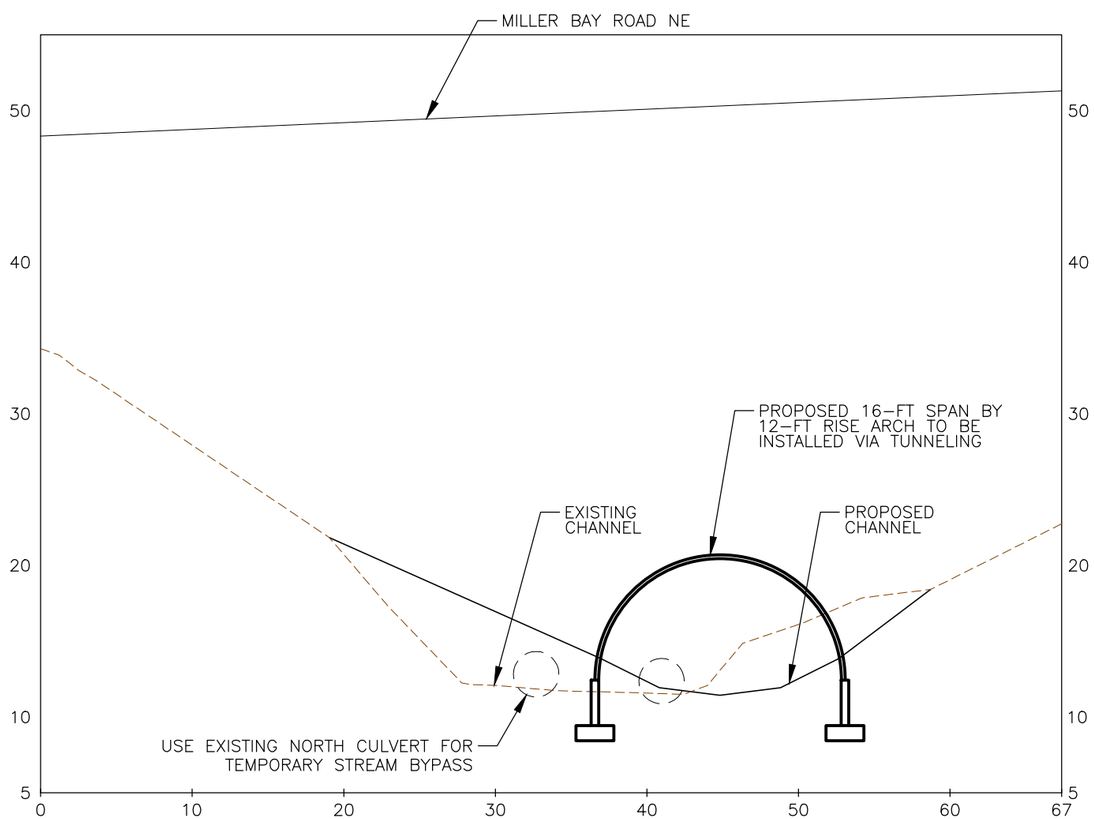
DATE:	FEBRUAR , 2017
DESIGNED B :	SM, JS
DRAWN B :	JS
CHECKED B :	SM
CARDNO JOB NO.	E316301200
SHEET NO.	1



1 - **COWLING CREEK PROFILE**
SCALE: AS SHOWN

SCALE 20 0 20 40 FEET HORIZONTAL

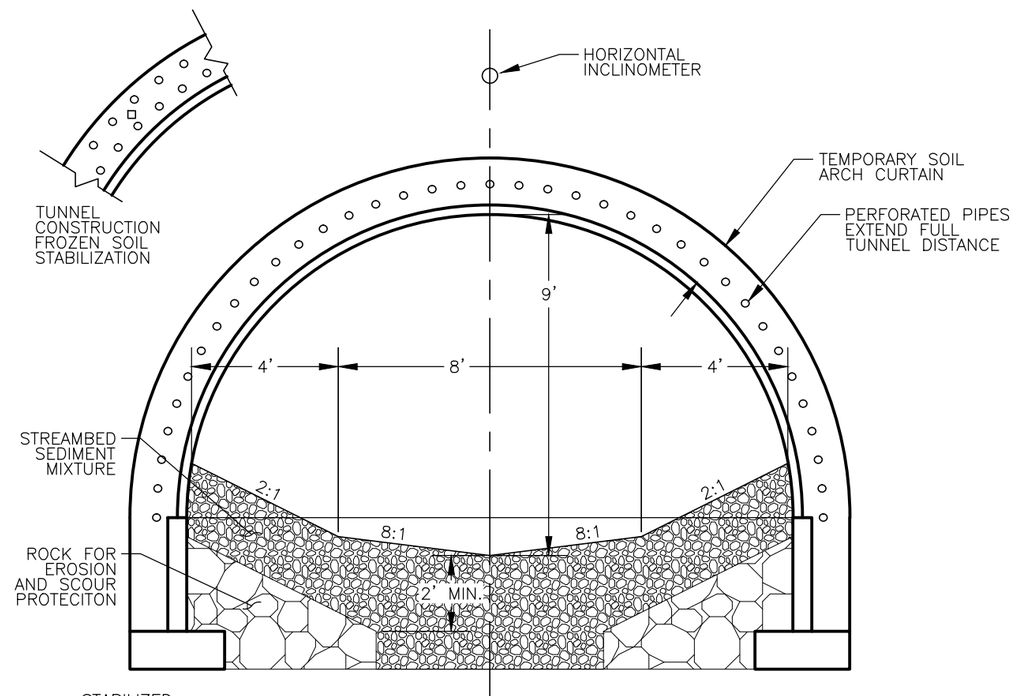
SCALE 20 0 20 40 FEET VERTICAL



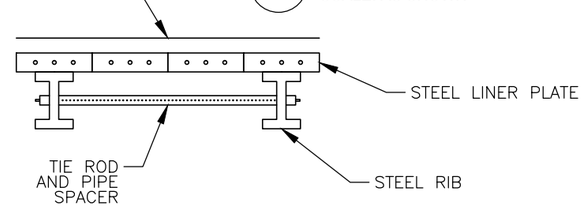
A - 1 **CULVERT INLET (LOOKING DOWNSTREAM)**
SCALE: AS SHOWN

SCALE 5 0 5 10 FEET HORIZONTAL

SCALE 5 0 5 10 FEET VERTICAL



2 - **TYPICAL CULVERT CHANNEL SECTION**
SCALE: AS SHOWN



SCALE 2 0 2 4 FEET HORIZONTAL

SCALE 2 0 2 4 FEET VERTICAL

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CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

OPTION 1 - PROFILE AND SECTION

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

DATE:	FEBRUAR , 2017
DESIGNED B :	SM, JS
DRAWN B :	JS
CHECKED B :	SM
CARDNO JOB NO.	E316301200
SHEET NO.	2

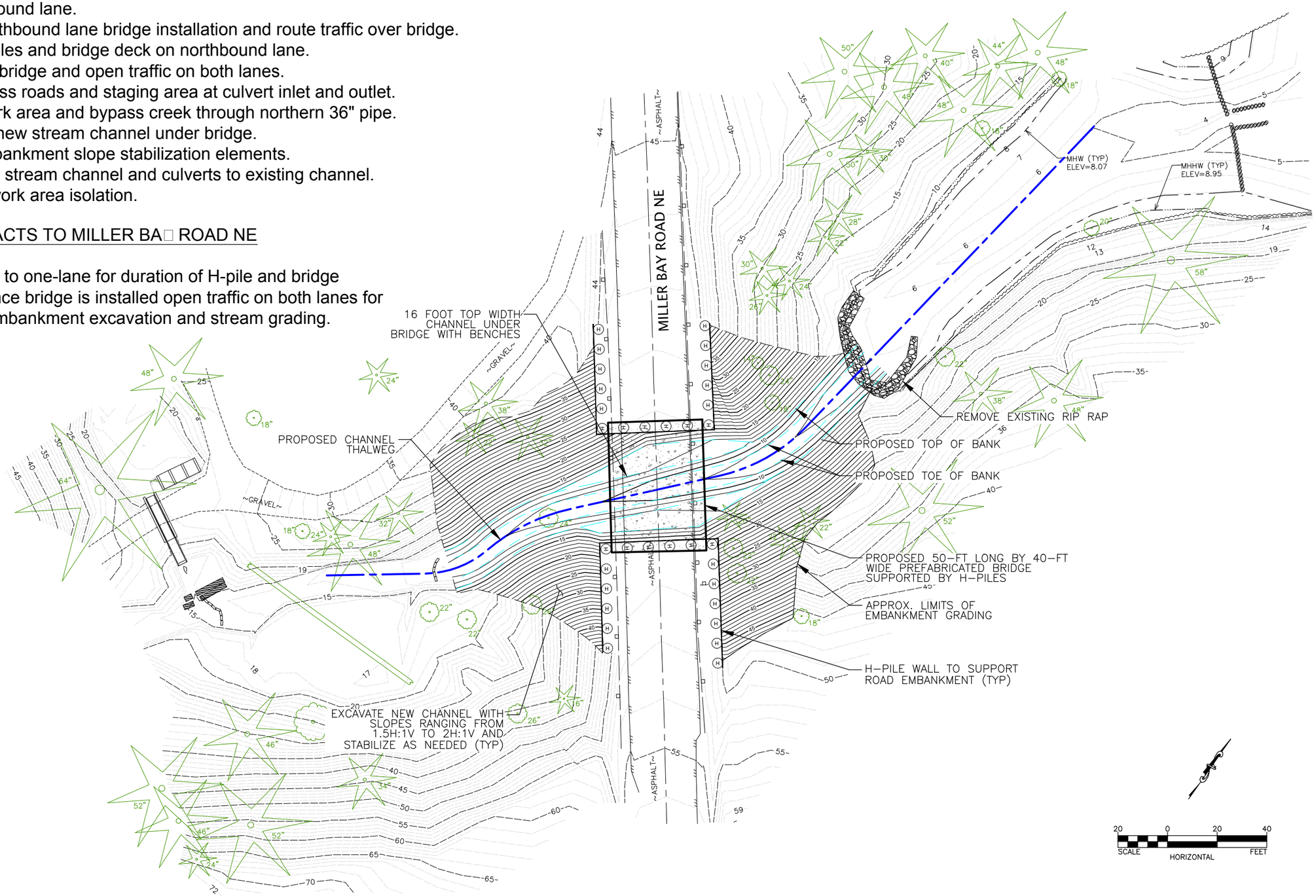
File Date: 02/07/17 5:48 PM - Jay Smith
File: T:\Geosyn\ALL PROJECTS\Cowling Creek MSFEG\Autocad\Design\Alternative\ALT1\Alt Tab 2 ALTERNATIVE 1 - PROFILE AND SECTION

T□PICAL CONSTRUCTION SEQUENCE

1. Close northbound lane of Miller Bay Road NE and install H-piles and bridge deck on southbound lane.
2. Finish southbound lane bridge installation and route traffic over bridge.
3. Install H-piles and bridge deck on northbound lane.
4. Complete bridge and open traffic on both lanes.
5. Build access roads and staging area at culvert inlet and outlet.
6. Isolate work area and bypass creek through northern 36" pipe.
7. Excavate new stream channel under bridge.
8. Install embankment slope stabilization elements.
9. Tie-in new stream channel and culverts to existing channel.
10. Remove work area isolation.

TRAFFIC IMPACTS TO MILLER BA□ ROAD NE

Restrict access to one-lane for duration of H-pile and bridge installation. Once bridge is installed open traffic on both lanes for remainder of embankment excavation and stream grading.



DATE	RE ISIONS	NO.

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CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

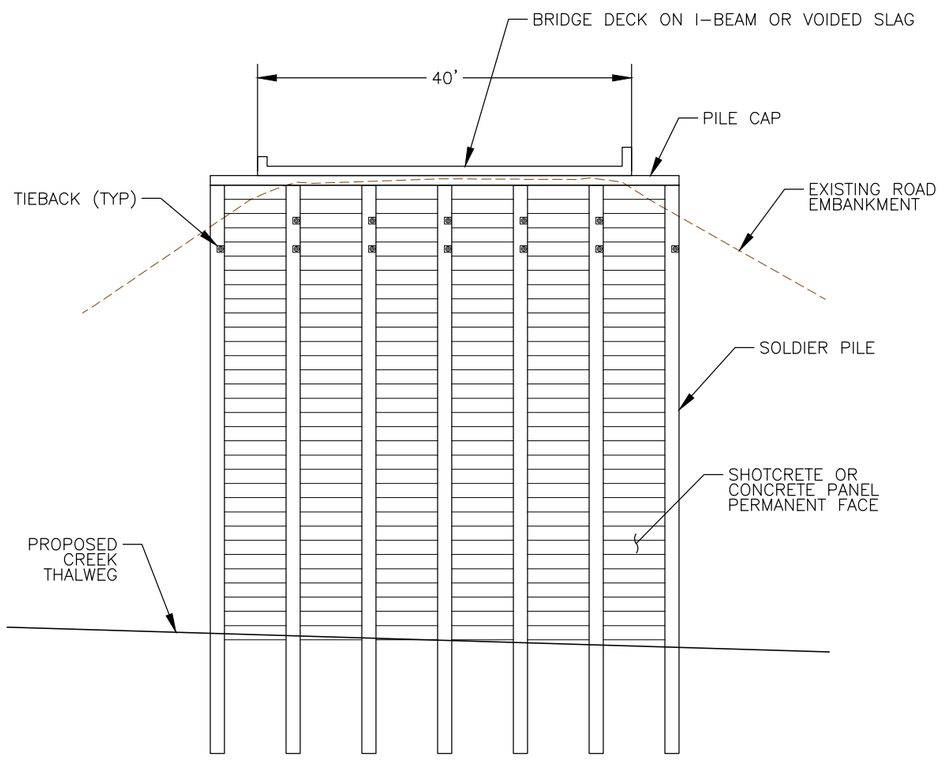
OPTION 2 - H-PILE BRIDGE

COWLING CREEK CULVERT REPLACEMENT

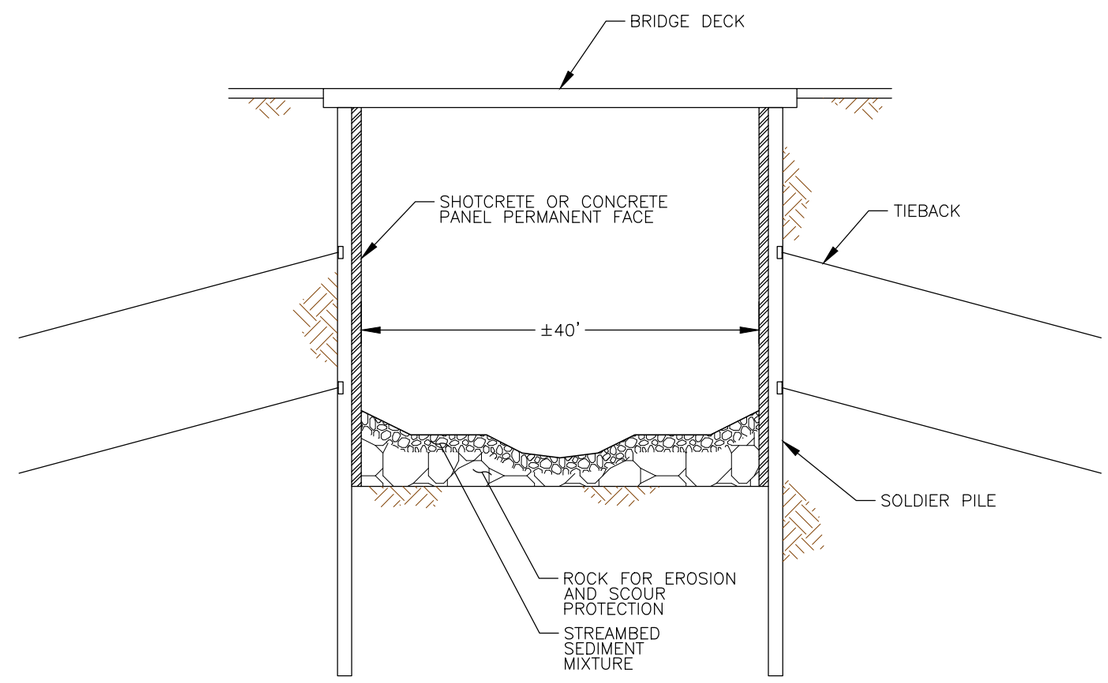
KITSAP COUNT□

DATE:	FEBRUAR□, 2017
DESIGNED B□:	SM, JS
DRAWN B□:	JS
CHECKED B□:	SM
CARDNO JOB NO.	E316301200
SHEET NO.	3

File Date: 4/20/2017 5:51 PM - Jay Smith
File: T:\Develop\ALL PROJECTS\Cowling Creek MSFEG\AutoCAD\Sheets\Design Alternatives\ALT3 H-PILE BRIDGE.dwg Tab: 3. ALTERNATIVE 3 - H-PILE BRIDGE



1 - **TYPICAL H-PILE SUPPORTED BRIDGE - ELEVATION VIEW**
NOT TO SCALE



2 - **TYPICAL H-PILE SUPPORTED BRIDGE - SECTION VIEW**
NOT TO SCALE

NO.	RE ISIONS	DATE

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MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

OPTION 2 - H-PILE BRIDGE SECTION

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

DATE:	FEBRUAR , 2017
DESIGNED B :	SM, JS
DRAWN B :	JS
CHECKED B :	SM
CARDNO JOB NO.	E316301200
SHEET NO.	4

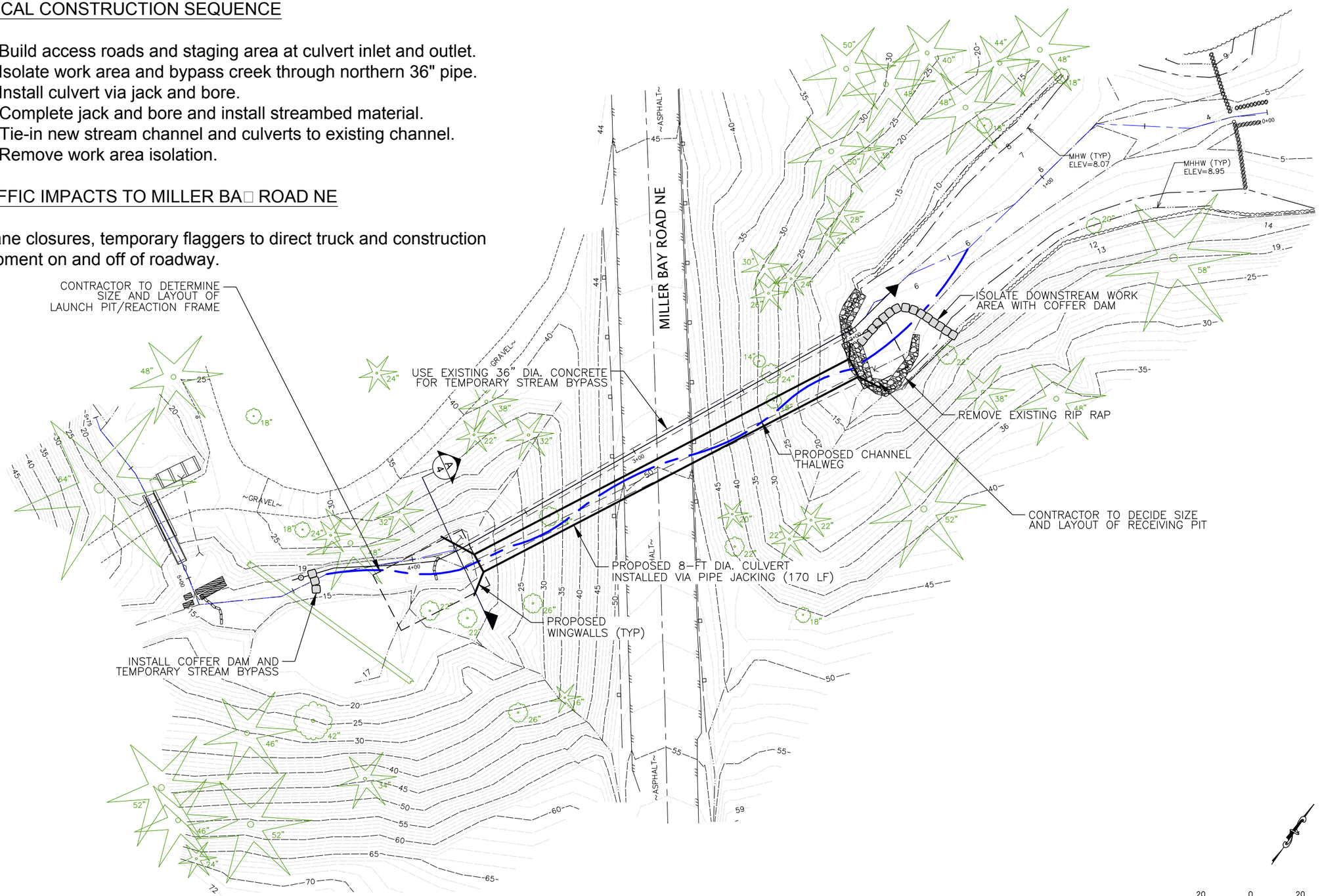
File Name: 4D4201F161FM - Jay-Smith
File: T:\Concept\ALL PROJECTS\Cowling Creek MBFEG\AutoCAD\Planet\Design Alternative\ALT3 HPILE BRIDGE.dwg Tab: 4_ALTERNATIVE 3 - H-PILE BRIDGE SECTION

T□PICAL CONSTRUCTION SEQUENCE

1. Build access roads and staging area at culvert inlet and outlet.
2. Isolate work area and bypass creek through northern 36" pipe.
3. Install culvert via jack and bore.
4. Complete jack and bore and install streambed material.
5. Tie-in new stream channel and culverts to existing channel.
6. Remove work area isolation.

TRAFFIC IMPACTS TO MILLER BA□ ROAD NE

No lane closures, temporary flaggers to direct truck and construction equipment on and off of roadway.



NO.	RE ISIONS	DATE

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MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

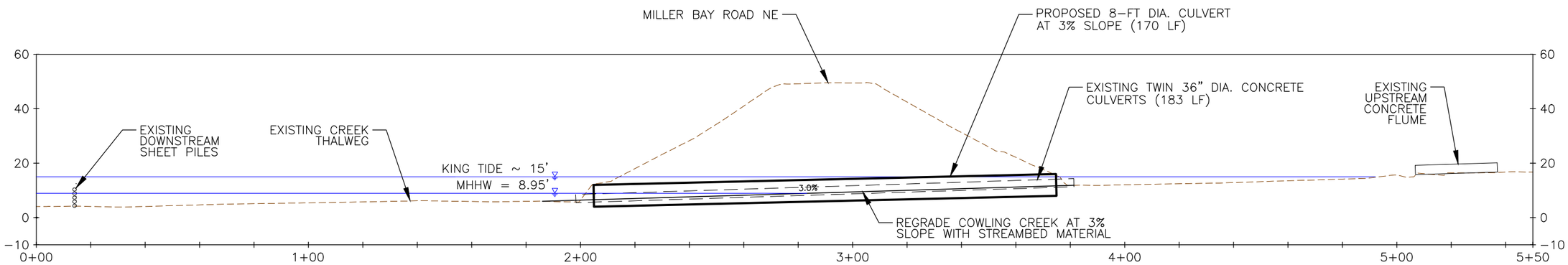
OPTION 3 - 8-FT CUL□ERT

COWLING CREEK CULVERT REPLACEMENT

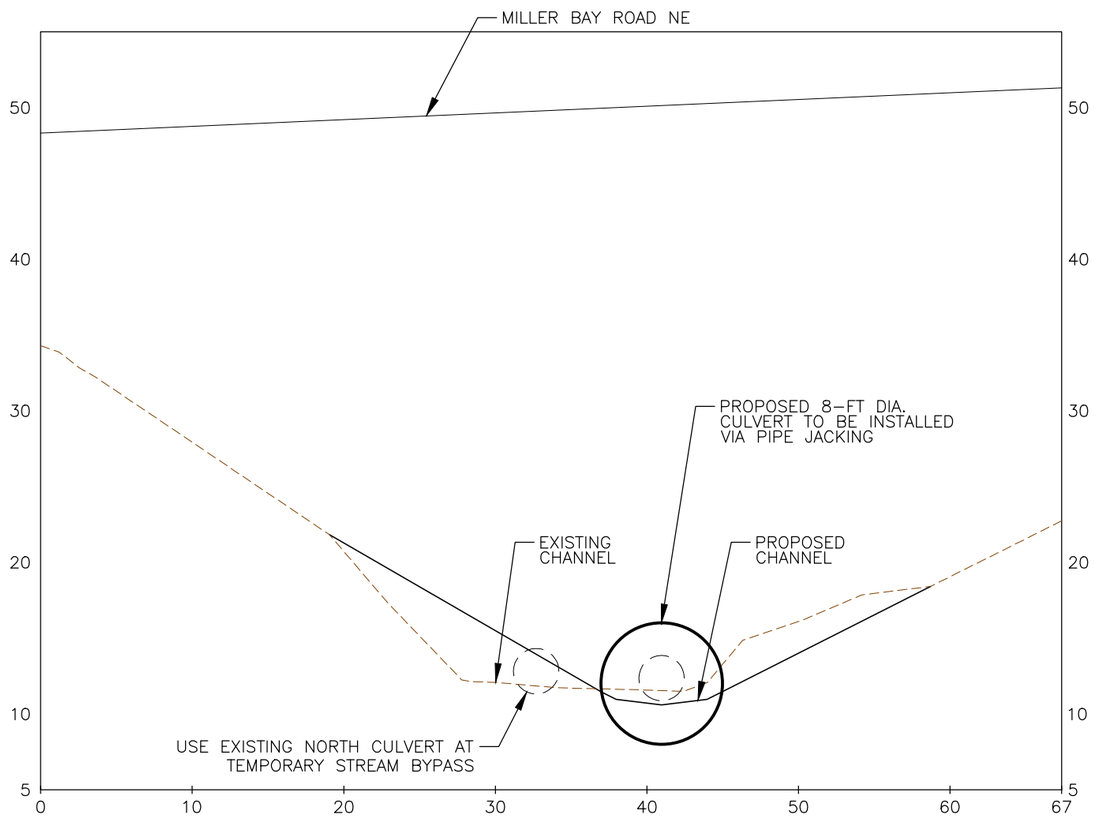
KITSAP COUNT□

DATE:	FEBRUAR□, 2017
DESIGNED B□:	SM, JS
DRAWN B□:	JS
CHECKED B□:	SM
CARDNO JOB NO.	E316301200
SHEET NO.	5

File Date: 4/20/2017 5:51 PM , Jon Smith
File: T:\Geospatial\ALL PROJECTS\Cowling Creek\MSFEO\AutoCAD\Sheets\Design Alternative\ALT3 8FT PIPE JACK.dwg Tab: 3 ALTERNATIVE 2 - 8-FT CULVERT



1 / - **COWLING CREEK PROFILE**
SCALE: AS SHOWN



A / 3 **CULVERT INLET (LOOKING DOWNSTREAM)**
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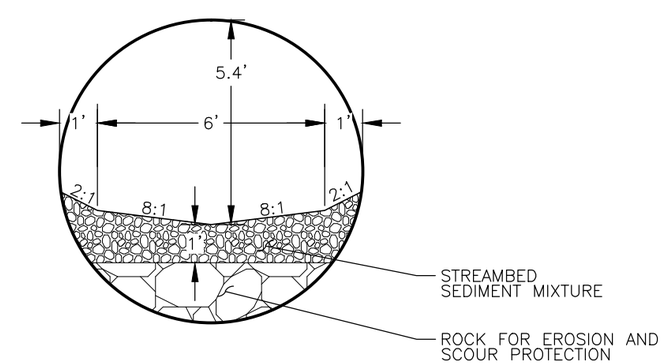

2 / - **TYPICAL CULVERT CHANNEL SECTION**
SCALE: AS SHOWN

SCALE 20 0 20 40 FEET HORIZONTAL
SCALE 20 0 20 40 FEET VERTICAL
SCALE 5 0 5 10 FEET HORIZONTAL
SCALE 5 0 5 10 FEET VERTICAL
SCALE 2 0 2 4 FEET HORIZONTAL
SCALE 2 0 2 4 FEET VERTICAL

NO.	RE ISIONS	DATE

SEAL

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801 SECOND A E, SUITE 700
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(206) 269-0104

MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

OPTION 3 - PROFILE AND SECTION

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

DATE:	FEBRUAR , 2017
DESIGNED B :	SM, JS
DRAWN B :	JS
CHECKED B :	SM
CARDNO JOB NO.	E316301200
SHEET NO.	6

File Date: 02/07/17 5:17 PM - Jay Smith
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Cowling Creek Culverts
Replacement at Miller Bay Road NE

APPENDIX

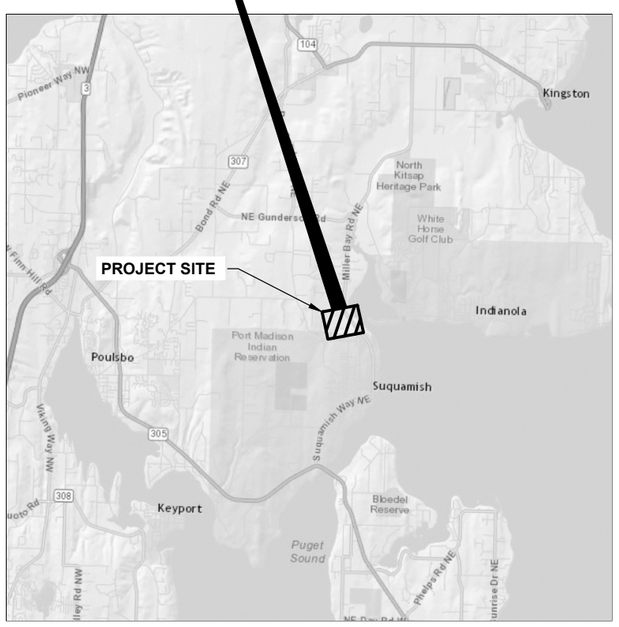
C

15% CONCEPTUAL DESIGN DRAWINGS

MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

COWLING CREEK CUL□ERT REPLACEMENT

KITSAP COUNT□, WASHINGTON



LOCATION MAP

N.T.S.

VICINITY MAP

N.T.S.

Sheet List Table	
Sheet Number	Sheet Title
1	COVER
2	EXISTING CONDITIONS
3	SITE PLAN
4	DETAILS

NO.	RE ISIONS	DATE

SEAL

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MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

CO□ER

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT□

DATE:	AUGUST, 2017
DESIGNED B□:	SM, JS
DRAWN B□:	JS
CHECKED B□:	SM
CARDNO JOB NO.	E316301200
SHEET NO.	1

File Date: 8/18/2017 3:01 PM - Jay Smith
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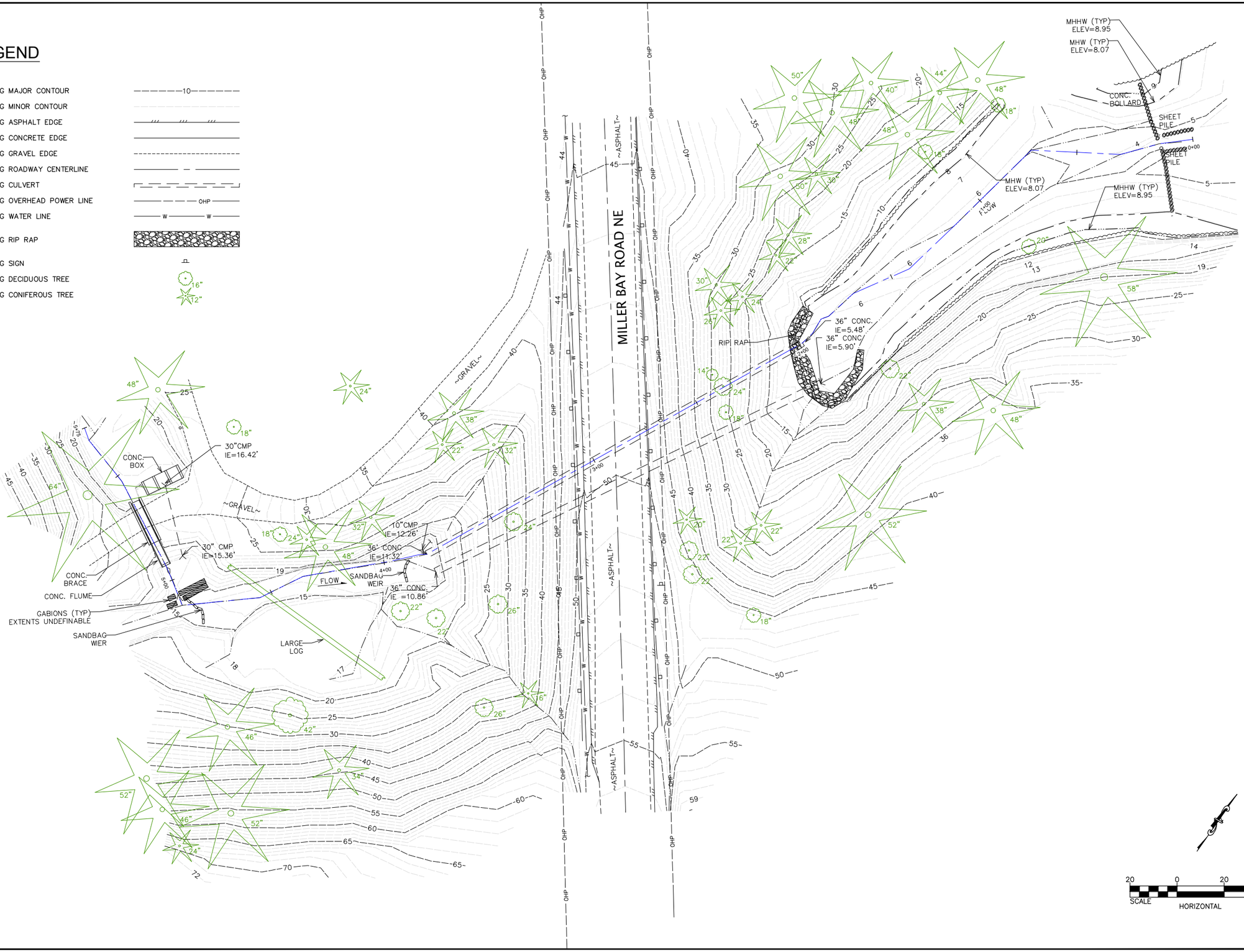
LEGEND

- EXISTING MAJOR CONTOUR
- EXISTING MINOR CONTOUR
- EXISTING ASPHALT EDGE
- EXISTING CONCRETE EDGE
- EXISTING GRAVEL EDGE
- EXISTING ROADWAY CENTERLINE
- EXISTING CULVERT
- EXISTING OVERHEAD POWER LINE
- EXISTING WATER LINE
- EXISTING RIP RAP
- EXISTING SIGN
- EXISTING DECIDUOUS TREE
- EXISTING CONIFEROUS TREE



SCALE 20 0 20 40 HORIZONTAL FEET

NO.	RE ISIONS	DATE

SEAL

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SEATTLE, WA 98104
(206) 269-0104

MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

E ISTING CONDITIONS

COWLING CREEK CULVERT REPLACEMENT

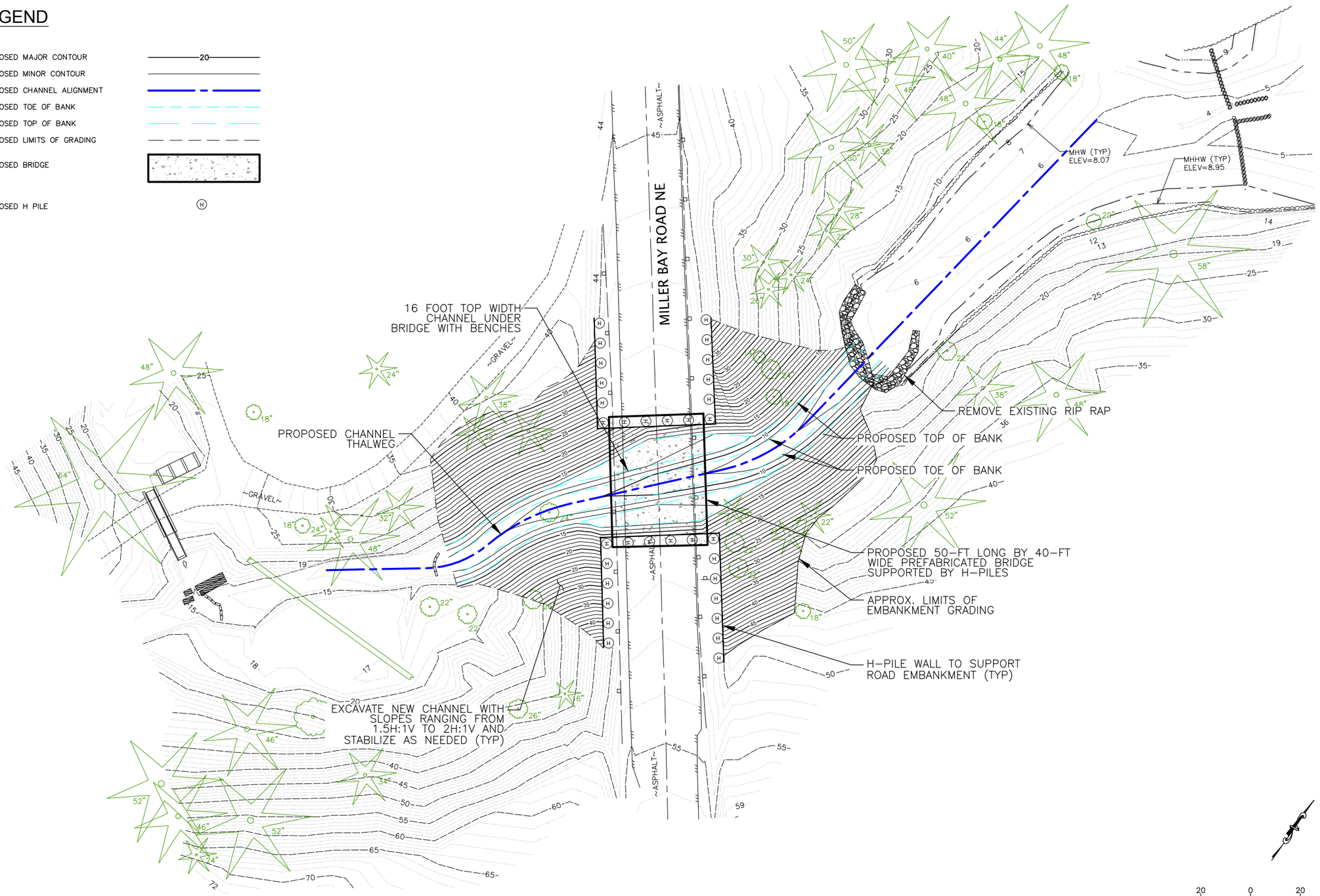
KITSAP COUNT

DATE:	AUGUST, 2017
DESIGNED B :	SM, JS
DRAWN B :	JS
CHECKED B :	SM
CARDNO JOB NO.	E316301200
SHEET NO.	2

Plot Date: 8/8/2017 3:18 PM , Jos Smith
File: T:\Geospg\ALL PROJECTS\Cowling Creek MERE\AutoCAD\Sheet 1\BUEXISTING CONDITIONS.dwg Tab: 2 EXISTING CONDITIONS

LEGEND

- PROPOSED MAJOR CONTOUR (20)
- PROPOSED MINOR CONTOUR
- PROPOSED CHANNEL ALIGNMENT
- PROPOSED TOE OF BANK
- PROPOSED TOP OF BANK
- PROPOSED LIMITS OF GRADING
- PROPOSED BRIDGE
- PROPOSED H PILE (H)



20 0 20 40
SCALE HORIZONTAL FEET

NO.	RE ISIONS	DATE

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MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

SITE PLAN

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

DATE:	AUGUST, 2017
DESIGNED B:	SM, JS
DRAWN B:	JS
CHECKED B:	SM
CARDNO JOB NO.	E316301200
SHEET NO.	3

Plot Date: 8/8/2017 3:36 PM Jim Smith
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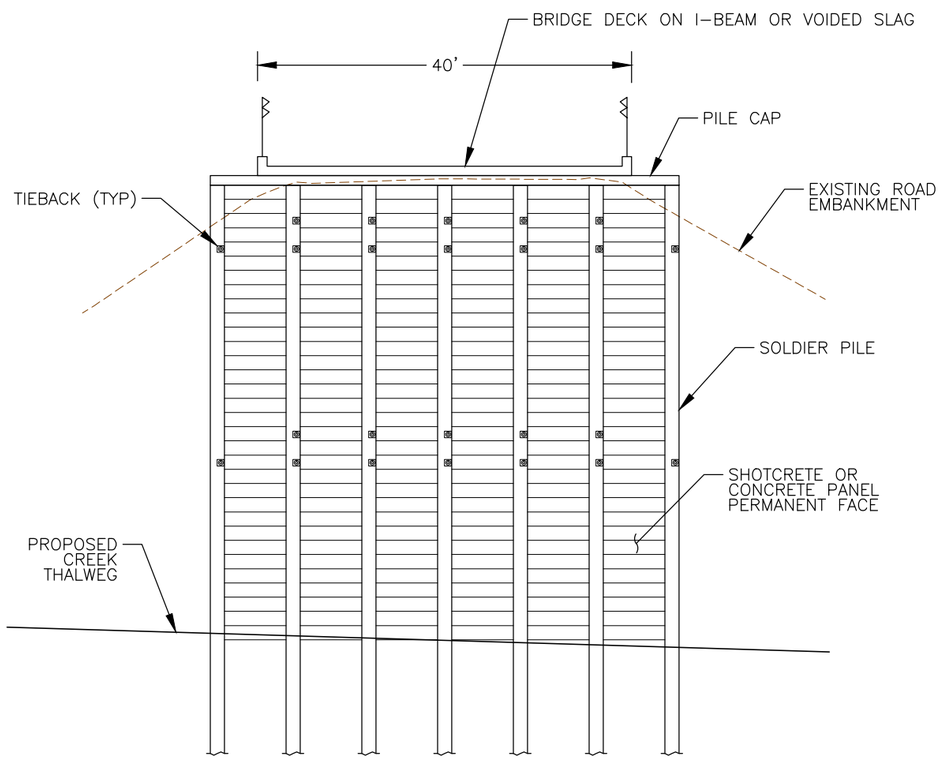
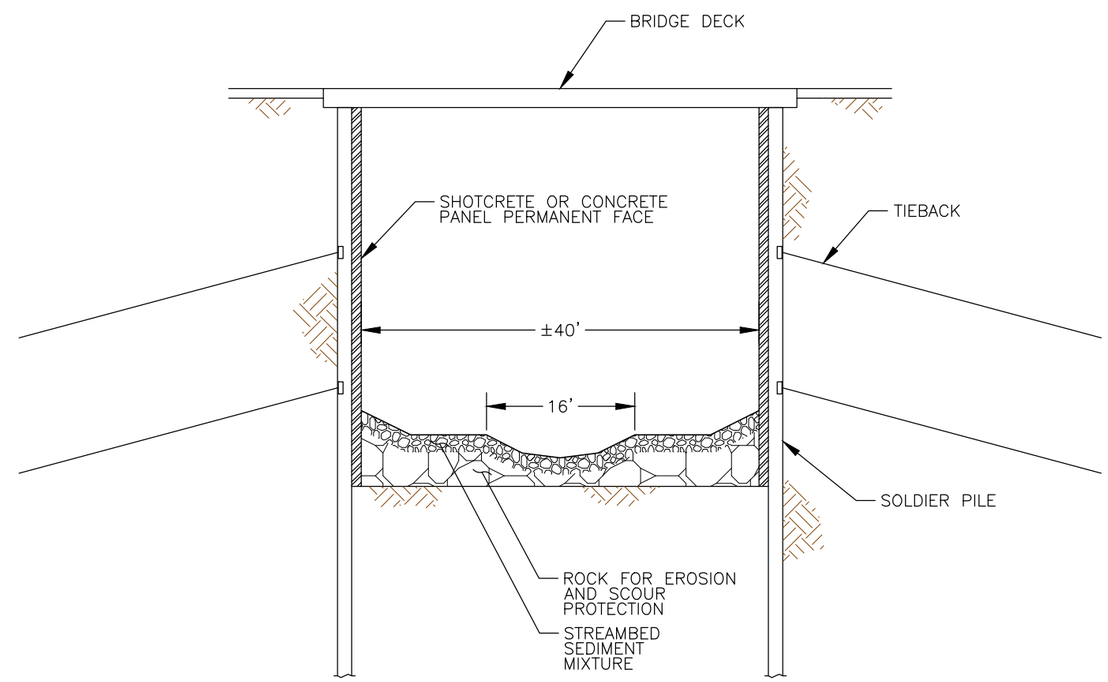


1 - **TYPICAL H-PILE SUPPORTED BRIDGE - ELEVATION VIEW**
NOT TO SCALE



2 - **TYPICAL H-PILE SUPPORTED BRIDGE - SECTION VIEW**
NOT TO SCALE

NO.	RE ISIONS	DATE

SEAL

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SEATTLE, WA 98104
(206) 269-0104

MID PUGET SOUND FISHERIES ENHANCEMENT GROUP

CONCEPTUAL DESIGN NOT FOR CONSTRUCTION

DETAILS

COWLING CREEK CULVERT REPLACEMENT

KITSAP COUNT

DATE:	AUGUST, 2017
DESIGNED B:	SM, JS
DRAWN B:	JS
CHECKED B:	SM
CARDNO JOB NO.	E316301200
SHEET NO.	4

Plot Date: 8/9/2017 1:38 PM Jon Smith
File T:\Sources\ALL PROJECTS\Cowling Creek MBFEG\AutoCAD\Sheets\1\SITE PLAN Reg Tab -4 DETAILS

Cowling Creek Culverts
Replacement at Miller Bay Road NE

APPENDIX D

15% CONCEPTUAL CONSTRUCTION COST ESTIMATE

Cowling Creek Culvert Replacement - 50-foot-long Bridge

Estimate of Construction Cost

Mid Puget Sound Fisheries Enhancement Group

10/13/2017 - Conceptual Level Estimate

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL
PREPARATION				
Mobilization - includes H-Pile Equipment	LS	1	$ 200,000	$ 200,000
Overhead Power Re-routing/Coordination	LS	1	$ 50,000	$ 50,000
CHANNEL GRADING				
Embankment Excavation and Haul	CY	6800	$ 40	$ 272,000
Soil Nail Stabilization	SY	950	$ 100	$ 95,000
BRIDGE				
Pre-cast 50' long by 40' wide deck	LS	1	$ 600,000	$ 600,000
Permanent Pile Wall and Tie Backs, incl. H-Piles	SF	7000	$ 175	$ 1,225,000
OTHER ITEMS				
Misc Construction Items (Traffic Control, Guard Rails, etc.)	LS	1	$ 250,000	$ 250,000
Contingency		30%		$ 800,000
CONSTRUCTION SUB- TOTAL				**$ 3,492,000**