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TECHNICAL MEMORANDUM

Date: May 13, 2026

Project: Lewis County Senn MP 0.87 Scour Mitigation
To: Doug Sarkkinen, PE, Project Manager | Otak

From: Isaac Fournier, PE | Osborn Consulting
Henry Moen, EIT | Osborn Consulting

cc:
Subject: Senn Hydraulic and Hydrology Report

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INTRODUCTION

Lewis County (County) is planning scour mitigation at Senn Road bridge (MP 0.87), to bring the structure's current National Bridge Inspection (NBI) Code 113 Rating 3 – Scour Critical, up to 7 – Design Countermeasures Installed. Osborn Consulting, Inc. (Osborn) performed hydrologic, hydraulic, and scour analyses of the existing conditions, and developed two concept level countermeasure designs for the Senn Road bridge crossing. Project location and vicinity are shown below in Figure 1.

Figure 1: Project Vicinity Map (Lewis County GIS Web Map)

SECTION 1 HYDROLOGIC ANALYSIS

A hydrologic analysis of Lucas Creek at the Senn Road crossing was conducted to inform hydraulic modeling efforts. Three methods were used to generate and analyze hydrologic conditions at this site: a flood frequency analysis performed by Northwest Hydraulic Consultants (NHC) as part of a previous project, a United States Geological Survey (USGS) Flood Q regression analysis and a Flood Q ratio analysis were both performed by Osborn as part of the project.

The flood frequency analysis used a nearby gage on the North Fork Newaukum River [USGS Gage #12024400 (NF Newaukum River Abv Bear Creek NR Forest, WA)] as a basis to calculate the statistical properties of the gaged river hydrology and then apply that data to the Lucas Creek basin using the drainage area ratio method. This resulted in a 500-year flow event of 4,579 cfs and a 2-year flow event of 828 cfs.

The USGS Flood Q regression analysis requires the basin drainage area, the annual precipitation and the region in Washington State as input parameters to estimate the hydrology. The drainage basin and annual precipitation were determined using the USGS Streamstats application (USGS 2019). The report generated by StreamStats, including the regression equation flows, is provided in Appendix A. Lucas Creek is in Regression Region 4 Washington and has a drainage area of 13.1 square miles and an annual precipitation of 56.5 inches. Based on these parameters, the 500-year flow event was estimated to be 2,240 cfs and the 2-year flow event as 414 cfs. A printout of the Flood Q regression and ratio tool is shown in Appendix B.

The USGS Flood Q ratio tool uses the same data as the Flood Q regression tool but compares the basin to a selected gage on a nearby river. Gage #12024400 was selected as it is the closest and most similar in size to the Lucas Creek Basin. The Flood Q ratio tool analysis resulted in a 500-year flow event of 2,000 cfs and a 2-year flow event of 374 cfs. A printout of the Flood Q regression and ratio tool is shown in Appendix B.

From these analysis methods the Flood Q regression hydrology was selected for the hydraulic model. The Flood Q ratio tool requires that the basin area being analyzed is within 50% of the size of the gaged basin for the weighted hydrology to be valid. The gage station basin area is 29.7 square miles, and the Lucas Creek basin area is only 45% of that area. The results of the flood frequency analysis were not considered as hydraulic modeling indicated the 2-year flow exceeded the field observed bankfull width by approximately 10 feet, suggesting the flows are overly conservative. The estimated flood discharges for Lucas Creek at the Senn Road crossing that were used for subsequent hydraulic modeling are summarized below in Table 1.Hydraulic modeling results indicated the 2-year flow included in Table 1 generally aligned with the field observed bankfull width and is considered to represent site conditions.

Table 1. Peak Flows for Lucas Creek at the Senn Road Bridge Crossing

Gage #, Analysis Tool	Flow Frequency Return Period					
	2-Year (cfs)	10-Year (cfs)	25-Year (cfs)	50-Year (cfs)	100-Year (cfs)	500-Year (cfs)
#12024400 Flood Q Regression Tool	414	897	1,180	1,410	1,650	2,240

Notes:
cfs = cubic feet per second

SECTION 2 HYDRAULIC ANALYSIS

Hydraulic analysis of the Senn Road bridge crossing was performed using the United States Bureau of Reclamation's SRH-2D Version 3.3.1 computer program, a two-dimensional hydraulic and sediment transport numerical model (USBR, 2020). Pre- and post- processing for this model was included using the Surface-water Modeling System (SMS) version 13.4.5 (Aquaveo, 2025). Results from the hydraulic modeling were used in coordination with the FHWA Hydraulic Toolbox software version 5.4, specifically the Bridge Scour Analysis Calculator (FHWA 2024). Scour calculations are discussed in Section 3.

The existing and proposed condition scenarios were analyzed using SRH-2D simulations for each flow frequency included in Table 1.

Model configuration including mesh qualities, roughness coefficients, and boundary conditions are discussed in the subsequent section.

2.1 Model Setup

Channel geometry data in the existing model was developed from topographic survey provided by Otak which extended 300 feet upstream and 300 feet downstream of the Senn Road bridge. Supplemental topographic information sourced from the Southwest Washington (SWWA) Foothills 2017 dataset on the Washington LiDAR Portal (USGS 2017) was merged with the survey. The existing conditions mesh extends from approximately 300 feet upstream to approximately 300 feet downstream of the crossing. The channel bed, banks, and thalweg were modeled using patch-type (quadrilateral) elements to capture the more singular-directional flow. Areas outside of the channel were modeled using paving-type (triangular) elements. The existing bridge abutments and piers were modeled as holes in the mesh. There are two groups of four cylindrical piers: one group on either side of the channel. The existing model mesh is shown in Figure 2.

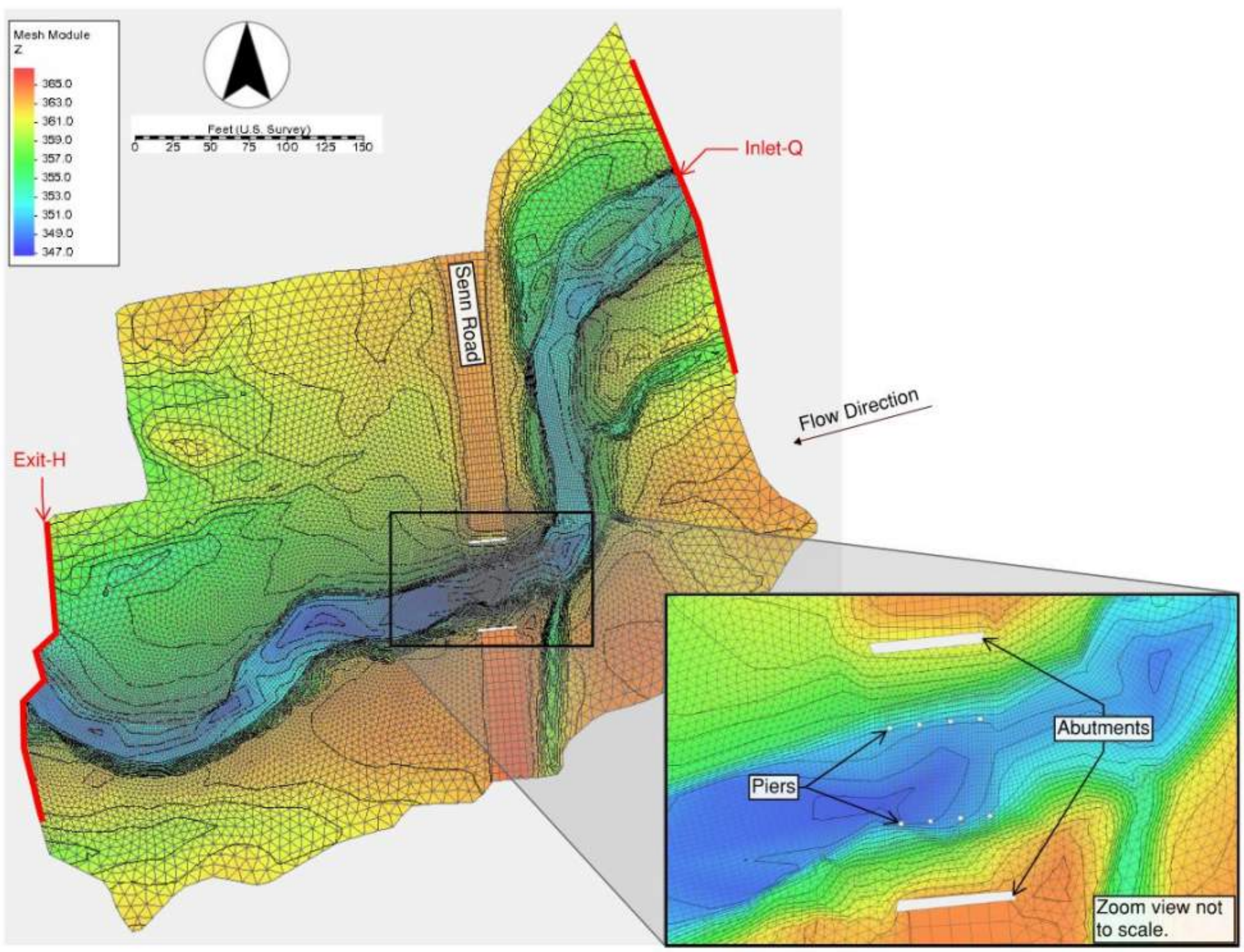

Figure 2: Existing conditions mesh and boundary conditions

In the proposed condition model, most properties of the existing condition model were kept the same as the existing condition, including mesh domain, element types, and existing abutments and piers represented as voids in the mesh. Proposed channel geometry was graded in Civil 3D and then exported as an XML to be used in SMS. Proposed grading extends from about 110 feet upstream to 75 feet downstream measured from the center of the bridge. Mesh areas affected by the proposed grading were adjusted so that patch-type elements were in line with the proposed channel. The proposed model mesh is shown in Figure 3.

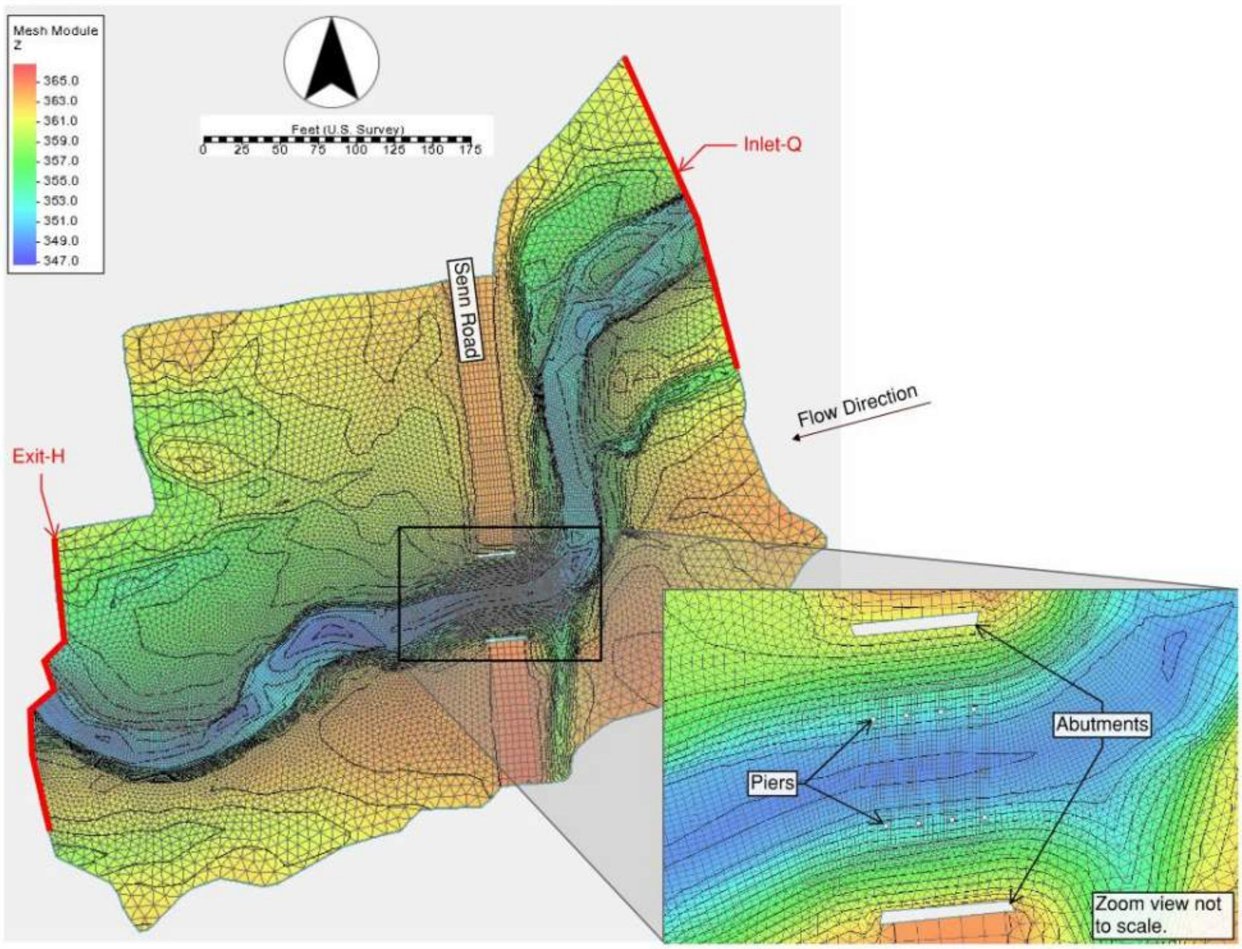

Figure 3 Proposed conditions mesh and boundary conditions

Boundary condition coverages were created for each simulation that was included in the model. Inflow boundary conditions used the Inlet-Q (subcritical flow) type with a constant discharge and conveyance distribution at the inlet. Discharge values varied by design flow event. The 2-year, 10-year, 25-year, 50-year, 100-year, and 500-year flow events were included in the model simulations (see Table 1 for flood discharge values). Outflow boundary conditions used the Exit-H (subcritical outflow) type with an exit water surface elevation that was generated from a normal depth rating curve. The rating curve was populated using the following inputs: Manning's n = 0.04, slope = 0.007 feet/foot, and flows ranging from 0 cfs to 2,240 cfs. Boundary arc locations are shown in Figure 2 and Figure 3.

Hydraulic roughness within each model simulation is defined by Manning's n values. Material coverages, which define the spatial distribution of roughness values, were delineated for the existing conditions model. The channel roughness coefficient was evaluated using the quasi-quantitative method (Arcement and Schneider 1989) in the Stream Channel Flow Resistance Coefficient Computation Tool (NSAEC 2018). Other material coverages including vegetated overbank, asphalt roadway, and agricultural land were assigned Manning's n roughness coefficients estimated from Chow's *Open-Channel Hydraulics* (Chow 1959). Manning's n values are listed in Table 2.

Table 2. Roughness Coefficients

Material	Manning's n
Existing Channel	0.04
Vegetated Overbank	0.08
Asphalt Roadway	0.02
Agricultural Land	0.045

2.2 Model Results

Hydraulic results for maximum channel depth, average channel velocity, and average channel shear stress were analyzed at representative cross section locations. Results were analyzed at seven cross section locations: three downstream (DS), one spanning the bridge, and three upstream (US). Cross section locations overlayed on the 100-year velocity dataset are shown below for existing and proposed conditions on Figure 4 and Figure 5, respectively. Main channel existing and proposed condition results are summarized in Table 3 and Table 4, respectively.

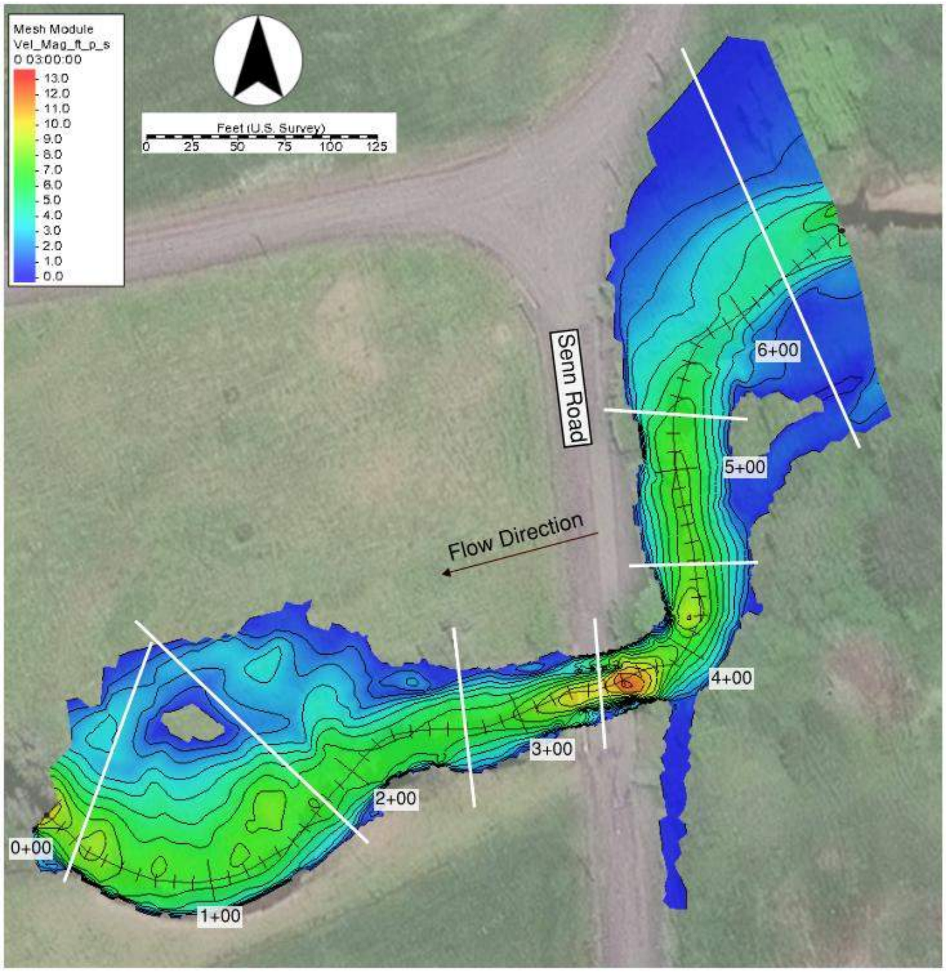

Figure 4: Existing conditions 100-year velocity dataset with locations of cross sections

Table 3. Existing Condition Main Channel Hydraulic Model Results

Hydraulic Parameter	Cross Section	2-Year	100-Year	500-Year
Max Depth (ft)	DS 0+23	3.7	7.3	7.4
	DS 1+78	3.5	6.7	7.2
	DS 2+70	5.5	8.9	9.6
	Structure 3+47	6.0	9.6	10.
	US 4+48	3.3	8.0	9.5
	US 5+31	4.3	8.9	10.4
	US 6+30	4.4	8.9	10.4
Average Velocity (ft/s)	DS 0+23	5.8	7.7	9.2
	DS 1+78	4.0	6.1	6.8
	DS 2+70	2.7	5.9	7.3
	Structure 3+47	3.3	7.0	8.6
	US 4+48	6.1	7.2	7.4
	US 5+31	4.4	5.9	5.6

Hydraulic Parameter	Cross Section	2-Year	100-Year	500-Year
	US 6+30	3.7	3.7	3.8
Average Shear (lb/SF)	DS 0+23	1.1	1.5	2.1
	DS 1+78	0.5	0.9	1.2
	DS 2+70	0.2	0.8	1.2
	Structure 3+47	0.3	1.2	1.8
	US 4+48	1.2	1.3	1.3
	US 5+31	0.6	0.9	0.7
	US 6+30	0.5	0.4	0.3

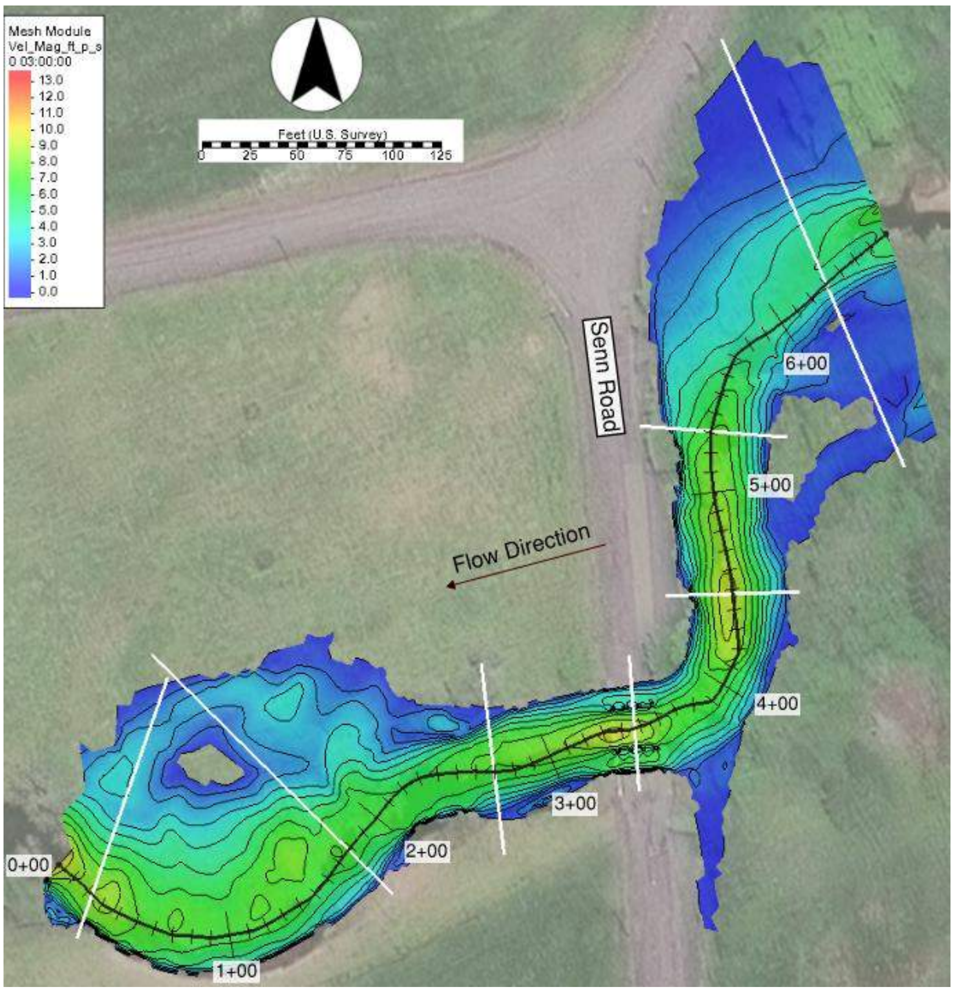

Figure 5 Proposed condition 100-year velocity dataset with locations of cross sections

Table 4 Proposed Condition Main Channel Hydraulic Model Results

Hydraulic Parameter	Cross Section	2-Year	100-Year	500-Year
Max Depth (ft)	DS 0+23	3.7	7.3	7.3

Hydraulic Parameter	Cross Section	2-Year	100-Year	500-Year
	DS 1+78	3.5	6.7	7.2
	DS 2+70	5.3	8.6	9.2
	Structure 3+47	5.8	9.3	10.1
	US 4+48	2.9	7.0	8.3
	US 5+31	4.2	8.4	9.8
	US 6+30	4.3	8.6	9.9
Average Velocity (ft/s)	DS 0+23	5.8	7.7	9.3
	DS 1+78	4.0	6.1	6.8
	DS 2+70	2.9	6.2	7.7
	Structure 3+47	3.5	6.9	8.2
	US 4+48	6.9	8.6	9.0
	US 5+31	4.5	6.5	6.7
	US 6+30	3.8	3.9	4.2
Average Shear (lb/SF)	DS 0+23	1.1	1.5	2.2
	DS 1+78	0.5	0.9	1.2
	DS 2+70	0.2	0.9	1.3
	Structure 3+47	0.4	1.3	1.8
	US 4+48	1.6	1.9	2.0
	US 5+31	0.7	1.1	1.1
	US 6+30	0.5	0.4	0.4

2.3 100-Year Water Surface Elevation Comparison

This project is within a FEMA Zone A Special Flood Hazard Area. Otak performed a no-rise analysis of the project reach (see Appendix C). Hydraulic results for average channel water surface elevation (WSE) were analyzed at representative cross section locations. Results show that there is not an increase in the 100-year water surface elevation anywhere within the modeled project reach. Results are provided in Table 5.

Table 5 1D No-Rise: 100-Year Average WSE Comparison

Cross Section STA	Existing WSE	Proposed WSE	WSE Difference[1]
US 4+98	360.1	359.8	-0.3
US 4+52	359.6	359.1	-0.5
US 3+89	359.2	358.3	-0.9
US 3+13	358.9	358.8	-0.1
DS 2+52	358.0	357.9	-0.1
DS 1+78	357.8	357.8	0.0
DS 1+27	357.7	357.7	0.0
DS 0+49	356.8	356.8	0.0

1) Difference calculated as Proposed WSE minus Existing WSE.

SECTION 3 SCOUR ANALYSIS

3.1 Scour Results

Hydraulic model results were exported from the bridge scour coverage in SMS to the Hydraulic Toolbox Bridge Scour Calculator (FHWA 2024). Placement of the bridge scour coverage arcs are shown in Figure 6. Contraction, pier, and abutment scour were evaluated for proposed conditions. All modeled flood events were evaluated to identify which flow produced the greatest amount of scour. Long-term degradation potential was evaluated separately.

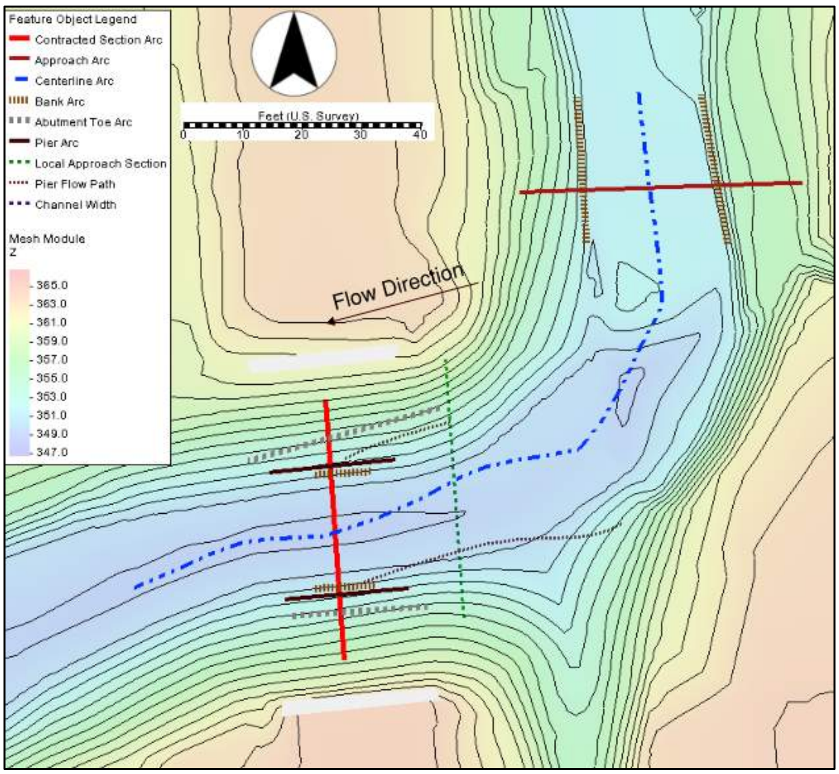

Figure 6: Bridge scour coverage arcs

Proposed conditions include the scour countermeasure material Rock for Erosion and Scour Protection Class B (Section 9-13.4(2)) (WSDOT 2025). Maximum contraction, pier, and abutment scour depths are expected to occur at the 500-year flow event, with values of 0.3 feet, 5.4, and 7.4 feet, respectively. All scour depths calculated in the Hydraulic Toolbox are summarized in Table 6.

Table 6. Hydraulic Toolbox Scour Summary

Flood Frequency	Scour Type				
	Contraction (feet)(1)	Pier #1 (feet)	Pier #2 (feet)	Abutment Existing Material(2) (feet)	Abutment Class B Rock(3) (feet)
500-Year	**0.0** (2)	**4.3**	**3.9**	**5.4**	0.0
100-Year	0.0	3.9	3.6	3.4	0.0
50-Year	0.0	3.4	3.4	2.4	0.0

Flood Frequency	Scour Type				
	Contraction (feet)(1)	Pier #1 (feet)	Pier #2 (feet)	Abutment Existing Material(2) (feet)	Abutment Class B Rock(3) (feet)
25-Year	0.0	3.2	3.2	1.4	0.0
10-Year	0.0	2.9	3.0	0.4	0.0
2-Year	0.0	2.0	2.5	0.0	0.0

(1) Clear-water scour conditions controlled for all flow events and was used as Total Scour.
(2) Abutment scour depths are the same for left and right abutment.
(3) Bold values indicate maximum scour depth for each type of scour.

Long-term degradation potential is considered low at the Senn Road crossing due to the relatively flat slope of the stream and large sediment supply. The channel gradient is generally consistent based on the LiDAR elevation data with an equilibrium slope of 0.4 percent measured over approximately one mile of the stream. Additionally, exposed hard till material was observed in the streambed downstream of the crossing which is relatively resistant to degradation. A cross section from the existing conditions mesh shows the thalweg approximately 25 feet below the bridge deck (see Figure 7). Cross section data shown in Figure 8 from the years 1990, 1997, 2019, and 2021 shows some channel adjustments of approximately 1.3 feet over the period of 30 years. This indicates that the channel has degraded over 30 years. With zero feet of contraction scour, 1.3 feet of long-term degradation was used as Total Scour. The scour countermeasures were placed 1.7 feet deeper, at a depth of 3 feet, to accommodate the installation of streambed material. If this trend were to continue, long-term degradation is not anticipated to negatively impact the structure over the remaining life of the bridge. The existing exposed hardpan material is anticipated to further reduce the impacts of long-term degradation at the Senn Road crossing.

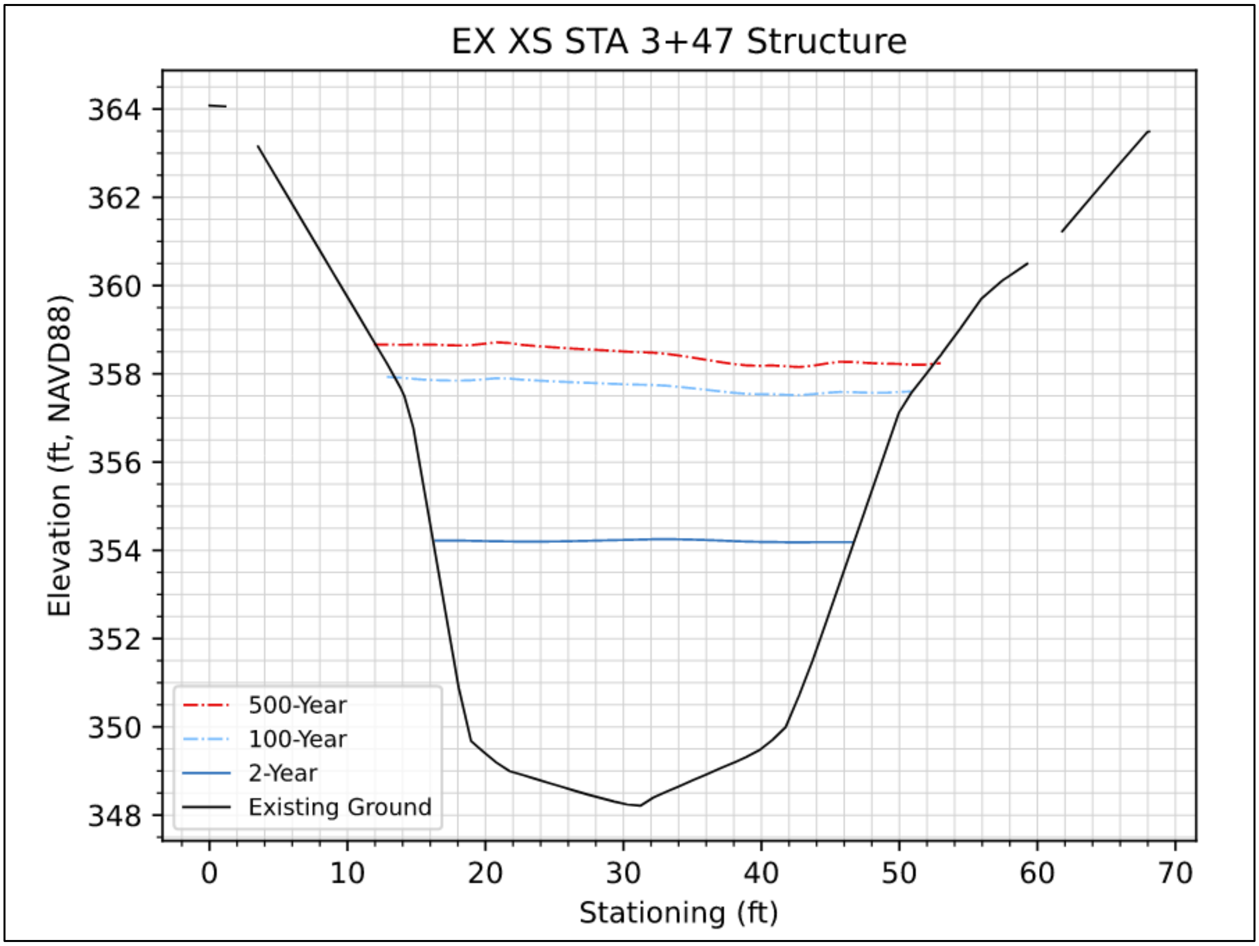

Figure 7: Existing (EX) cross section (XS) spanning the existing bridge with 2-year, 100-year, and 500-year water surface elevations

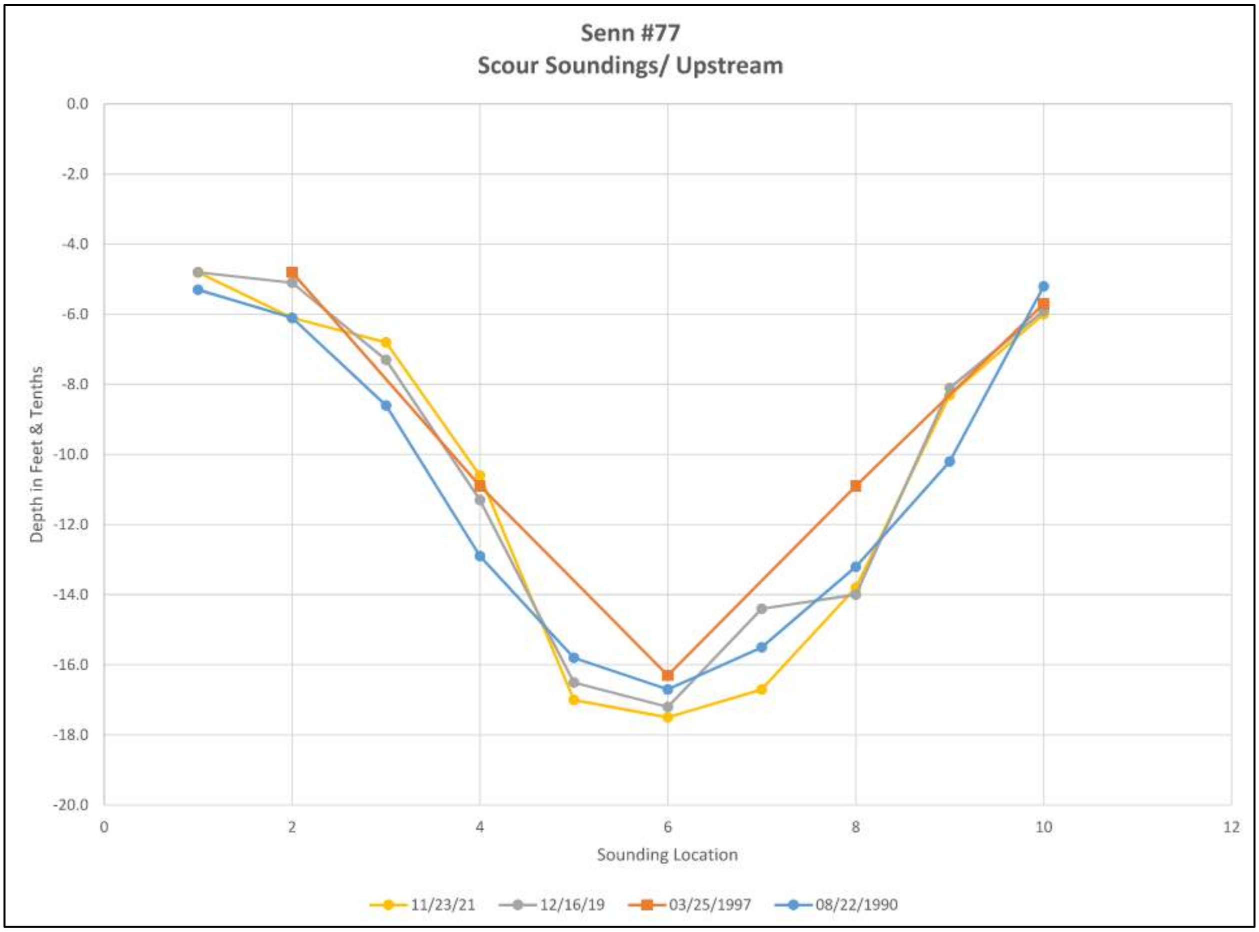

Figure 8: Historical Cross Section Data

3.2 Scour Countermeasure Design

The proposed scour countermeasures were designed using the hydraulic and scour results described in Section 2 and Section 3. The top of the proposed countermeasure is located an additional 1.7 feet beneath the depth of the sum of contraction scour and long-term degradation, at a depth of 3 feet. The depth between the top of the proposed scour countermeasures and finished grade was increased from the total scour depth of 1.7 feet to 3 feet to allow for installation of streambed material. Proposed designs are detailed in this section and plans are included in Appendix E.

3.2.1 Material Sizing

The scour countermeasure rock class was calculated using the Federal Highway Administration's (FHWA) HEC-23: Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Volume 2 (HEC 23 V2), Equation 4.1 Design Guidelines for Revetment Riprap (FHWA 2009). The inputs and results of this calculation are provided in Appendix F. A conservative factor of safety of 1.4 was used in the calculations compared to the recommended safety factor of 1.1 from HEC 23. Revetment scour equations initially indicated Rock for Erosion and Scour Protection Class A would be appropriate. The factor of safety was increased to require Rock for Erosion and Scour Protection Class B, consistent with the pier scour equation. Based on these calculations the D_{30} and D_{50} of the proposed rock material must be greater than 1.5 and 1.8 feet, respectively, and therefore Washington

State Department of Transportation (WSDOT) Standard Specifications 9-13.4(2) Rock for Erosion and Scour Protection Class B is proposed (WSDOT 2025).

Pier scour was evaluated separately using Equations 11.1 and 11.3 from HEC 23 V2, and the results are presented in Appendix G. The results indicate Rock for Erosion and Scour Protection Class B is appropriate for pier protection.

3.2.2 Layout

Layout of the revetment and aprons was performed using the FHWA Hydraulic Considerations of Abutments on Deep Foundations and Bridge Embankment Protection Tech Brief (TechBrief) (FHWA 2023) as well as recommendations from HEC 23 V2 Design Guidelines 4 and 11 (FHWA 2009). Figure 9, Figure 10, Figure 11, and Figure 12 illustrate the suggested countermeasure extents from the two guidelines. Proposed plan and section views are shown in Appendix E.

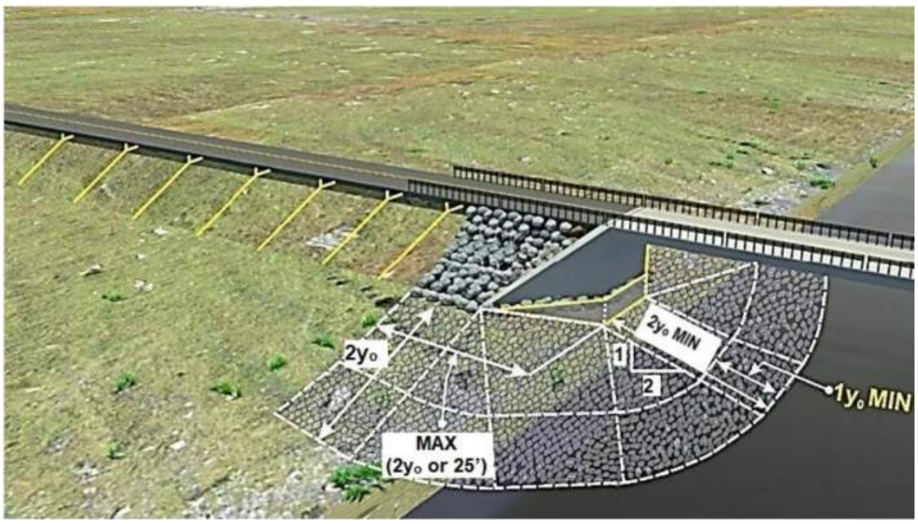

Figure 9: Countermeasure extents (y_o = main channel flow depth through the bridge) (TechBrief)

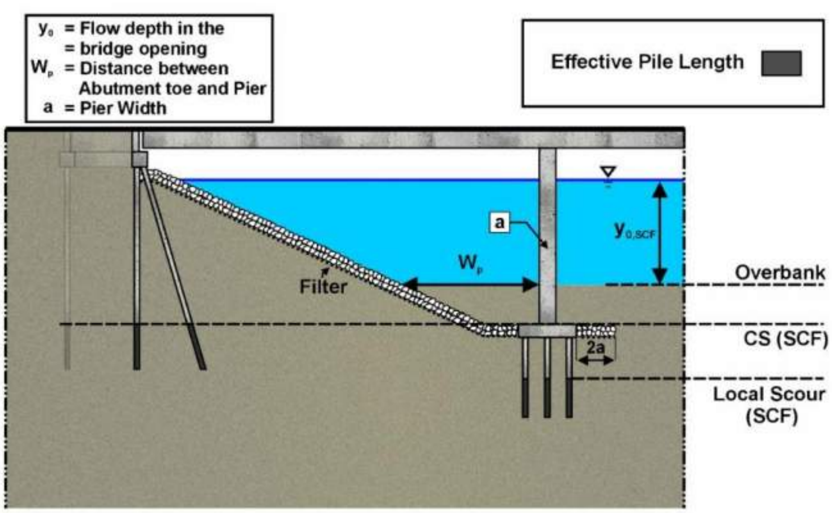

Figure 10: Countermeasure for scour condition B and a closely located pier (TechBrief)

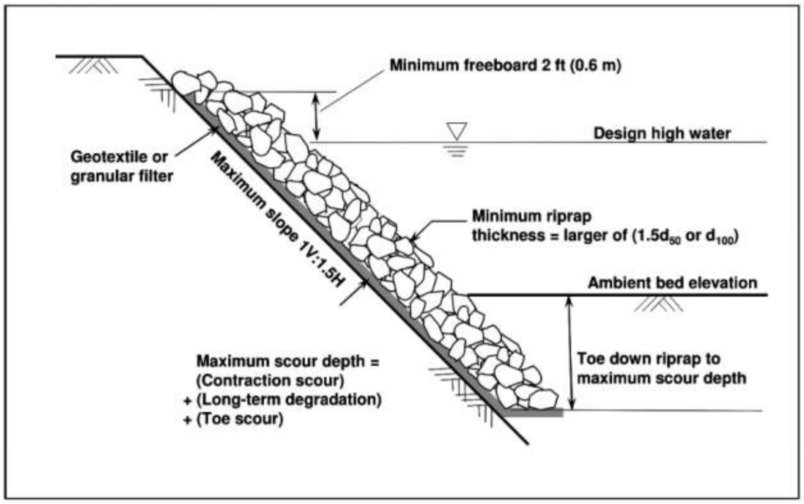

Figure 11: Riprap revetment with buried toe (HEC 23 V2 Design Guideline 4)

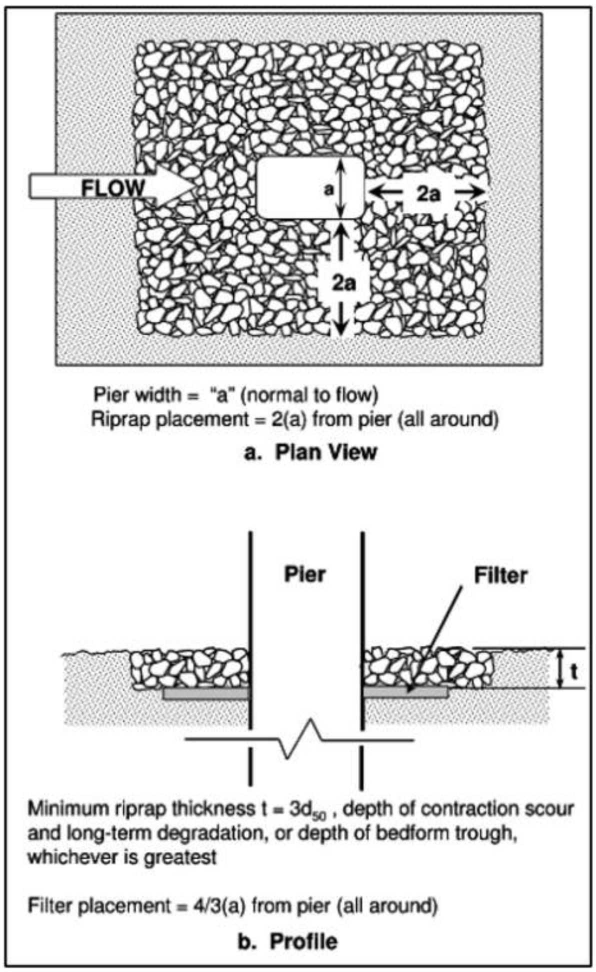

Figure 12: Riprap Pier Protection (HEC 23 V2 Design Guideline 11)

The proposed design incorporates elements from several of the recommended scour countermeasure treatments shown above, while accommodating project requirements, site constraints, constructability, and cost considerations. The proposed scour countermeasure cross section is shown in Figure 13. The proposed design plans are included in Appendix E.

1. 1.5H:1V slopes were used for the revetment design, as the proposed rock for erosion and scour protection material can sustain that slope.

2. The top of the countermeasure provides 2 feet of freeboard over the 500-year water surface elevation.

3. The proposed revetment and apron do not extend out from the abutment parallel to the roadway at the countermeasure design depth. The cross sections shown in Figure 6 indicate that over the last 30 years there has not been channel migration at the crossing. Furthermore, review of Google Earth Aerial Imagery indicates there has been limited to no change in channel alignment since 1990 within 500 feet of the bridge structure. Lateral migration of streams generally occurs at bends and progresses from upstream to downstream. The scour countermeasure and bridge structure will prevent lateral migration downstream of the structure.

   Upstream of the structure, the banks are vegetated and appear stable. The nearest river bend that could progress towards the roadway embankment on the right side of the channel is approximately 280 feet upstream which is sufficiently far upstream to ensure the structure should not be impacted within its design life. Large woody material was installed along the bank toe on the right bank upstream of the bridge to stabilize the bank. The stream makes a sharp right turn approximately 50 feet upstream of the bridge inlet where the stream flows along the steep left bank that appears to be hardpan material. The hardpan material is resistant to erosion and likely limits lateral migration.

4. The toe of the revetment and the apron was extended further below the channel grade than required to provide for placement of streambed material above the countermeasure. Placing streambed material above the countermeasure is beneficial from a habitat and project permitting perspective and will not negatively impact the effectiveness of the countermeasure.

5. Design Guideline 11 recommends that rock pier protection be installed at the surface to facilitate inspection. In this case since the piers are located near the abutments, the proposed pier protection would be installed as shown in the TechBrief.

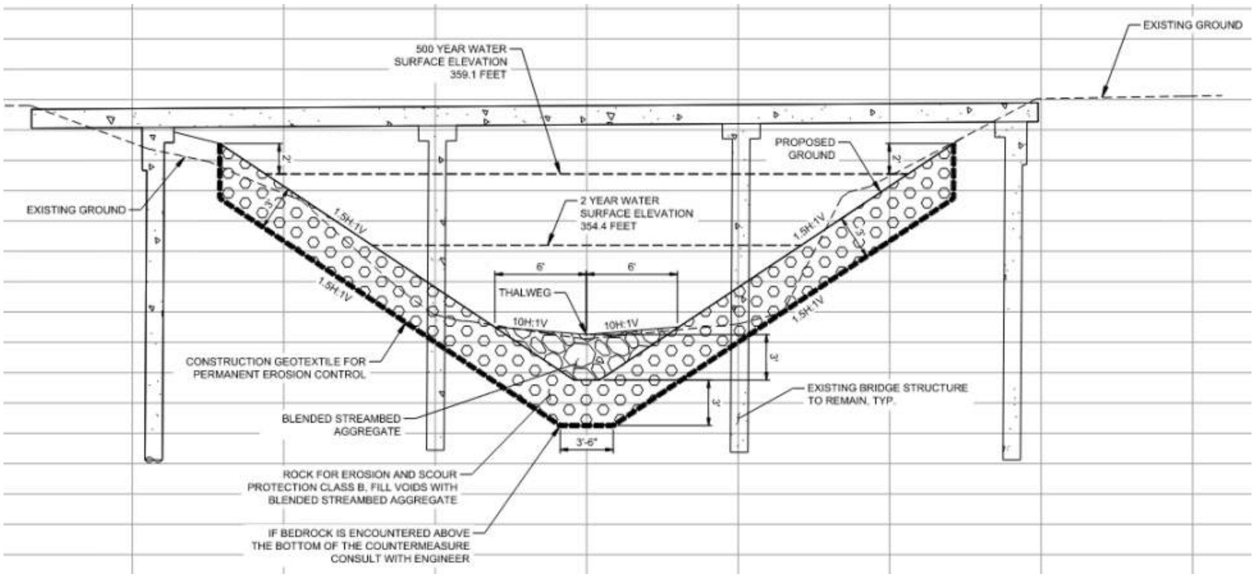

Figure 13: Proposed Scour Countermeasure Section View

SECTION 4 STREAM BYPASS SIZING

The conceptual stream diversion plan in Appendix E includes a force main pump and stream bypass pipe. Average maximum daily flows during the date range from July 15th to September 31st were used to anticipate expected flow during construction. The average flow is approximately 14 cfs (6,283 gpm). Flows during construction were based on the average flows in early October over the span of 17 years as found in the Lucas Creek Bank Protection Design at Senn Road report developed by NHC in 2016 as part of a bank stabilization project upstream of the Senn Road crossing. These flows represent a conservative estimate of the site conditions during the in-stream work window for construction. It is anticipated that flows will be bypassed using a combination of pumps to convey up to the design flow of 6,283 gpm.

SECTION 5 CONCLUSIONS

The hydrologic and hydraulic analyses of the project site were used to develop a scour countermeasure design which is expected to provide protection to the piers and abutments of the existing Senn Road Bridge. Installing this countermeasure would reach the project goal of raising the NCI Code rating for this bridge to a 7 - Design Countermeasures Installed.

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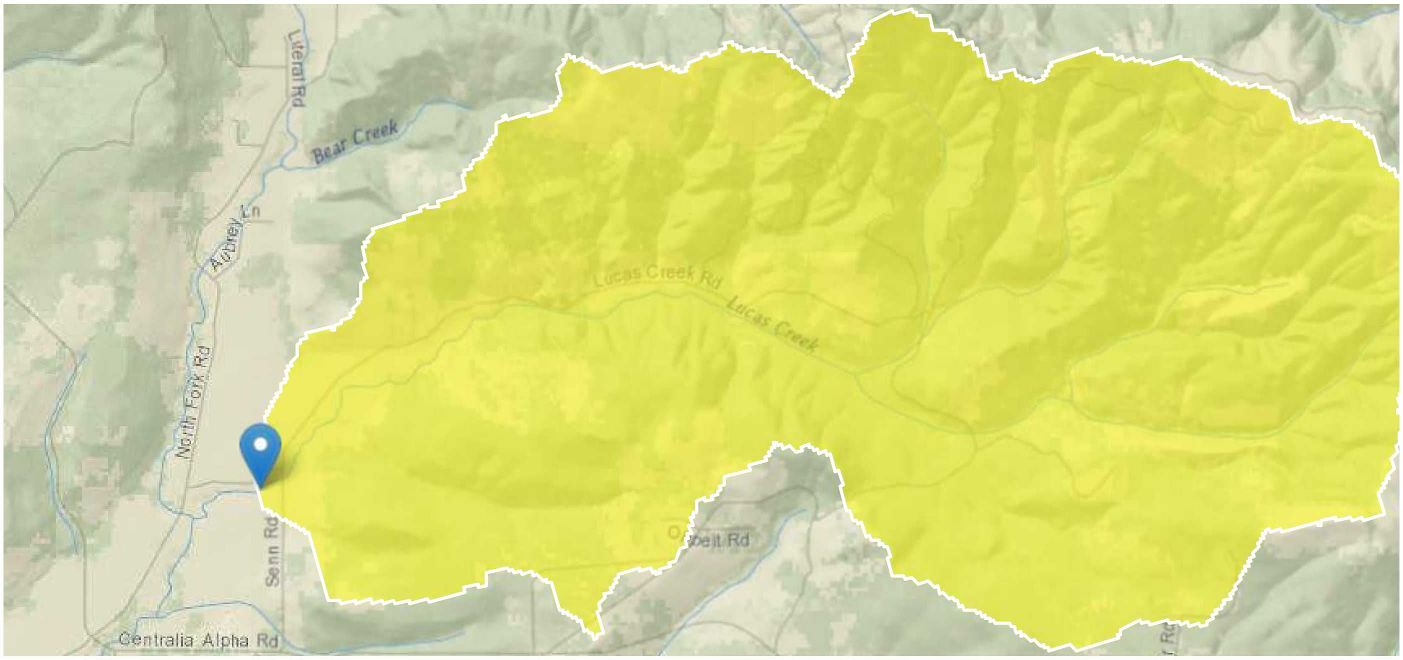
Attachments: **Appendix A: StreamStats Report**
Appendix B: Flood Q Regression and Ratio Tool
Appendix C: No-Rise Analysis
Appendix D: Original Bridge Plans
Appendix E: Proposed Plans
Appendix F: Revetment Rock Sizing Calculations
Appendix G: Pier Rock Sizing Calculations

Appendix A: StreamStats Report

StreamStats Report

Region ID: WA
Workspace ID: WA20250305190040916000
Clicked Point (Latitude, Longitude): 46.63823, -122.77406
Time: 2025-03-05 11:01:19 -0800



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
CANOPY_PCT	Percentage of drainage area covered by canopy as described in OK SIR 2009_5267	68.6	percent
DRNAREA	Area that drains to a point on a stream	13.14	square miles
PRECIP	Mean Annual Precipitation	56.5	inches
PRECPRIS10	Basin average mean annual precipitation for 1981 to 2010 from PRISM	58	inches

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Peak Region 4 2016 5118]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	13.14	square miles	0.18	2230
PRECPRIS10	Mean Annual Precip PRISM 1981 2010	58	inches	11.9	187

Peak-Flow Statistics Flow Report [Peak Region 4 2016 5118]

PIL: Lower 90% Prediction Interval, PIU: Upper 90% Prediction Interval, ASEp: Average Standard Error of Prediction, SE: Standard Error, PC: Percent Correct, RMSE: Root Mean Squared Error, PseudoR^2: Pseudo R Squared (other -- see report)

Statistic	Value	Unit	PIL	PIU	ASEp
50-percent AEP flood	425	ft^3/s	113	1600	52.5
20-percent AEP flood	697	ft^3/s	190	2560	50.6
10-percent AEP flood	914	ft^3/s	246	3390	50.5
4-percent AEP flood	1200	ft^3/s	308	4680	51.7
2-percent AEP flood	1430	ft^3/s	351	5830	52.9
1-percent AEP flood	1670	ft^3/s	392	7110	54.2
0.5-percent AEP flood	1900	ft^3/s	428	8430	55.5
0.2-percent AEP flood	2270	ft^3/s	477	10800	58

Peak-Flow Statistics Citations

Mastin, M.C., Konrad, C.P., Veilleux, A.G., and Tecca, A.E.,2016, Magnitude, frequency, and trends of floods at gaged and ungaged sites in Washington, based on data through water year 2014 (ver 1.1, October 2016): U.S. Geological Survey Scientific Investigations Report 2016–5118, 70 p. (http://dx.doi.org/10.3133/sir20165118)

Maximum Probable Flood Statistics

Maximum Probable Flood Statistics Parameters [Crippen Bue Region 17]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	13.14	square miles	0.1	10000

Maximum Probable Flood Statistics Flow Report [Crippen Bue Region 17]

Statistic	Value	Unit
Maximum Flood Crippen Bue Regional	48300	ft^3/s

Maximum Probable Flood Statistics Citations

Crippen, J.R. and Bue, Conrad D.1977, Maximum Floodflows in the Conterminous United States, Geological Survey Water-Supply Paper 1887, 52p. (https://pubs.usgs.gov/wsp/1887/report.pdf)

Application Version: 4.28.0

StreamStats Services Version: 1.2.22

NSS Services Version: 2.2.1

Appendix B: Flood Q Regression and Ratio Tool

Flood Q Regression Tool. Use to estimate flood discharge in Washington State at ungaged sites based on regional regression equations and user-determined basin characteristics.

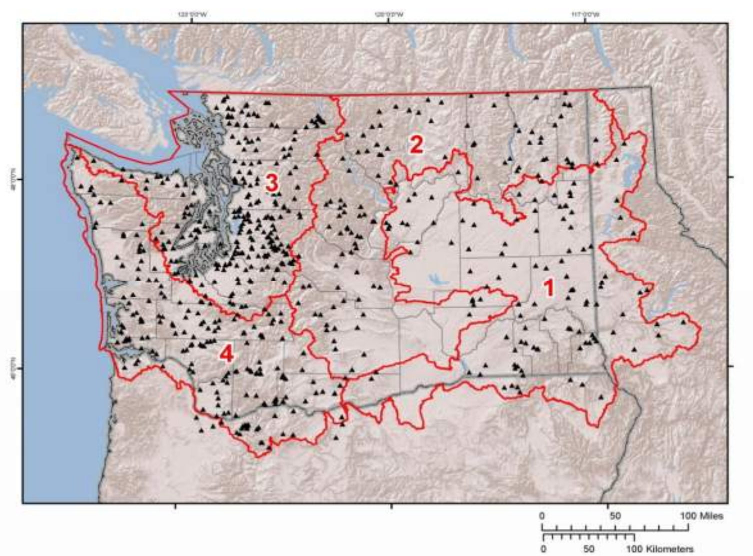
DA = Drainage Area, in square miles; P = Average Basin Annual Precipitation, in inches (from PRISM data set, years 1981-2010); CAN = Percent canopy cover (NLCD 2001); AEP = Annual Exceedance Probability; Qu = Flood Discharge, in cubic feet per second at ungaged site for the indicated AEP; PI_L, PI_U= Predition Intervals (L=Lower and U=Upper)

Instructions for using the Flood Q Tool to estimate Flood Discharges at Ungaged Sites using the regional regression equations

Steps	Instructions
1	Select the Regression Region below from the List Box
2	Determine the drainage area, *DA* and the Annual Precipitation, *P* for the ungage drainage basin. If you pick Regression Region 1 or 2, determine the percent canopy cover, *CAN*.
3	Enter these basin characteristic values in the green-shade cells. If the cell changes to red, than the value is outside the range of valid values for this regression. Valid value range listed to the right of the green cells.
4	Rows 23-30 will have the results. Estimated flood discharge, Qu, will be found in column O and the 90% prediction limits for these flood discharges will be found in columns R and T.

Regression Region 1
Regression Region 2
Regression Region 3
Regression Region 4

Regression Regions in Washington State



User determined basin characteristics for ungaged site

Selected Region: Regression Region 4

			Range of values that are valid for the regression
Drainage Area, *DA*	=	13.14 square miles	0.18 - 2230
Annual Precipitation, *P*	=	56.50 inches	11.94 - 186.6
Percent Canopy, *CAN*	=	68.6 %	value not used in regression

Selected Region: Regression Region 4

Estimate of indicated flood discharge for Regression Region 4 using regional regression equations

AEP		$^{*}Q_u$, ft^3/s	Prediction Intervals, 90% confidence level PI_L, in ft^3/s	PI_U, in ft^3/s
0.5	=	414	112.9	1517.9
0.2	=	683	190.9	2443.4
0.1	=	897	248.2	3241.2
0.04	=	1180	311.0	4477.4
0.02	=	1410	356.0	5584.4
0.01	=	1650	399.6	6812.5
0.005	=	1870	434.9	8041.6
0.002	=	2240	486.7	10308.5

*rounded to 3 significant figures

Flood Q Ratio Tool. Estimates of flood discharge for an ungaged sites in Washington State using the Drainage-Area Ratio Method

Station Index= 26

A_u, contributing drainage area of ungaged site; A_g, contributing drainage area of the streamflow gaging site; $Q_{(u)g}$, gage-based estimate of flood discharge in cubic feet per second at ungaged site; $Q_{(u)wtd}$, weighted estimate of flood discharge in cubic feet per second for the ungaged site and indicated streamflow characteristic; $Q_{(u)reg}$, regression estimate of flood discharge in cubic feet per second for ungaged site; Q(g)wtd, weighted estimate of flood discharge for streamgaging station]

Instructions for estimating streamflow characteristics at an ungaged site using the Drainage-Area Ratio Method

Steps	Instructions
1	Select a streamflow-gaging station from the list box below which is on the same stream as the ungaged site.
2	Determine contributing drainage area for ungaged site (A_u, in square miles) and enter value in the green cell (F12), where $0.5A_g \le A_u \le 1.5A_g$. If the drainage area is outside the appropriate range, the cell will turn red. An initial estimate of the flood discharges will be computed in column G, rows 23 to 30.
3	The estimates will be improved by computing weighted estimates using the regional regression estimates. To do this, enter the Mean Annual Precipitation in the green box (column I, row x). If the site is in regression region 1 or 2, enter the percent canopy coverage for the ungaged basin in the green box (column I, row x). If either of these values are outside the acceptible range, the cell will turn red.
4	The final answer, the weighted estimates of flood discharge ($Q_{(u)wtd}$), will be listed in column G, rows 37-44.

12016700 JOE CREEK NEAR COSMOPOLIS, WA
12017000 NORTH RIVER NEAR RAYMOND, WA
12019600 WATER MILL CREEK NEAR PE ELL, WA
12020000 CHEHALIS RIVER NEAR DOTY, WA
12020500 ELK CREEK NEAR DOTY, WA
12020800 SOUTH FORK CHEHALIS RIVER NEAR WILDWOOD, WA
12020900 SOUTH FORK CHEHALIS RIVER NEAR BOISTFORT, WA
12021000 SOUTH FORK CHEHALIS RIVER AT BOISTFORT, WA
12024000 SOUTH FORK NEWAUKUM RIVER NEAR ONALASKA, WA
12024400 NF NEWAUKUM RIVER ABV BEAR CREEK NR FOREST, WA
12025000 NEWAUKUM RIVER NEAR CHEHALIS, WA
12025300 SALZER CREEK NEAR CENTRALIA, WA
12025700 SKOOKUMCHUCK RIVER NEAR VAIL, WA
12026000 SKOOKUMCHUCK RIVER NEAR CENTRALIA, WA
12026150 SKOOKUMCHUCK RIVER BLW BLDY RUN CR NR CENTRALIA, WA
12026300 SKOOKUMCHUCK RIVER TRIBUTARY NEAR BUCODA, WA
12027500 CHEHALIS RIVER NEAR GRAND MOUND, WA
12029700 CHEHALIS RIVER NEAR OAKVILLE, WA
12030000 ROCK CREEK AT CEDARVILLE, WA
12031000 CHEHALIS RIVER AT PORTER, WA
12032500 CLOQUALLUM CREEK AT ELMA, WA
12034200 EAST FORK SATSOP RIVER NEAR ELMA, WA
12034700 WEST FORK SATSOP RIVER TRIBUTARY NEAR MATLOCK, WA
12035000 SATSOP RIVER NEAR SATSOP, WA

Estimates of streamflow characteristics based on Drainage-Area Ratio method

Contributing drainage areas of the streamflow-gaging station (A_g) and ungaged site (A_u), located on the same stream				Range of drainage areas for which drainage area adjustment is valid
Drainage area of selected index site, A_g	=	29.7	square miles	--
Drainage area of ungaged location, A_u	=	13.14	square miles	14.8 - 44.5

	Station Number	Number of Years of Record
Selected Streamflow-gaging Station:	12024400	16

Regression Region: 4

Initial estimate of Flood Discharges at ungaged site, Q_u (eq. 10)

AEP		$Q_{(g)wtd}$	$Q_{(u)g}$, ft³/s
0.5	=	1,610.0	766.2
0.2	=	2,810.0	1,335.0
0.1	=	3,700.0	1,755.0
0.04	=	4,920.0	2,328.0
0.02	=	5,870.0	2,773.0
0.01	=	6,840.0	3,228.6
0.005	=	7,820.0	3,685.1
0.002	=	9,210.0	4,329.5

$2(|A_g-A_u|)/A_g=$ 1.114853486

Regression Region: 4

Weighted estimate of Flood Discharge at ungaged site, using regression estimate, $Q_{(u)wtd}$ (eq. 11)

AEP		$Q_{(u)reg}$	*$Q_{(u)wtd}$
0.5	=	414.5	374
0.2	=	682.8	608
0.1	=	896.6	798
0.04	=	1,178.7	1050
0.02	=	1,406.2	1250
0.01	=	1,649.0	1470
0.005	=	1,874.4	1670
0.002	=	2,240.1	2000

*rounded to 3 significant figures

Basin Characteristics for Regional Regression Estimate, $Q_{(u)reg}$

		Applicable Range
Enter Mean Annual Precipitation of ungaged basin=	56.5	11.94 - 186.6
If ungaged location is in regression region 1 or 2, enter percent canopy coverage=	0.8	not applicable

Appendix C: No-Rise Analysis

No-Rise Analysis

From: Nick Cook <Nick.Cook@otak.com>
Sent: Tuesday, March 3, 2026 8:25 AM
To: Mike Zarecor <mikez@osbornconsulting.com>; Doug Sarkkinen <doug.sarkkinen@otak.com>
Subject: RE: Senn and Wildwood Scour Repair

CAUTION: This email originated from outside of the organization. Do not click links or open attachments unless you know the content is safe.

Hi Mike –

We were able to get there for the Senn site with a 1D model (results below). Not sure how much detail you need to add it in your report, but I can provide some text if you'd like. Otherwise it could just be "Otak performed a 1D no-rise analysis of the project reach, results are provided…."

Reach	River Sta	Profile	Plan	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Wic (ft)
Reach 1	498	100-Year	EC_Updated	1650.00	350.46	360.14		360.42	0.001248	4.92	524.47	108.
Reach 1	498	100-Year	PC_Updated	1650.00	350.46	359.79		360.11	0.001479	5.22	488.06	103.
Reach 1	452	100-Year	EC_Updated	1650.00	351.41	359.58		360.28	0.003750	6.80	263.34	51.
Reach 1	452	100-Year	PC_Updated	1650.00	351.41	359.13		359.94	0.006161	7.32	240.68	48.
Reach 1	389	100-Year	EC_Updated	1650.00	351.43	359.22		360.02	0.004442	7.42	265.51	58.
Reach 1	389	100-Year	PC_Updated	1650.00	351.43	358.34		359.49	0.007451	8.76	216.07	54.
Reach 1	313	100-Year	EC_Updated	1650.00	349.79	358.85	356.36	359.78	0.005087	7.75	219.32	38.
Reach 1	313	100-Year	PC_Updated	1650.00	349.18	358.81	354.33	359.12	0.001230	4.59	435.85	88.
Reach 1	280			Bridge								
Reach 1	252	100-Year	EC_Updated	1650.00	347.81	357.95		358.66	0.003967	6.81	249.37	44.
Reach 1	252	100-Year	PC_Updated	1650.00	348.69	357.93		358.66	0.003664	6.94	250.07	44.
Reach 1	178	100-Year	EC_Updated	1650.00	348.18	357.78		358.26	0.002225	5.59	312.99	63.
Reach 1	178	100-Year	PC_Updated	1650.00	348.18	357.78		358.26	0.002225	5.59	312.99	63.
Reach 1	127	100-Year	EC_Updated	1650.00	348.55	357.72		358.10	0.002091	5.27	382.85	131.
Reach 1	127	100-Year	PC_Updated	1650.00	348.55	357.72		358.10	0.002091	5.27	382.85	131.
Reach 1	49	100-Year	EC_Updated	1650.00	349.45	356.75	356.11	357.77	0.007008	9.05	321.32	147.
Reach 1	49	100-Year	PC_Updated	1650.00	349.45	356.75	356.11	357.77	0.007008	9.05	321.32	147.

Nicholas Cook, PhD, PE | Water Resources

808 SW Third Ave., Suite 800 | Portland, OR 97204
Direct: 503.415.2362 | | Cell: 828.291.8004 Main: 503.287.6825
www.otak.com

From: Mike Zarecor <mikez@osbornconsulting.com>
Sent: Tuesday, December 30, 2025 11:24 AM
To: Doug Sarkkinen <doug.sarkkinen@otak.com>
Cc: Nick Cook <Nick.Cook@otak.com>
Subject: RE: Senn and Wildwood Scour Repair

Hi Doug and Nick,

Following up on this, we just finished with the 65% plans and a few minor grading adjustments as we wrapped that up. We will have the 2D model updated with those latest adjustments early next week and will send it your way.

Best,

Mike

Osborn Consulting

Mike Zarecor, PE
Civil Engineer

1499 SE Tech Center Place, Suite 160
Vancouver, WA 98683
direct. (360) 335-2136
office. (509) 867-3654
email. mikez@osbornconsulting.com

www.osbornconsulting.com

Appendix D: Original Bridge Plans (1966)

GENERAL NOTES

All materia & workmanship sh be in accordance with the requirements - the State of Washington, Departmen of Highways, Standard Specifications for ad and Bridge Construction dated 1963.

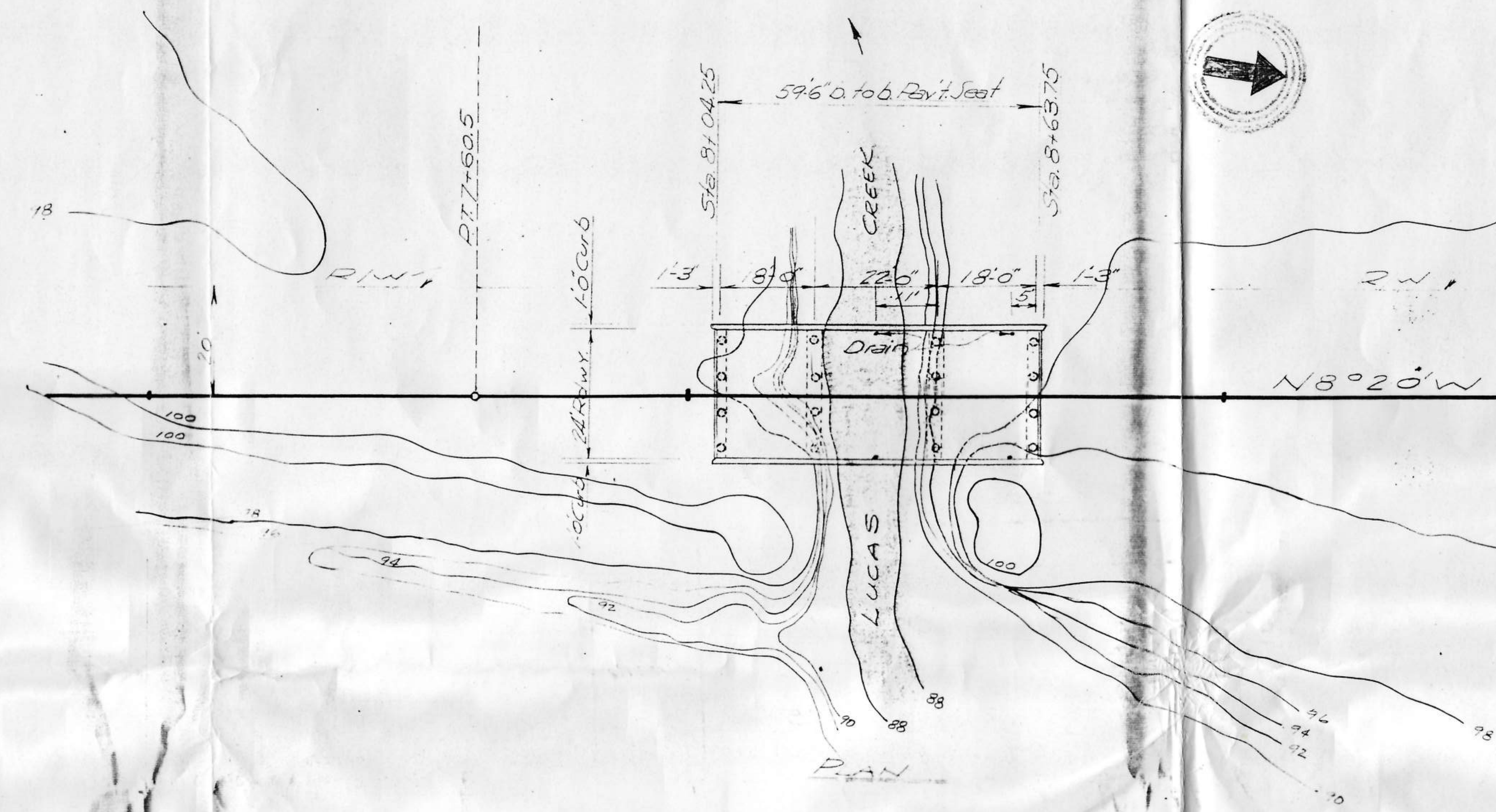
All concrete shall be Class A mix

Each pile shall be driven to a dep sufficient to develop a minimum load bearin capacity of twenty (20) tons.

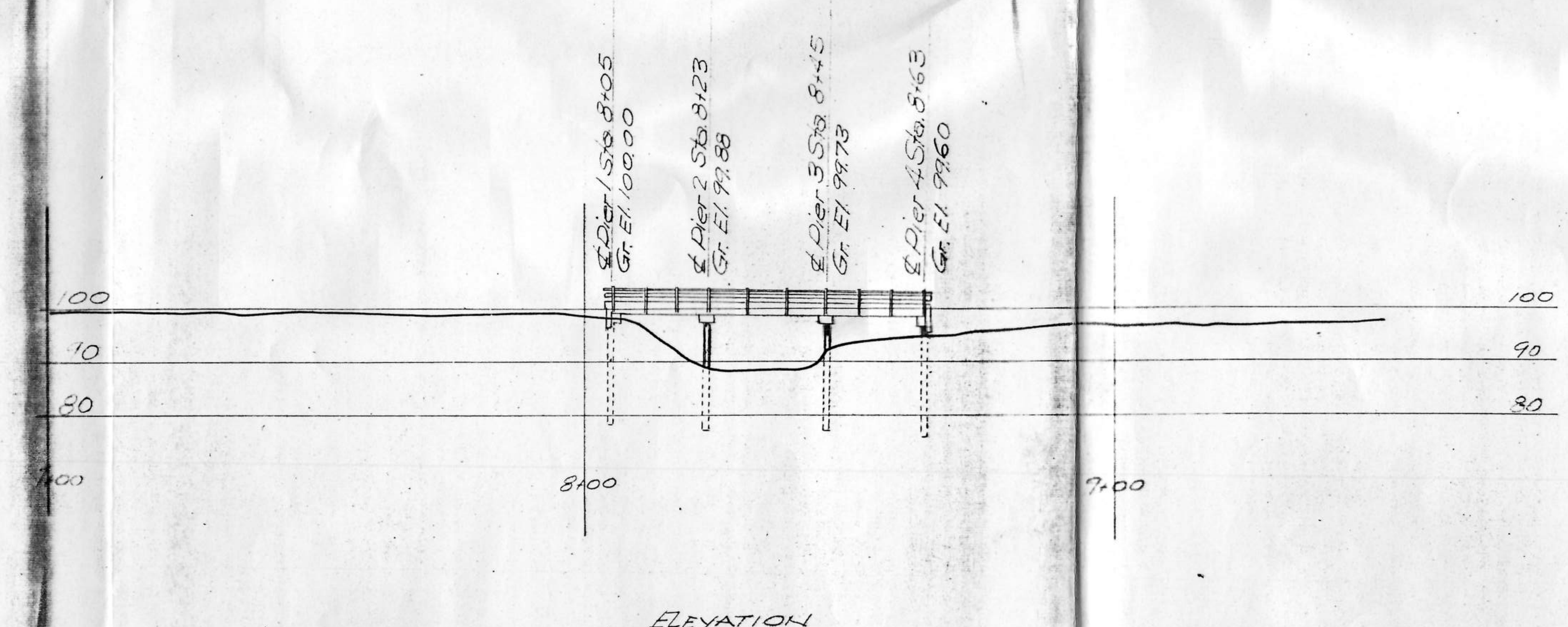
Falsework shall not be release in any span until all concrete has been in plac the required length of time and has develope sufficient strength as outlined in the specif ations

Falsework shall be carefully re ased to prevent impact or undue stresses in the structure.

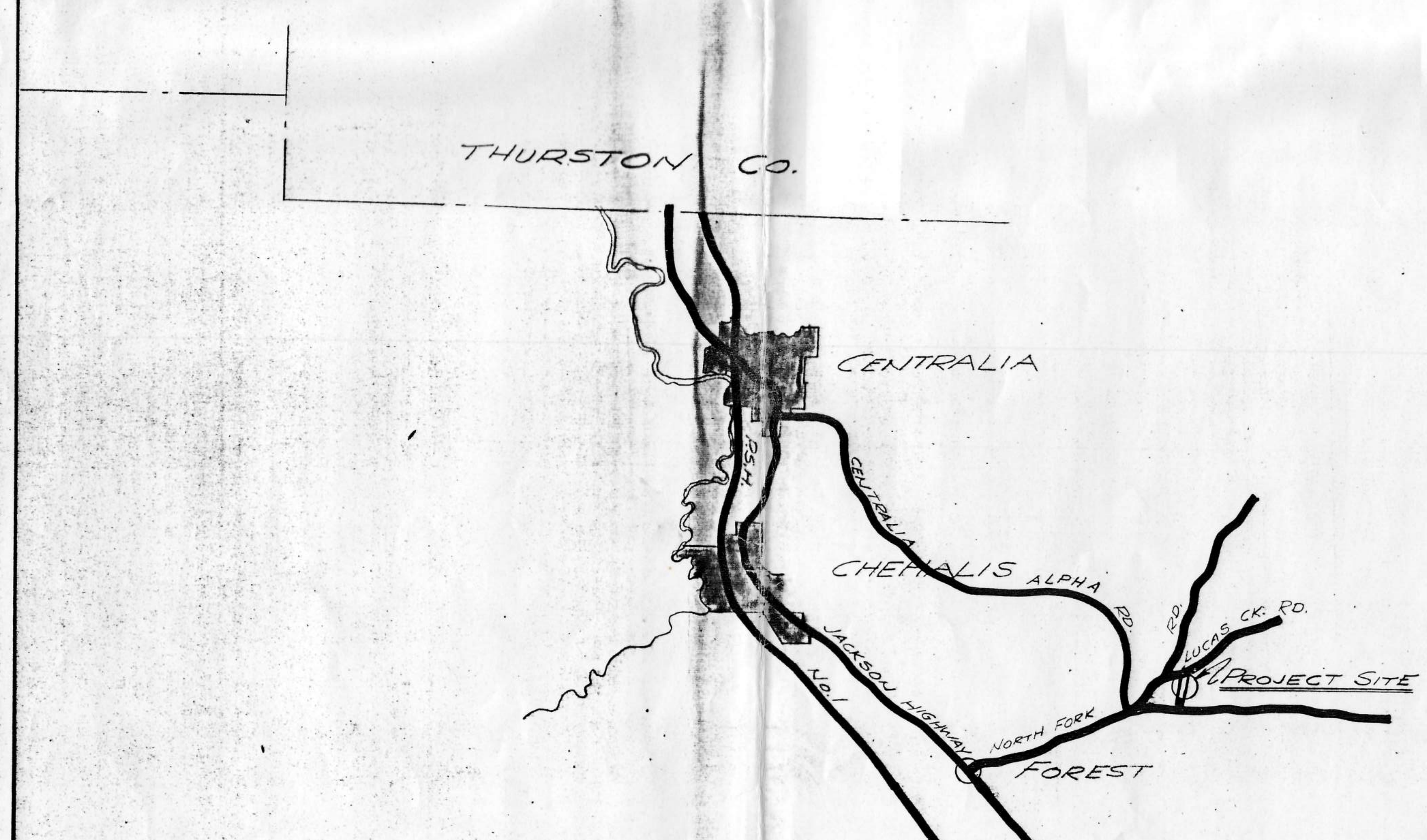
SUMMARY OF QUANTITIES .1 Lin. Ft. = 0.01 Mile			
ITEM No.	ITEM	QUANTITY	UNIT
1.	Superstructure	Lump Sum	L.S.
2.	Special Bridge Railing	122	L.F.
3.	Furn & Drive Concrete Test Pile	1	Only
4.	Furnishing Concrete Piling	450	L.F.
5.	Driving Concrete Piles	15	Only
6.	Removing Existing Structure	Lump Sum	L.S.



PLAN



ELEVATION



COUNTY ROAD PROJECT NO. 813

#77

LUCAS CREEK BRIDGE

PAUL BOURGAULT STATE OF WASHINGTON 8522 REGISTERED PROFESSIONAL ENGINEER

County Road Engineer March, 1966

Sheet 4 of 6 Sheets

PAUL BOURGAULT STATE OF WASHINGTON 8522 REGISTERED PROFESSIONAL ENGINEER

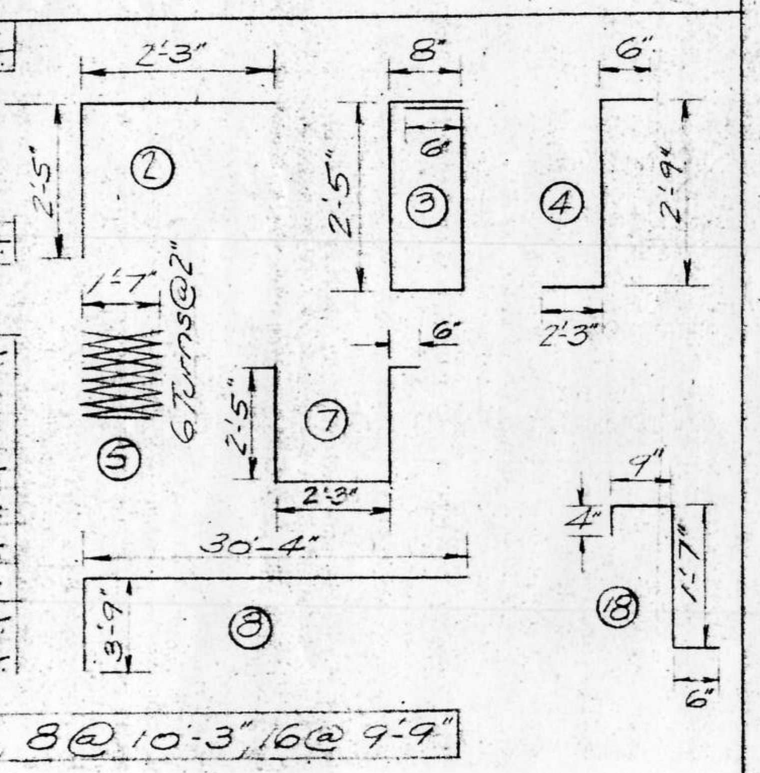
COUNTY ROAD PROJECT NO. 813

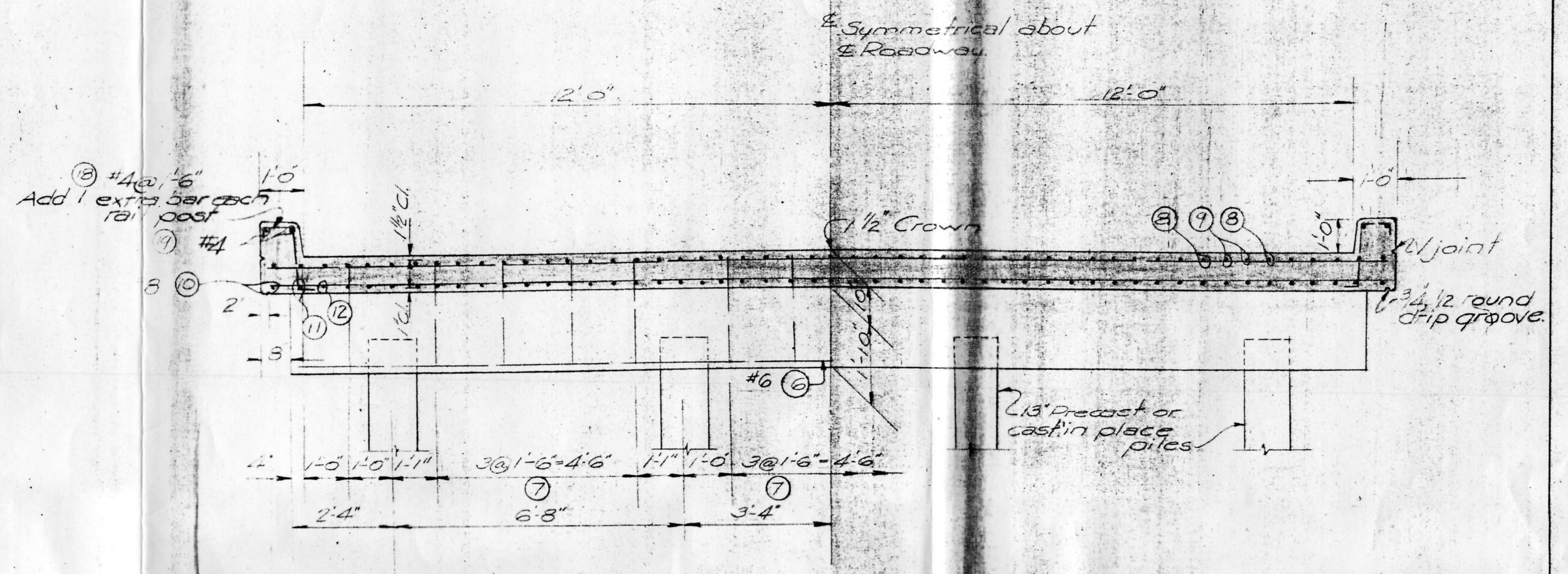
LUCAS CREEK BRIDGE

Paul Bourgault
County Road Engineer March 1966
Sheet 5 of 6 Sheets

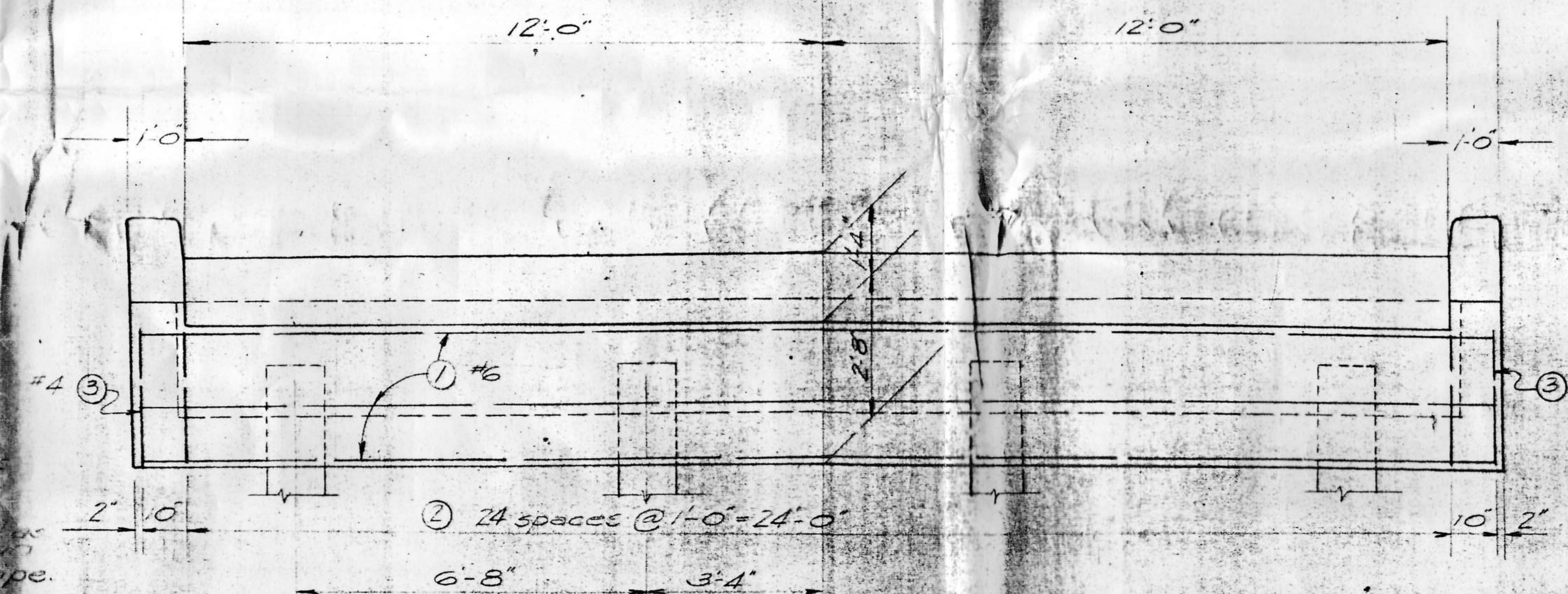
1.	Endwall, Transv.	12	5	25'-8"	Str.
2.	Endwall, Vert.	50	4	4'-8"	
3.	" , Vert.	4	4	6'-8"	
4.	Caps, Piers 1&4, Vert.	32	4	5'-6"	
5.	" , Pile Hoops	16	2	30'-0"	
6.	All Caps, Horizontal	12	6	24'-4"	Str.
7.	Caps, Interm. Pier Stirrup	32	4	8'-1"	
8.	Roadway Slab, Long. Top	34	8	34'-1"	
9.	" " " Top	68	8	3'-6"	Str.
10.	" " " Bott.	34	8	9'-3"	Str.
11.	" " " Bott.	34	8	15'-6"	Str.
12.	" " " Bott.	34	8	17'-9"	Str.
13.	" " " Bott.	17	8	23'-6"	Str.
14.	" " " Bott.	17	8	15'-0"	Str.
15.	" " " Bott.	17	8	20'-6"	Str.
16.	" " Trans. Top	42	5	25'-8"	Str.
17.	" " " Bott.	46	6	25'-8"	Str.
18.	Curb, Ties	100	4	3'-2"	
19.	" , Long.	24	4	Varies, 8@10'-3", 16@9'-9"	

BAR LIST





SECTION - INTERMEDIATE CAP



END ELEVATION

GENERAL NOTES

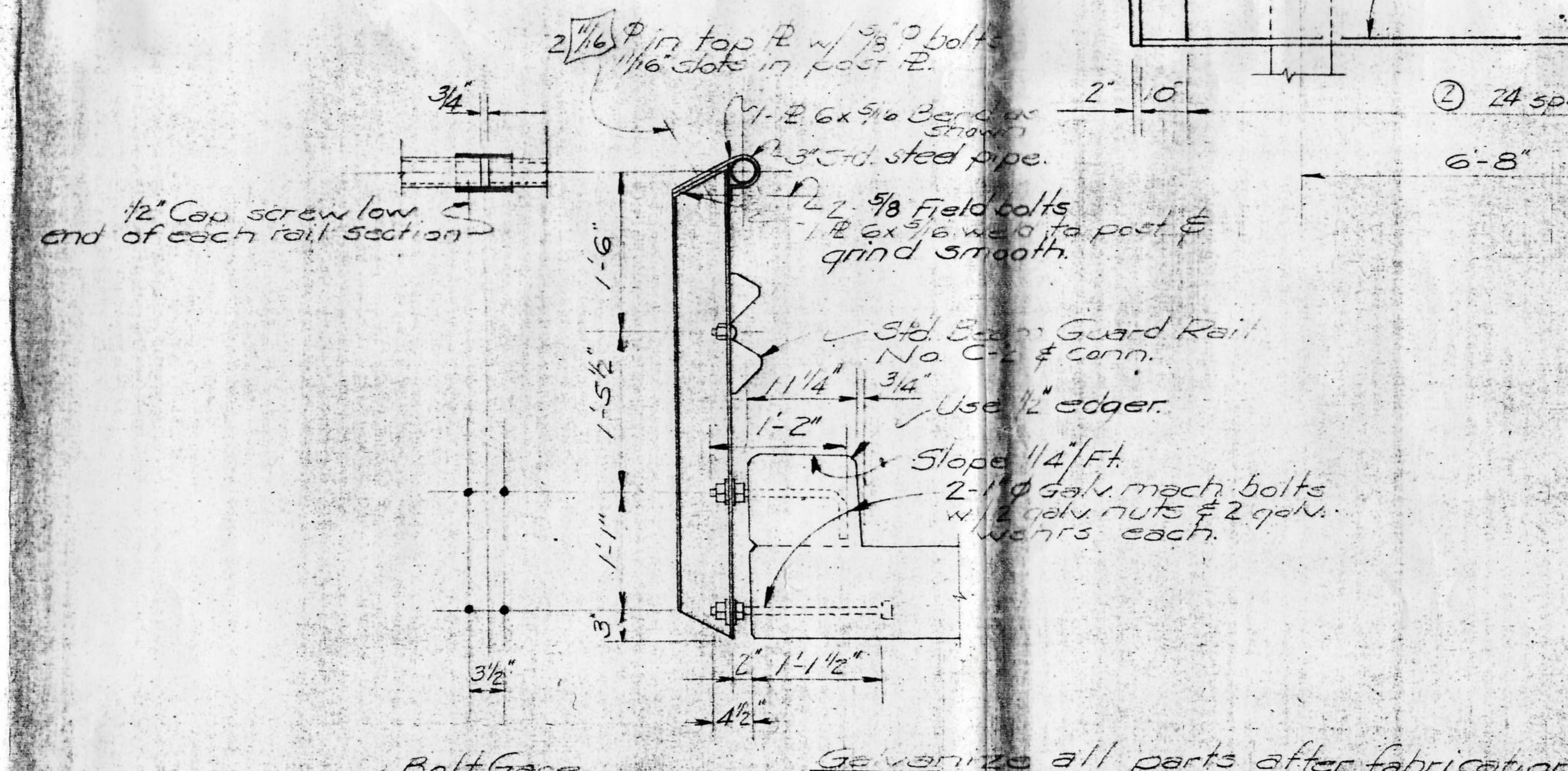
All material & workmanship shall be in accordance with the requirements of the State of Washington, Department of Highways, Standard Specifications for Road & Bridge Construction dated 1963.

All concrete shall be Class A mix.

Each pile shall be driven to a depth sufficient to develop a minimum load bearing capacity of twenty (20) tons.

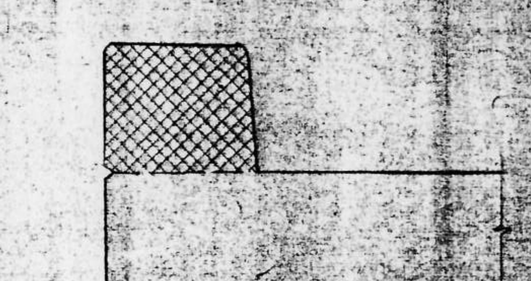
Falsework shall not be released in any span until all concrete has been in place the required length of time and has developed sufficient strength as outlined in the specifications.

Falsework shall be carefully released to prevent impact or undue stresses in the structure.



Galvanize all parts after fabrication

BRIDGE RAIL DETAIL



NOTE:
Place 1/4 joint with asphaltic filler over crosshatched area at points shown on plan sheets. Fasten to one side with 8d nails at 3' staggered.

COUNTY ROAD PROJECT NO. 813

LUCAS CREEK BRIDGE

PAUL BOURGAULT
STATE OF WASHINGTON
8522
REGISTERED
PROFESSIONAL ENGINEER

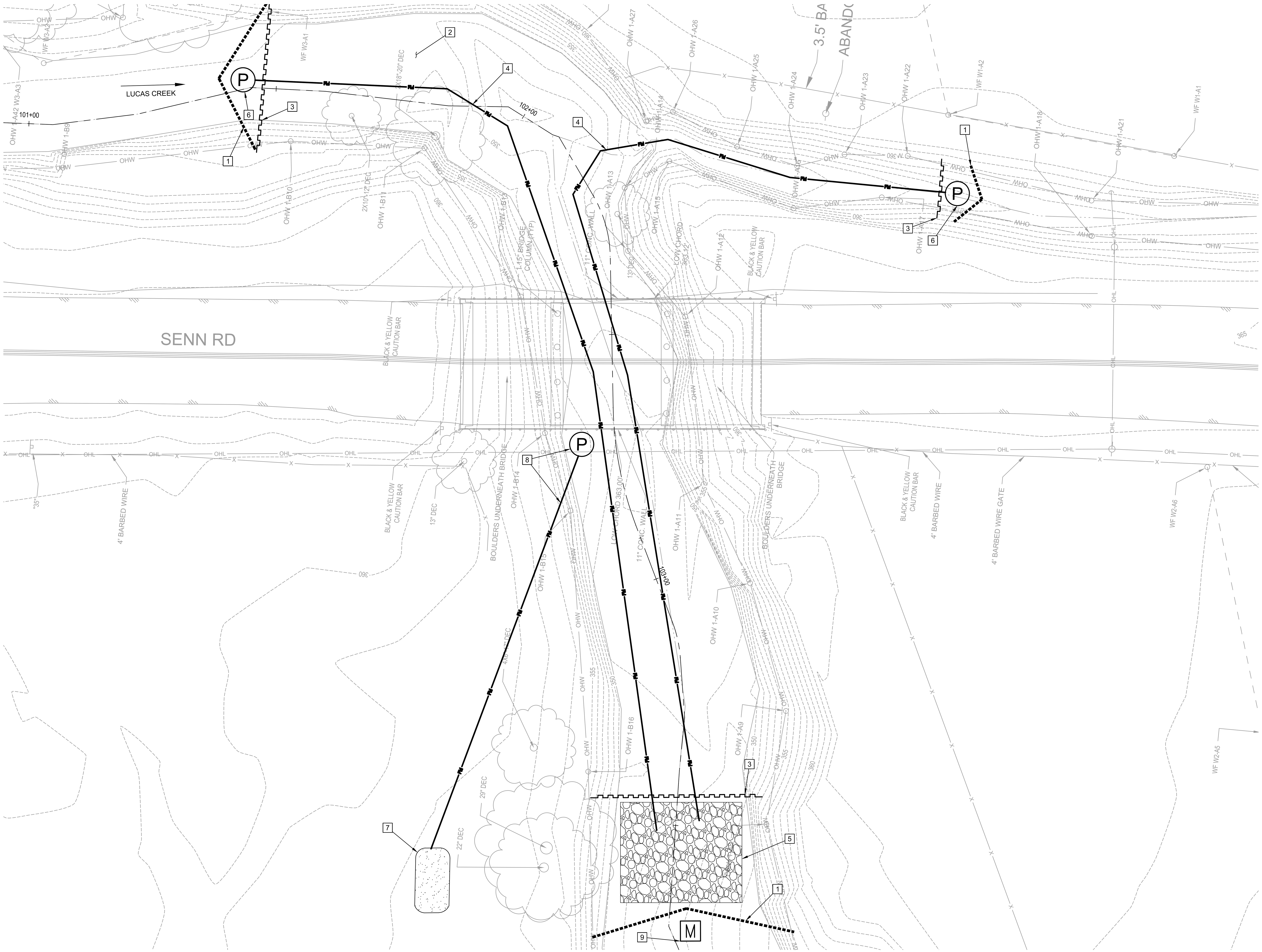
Paul Bourgault
County Road Engineer

March, 1966

Sheet 6 of 6 Sheets

18-89-1

Appendix E: Proposed Plans



GENERAL NOTE:

1. ALL MATERIAL AND WORK SHALL BE IN ACCCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION, "STANDARD SPECIFICATIONS FOR ROAD, BRIDGE, AND MUNICIPAL CONSTRUCTION" 2026 EDITION AND SPECIAL PROVISIONS.

TEMPORARY STREAM DIVERSION NOTES:

1. THE TEMPORARY STREAM DIVERSION SHOWN IS CONCEPTUAL. SEE SPECIAL PROVISIONS. THE TEMPORARY STREAM DIVERSION MAY BE PHASED IN ORDER TO UTILIZE THE EXISTING STREAM CHANNEL TO THE EXTENT FEASIBLE DURING CONSTRUCTION.
2. CONTRACTOR SHALL SUBMIT THE TEMPORARY STREAM DIVERSION PLAN TO THE ENGINEER FOR REVIEW AND APPROVAL WITHIN 7 DAYS OF NOTICE TO PROCEED.
3. THE TEMPORARY STREAM DIVERSION SHALL BE INSPECTED DAILY BY THE CONTRACTOR AND MAINTAINED TO ENSURE CONTINUED PROPER FUNCTIONING.
4. THE STREAM DIVERSION SYSTEM SHALL BE CAPABLE OF CONVEYING THE FLOW SPECIFIED IN THE CONTRACT DOCUMENTS.
5. FISH EXCLUSION AND FISH REMOVAL SHALL BE PERFORMED BY THE CONTRACTOR, BEFORE IN-WATER WORK IN ACCORDANCE WITH THE WASHINGTON DEPARTMENT OF FISH AND WILDLIFE HPA.

TEMPORARY STREAM DIVERSION KEY NOTES:

1 INSTALL FISH SCREEN.

2 CAPTURE AND REMOVE ALL FISH BETWEEN THE FISH SCREENS IN ACCORDANCE WITH THE HPA. FISH CAPTURE SHALL BE PERFORMED UNDER THE SUPERVISION OF AN EXPERIENCED FISHERY BIOLOGIST.

3 INSTALL COFFERDAM. SEE DETAIL 1 ON SHEET 6. INSTALL COFFERDAMS AT STA 101+47.1 AND STA 103+44.1.

4 INSTALL TEMPORARY STREAM DIVERSION PIPE. ADJUST LOCATION OF DIVERSION PIPE AS NEEDED TO PERFORM WORK.

5 INSTALL OUTLET PROTECTION TO PREVENT EROSION AT TEMPORARY STREAM DIVERSION OUTFALL. OUTLET PROTECTION SHALL BE REMOVED WHEN CONSTRUCTION IS COMPLETE. SEE DETAIL 1 ON SHEET 6.

6 INSTALL STREAM BYPASS PUMP(S) AND PIPES AS NEEDED DURING CONSTRUCTION. ADJUST LOCATION OF BYPASS PUMPS AND PIPES AS NEEDED TO PERFORM WORK.

7 INSTALL FILTER BAG FOR SEDIMENT REMOVAL FROM ANY DEWATERING ACTIVITIES THAT MAY BE NECESSARY DURING CONSTRUCTION. SEE DETAIL 2 ON SHEET 6.

8 INSTALL DEWATERING PUMP AND PIPES AS NEEDED DURING CONSTRUCTION. ADJUST LOCATION OF DEWATERING PUMPS AND PIPES AS NEEDED TO PERFORM WORK.

9 TURBIDITY MONITORING STATION. SEE SPECIAL PROVISIONS FOR TURBIDITY MONITORING REQUIREMENTS.

LEGEND

COFFER DAM

FISH SCREEN

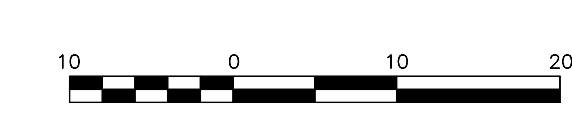
FORCE MAIN

PUMP

TURBIDITY MONITORING POINT

TEMPORARY QUARRY SPALLS

FILTER BAG



Lewis County Department of Public Works
57 W. MAIN ST.
CHEHALIS WA 98532
PHONE # (360) 740-1123
FAX # (360) 740-2719

DESIGNED BY : IMF
DRAWN BY : JAR
CHECKED BY : LCR
DATE : 05/14/202

NO.	DATE	REVISION	BY	APP.

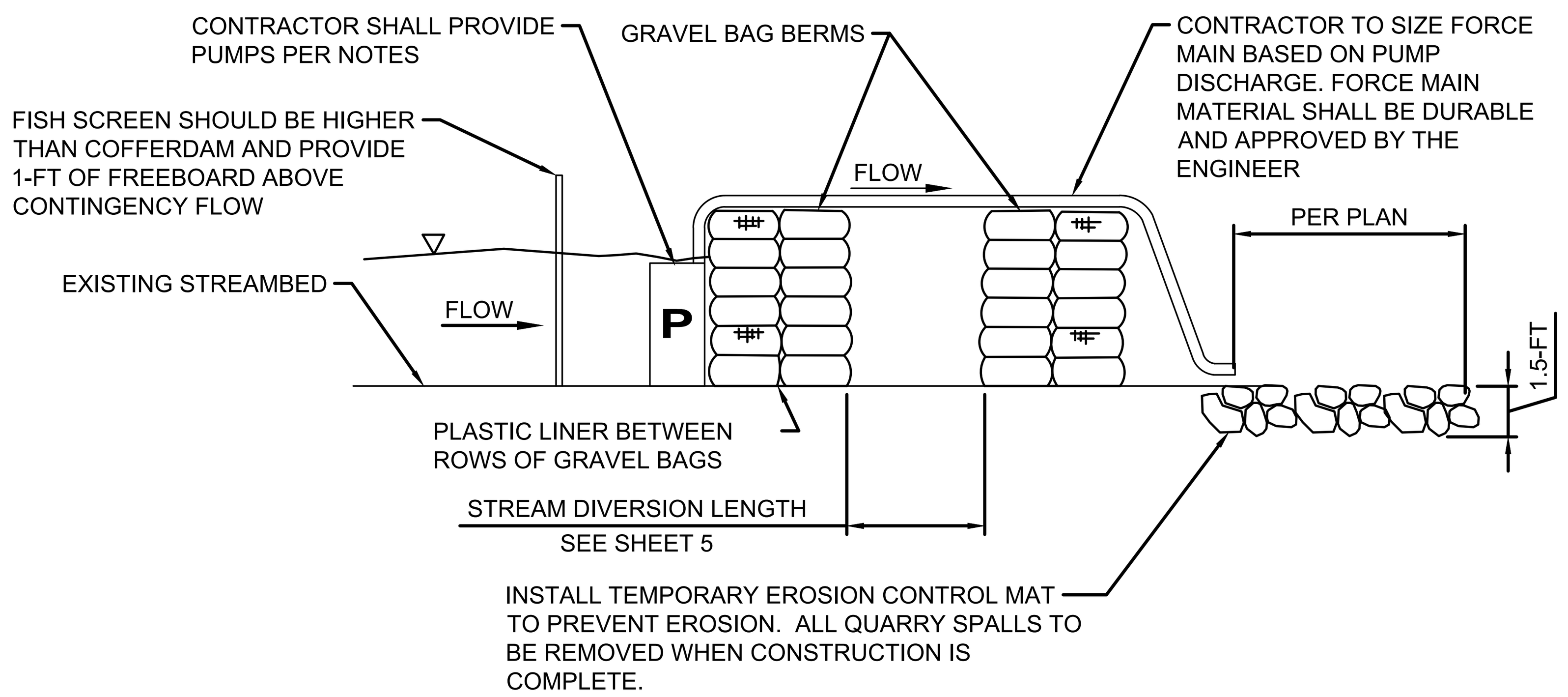
SENN MP 0.87 BRIDGE SCOUR REPAIR

99% CONSTRUCTION PLANS
COUNTY ROAD PROJECT NO: 2159R
TEMPORARY STREAM DIVERSION PLAN

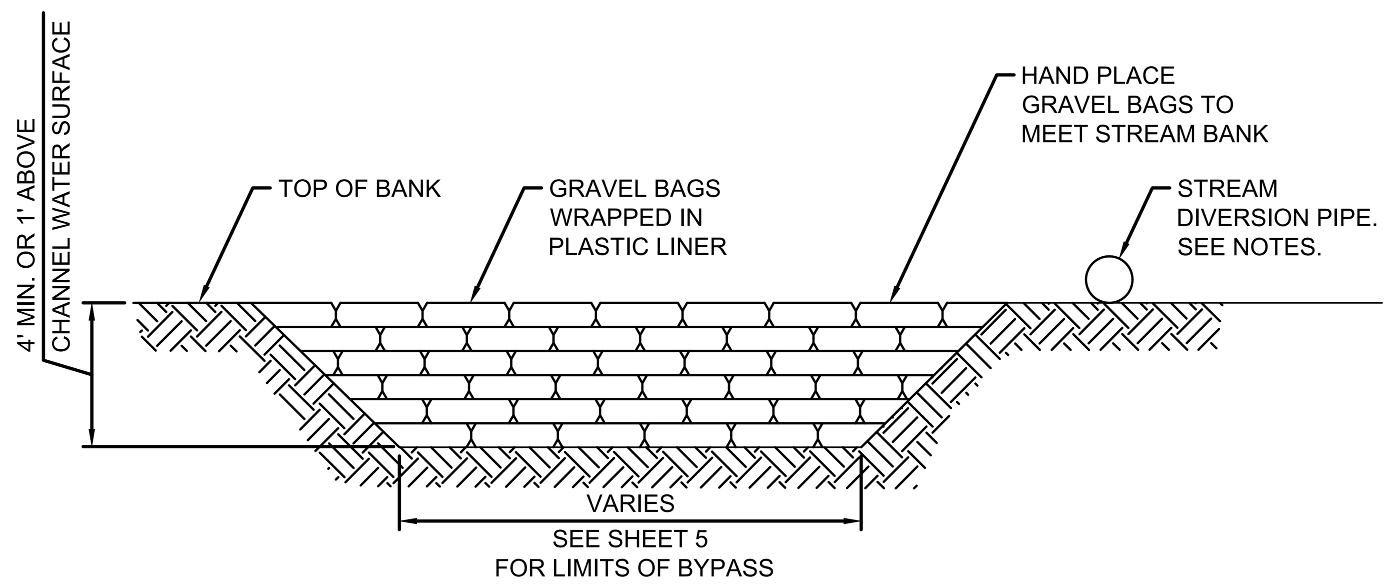
SHEET 5 OF 15

CALL 48 HOURS BEFORE YOU DIG 1-800-424-5555 "It's the Law" Utilities Underground Location Center

Osborn Consulting



SIDE VIEW

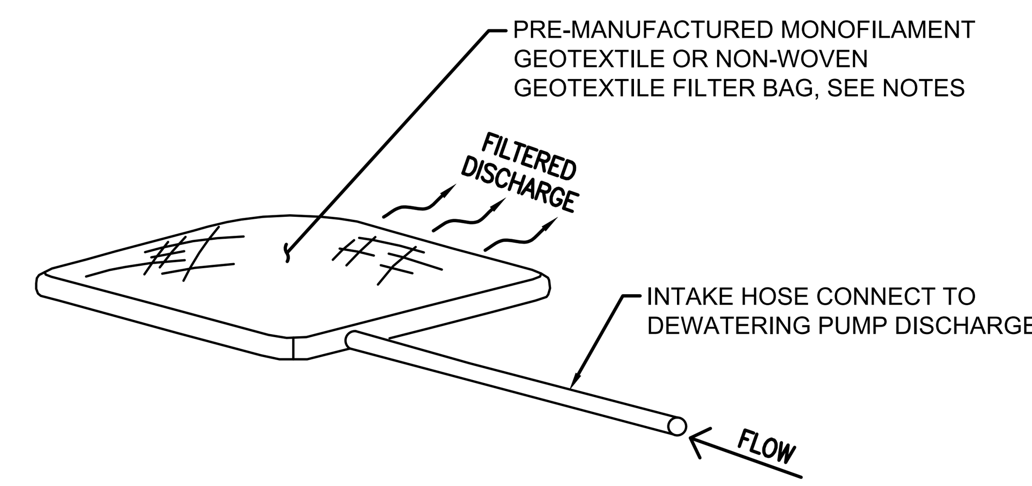


FRONT VIEW

NOTES:

1. COFFERDAM IS CONCEPTUAL. CONTRACTOR MAY USE ALTERNATIVE AS APPROVED BY ENGINEER.
2. COFFERDAM SHALL BE CONSTRUCTED BY THE CONTRACTOR WITH ROUNDED PEA GRAVEL BAGS, OR OTHER MEANS PRE-APPROVED BY THE ENGINEER. GRAVEL BAGS SHALL BE CONSTRUCTED OF WOVEN SYNTHETIC FIBER. NO ANGULAR OR CRUSHED MATERIALS MAY BE USED. GRAVEL BAGS SHALL BE WASTEHAULED AFTER REMOVAL OF TEMPORARY STREAM DIVERSION.
3. THE HEIGHT AND WIDTH OF THE COFFERDAM SHALL BE DETERMINED BY THE CONTRACTOR BASED ON THE WATER SURFACE ELEVATION AND CHANNEL SHAPE AT THE TIME OF CONSTRUCTION. THE COFFERDAMS SHALL BE REVIEWED AND APPROVED BY THE ENGINEER. THE AVERAGE MAXIMUM DAILY FLOW IS APPROXIMATELY 14 CFS.
4. REMOVE LOOSE COBBLES AND BOULDERS FROM THE STREAMBED PRIOR TO INSTALLING COFFERDAM.
5. EXTEND BOTH ENDS OF THE COFFERDAM UP THE BANKS OF THE CHANNEL AS NEEDED TO PREVENT THE STREAM FROM END-RUNNING THE COFFERDAM.
6. PUMP INTAKE SCREEN SHALL BE BACKWATERED WITH THE PLACEMENT OF THE GRAVEL BAG BERM TO PREVENT FISH FROM GETTING IMPINGED OR ENTRAINED ON THE INTAKE SCREEN.
7. COFFERDAM SHALL BE MAINTAINED BY THE CONTRACTOR AND IMPROVED AS NECESSARY AND DIRECTED BY THE ENGINEER.
8. CONTRACTOR SHALL HAVE ON SITE, EMERGENCY DIVERSION PIPE, PUMPS, GENERATOR, AND APPURTENANCES IN THE EVENT OF HIGH FLOWS AS WELL AS BACKUP PUMP(S) IN CASE OF PRIMARY PUMP FAILURE. EMERGENCY DIVERSION PUMP INTAKE SHALL BE SCREENED TO PREVENT FISH FROM ENTERING DIVERSION SYSTEM. SEE THE CONTRACT PROVISIONS AND HPA FOR ADDITIONAL INFORMATION ON TEMPORARY STREM DIVERSION.
9. COFFERDAM MATERIALS SHALL BE REMOVED FROM THE SITE AND BECOME PROPERTY OF CONTRACTOR.

1 / 5 COFFER DAM DETAIL
NOT TO SCALE

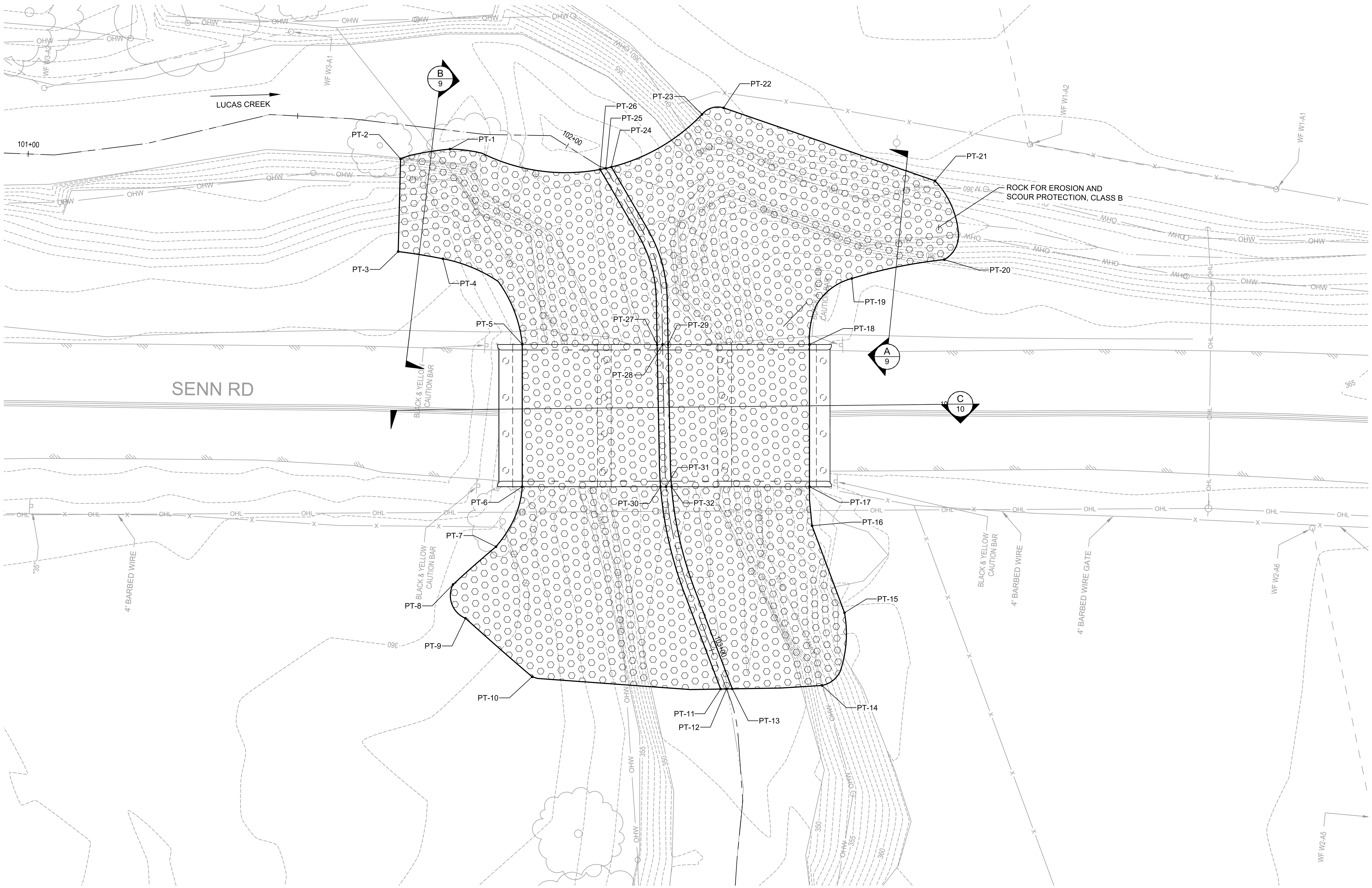


NOTES:

1. FILTER BAG SHALL BE MINIMUM 10-FT X 15-FT AND REPLACED AS NEEDED TO ACCOMMODATE ACTUAL SEDIMENT LOAD CONDITIONS (I.E. VOLUME, TYPE OF SEDIMENT, ETC.)
2. DRAIN FILTER BAG TO COUNTY APPROVED RECEIVING AREA. RECEIVING AREA SHALL BE VEGETATED AND UPLAND OF THE OHWM AND TURBIDITY MONITORING STATION.
3. FILTER BAG SYSTEM TO BE INSPECTED DAILY WHEN IN USE TO VERIFY ADEQUATE PERFORMANCE.
4. TURBIDITY MONITORING STATION INSTALLED AND MAINTAINED PER PERMIT REQUIREMENTS (TO BE DETERMINED), SHOULD A TURBIDITY EXCEEDENCE OCCUR, THE FILTER BAG SYSTEM IS TO BE INSPECTED IMMEDIATELY AND REPLACED IF ANOTHER SEDIMENT SOURCE IS NOT IDENTIFIED.

2 / 5 FILTER BAG DETAIL
NOT TO SCALE

Lewis County Department of Public Works	57 W. MAIN ST. CHEHALIS WA 98532 PHONE # (360) 740-1123 FAX # (360) 740-2719	DESIGNED BY : IMF DRAWN BY : JAR CHECKED BY : LCR DATE : 05/14/202	NO.	DATE	REVISION	BY	APP.	SENN MP 0.87 BRIDGE SCOUR REPAIR	99% CONSTRUCTION PLANS COUNTY ROAD PROJECT NO: 2159R TEMPORARY STREAM DIVERSION DETAILS	SHEET 6 OF 15	CALL 48 HOURS BEFORE YOU DIG 1-800-424-5555 "It's the Law" Utilities Underground Location Center	Osborn Consulting



SCOUR COUNTERMEASURE NOTES:

1. SEE SCOUR COUNTERMEASURE LAYOUT TABLE ON THIS SHEET FOR LAYOUT POINTS AND TOP OF COUNTERMEASURE ELEVATIONS.
2. SEE SHEET 8 FOR SCOUR COUNTERMEASURE PROFILE.
3. SEE SHEETS 9 AND 10 FOR SCOUR COUNTERMEASURE SECTIONS.
4. MINIMIZE DISTURBANCE OF THE STREAMBED OUTSIDE OF THE SCOUR COUNTERMEASURE FOOTPRINT TO THE MAXIMUM EXTENT PRACTICABLE. STREAMBED OUTSIDE THE SCOUR COUNTERMEASURE LIMITS THAT IS DISTURBED SHALL BE RETURNED TO PRE-CONSTRUCTION CONDITIONS.
5. SEE THIS SHEET FOR ALIGNMENT LAYOUT TABLE.
6. FILL VOIDS IN ROCK FOR EROSION AND SCOUR PROTECTION CLASS B WITH BLENDED STREAMBED AGGREGATE (APPROXIMATELY 383 TONS)

LEGEND

ROCK FOR EROSION AND SCOUR PROTECTION CLASS B
1,485 TON

CONTROL POINT TABLE

Point ID	STA	OFFSET	TOP OF COUNTER MEASURE ELEVATION
PT-1	101+78.4	3.4' RT	348.9'
PT-2	101+69.6	6.2' RT	349.0'
PT-3	101+71.2	23.1' RT	360.0'
PT-4	101+79.4	23.5' RT	360.0'
PT-5	102+42.3	25.6' RT	360.0'
PT-6	102+67.3	26.2' RT	360.1'
PT-7	102+74.6	32.2' RT	360.0'
PT-8	102+78.1	41.2' RT	359.9'
PT-9	102+82.3	40.5' RT	359.6'
PT-10	102+93.0	32.4' RT	358.5'
PT-11	103+07.3	1.0' RT	345.9'
PT-12	103+07.6	0.0' CT	345.9'
PT-13	103+08.0	1.0' LT	345.9'
PT-14	103+11.6	16.8' LT	355.0'
PT-15	103+02.2	25.1' LT	360.0'
PT-16	102+83.4	25.1' LT	360.0'
PT-17	102+72.2	26.1' LT	360.3'
PT-18	102+43.4	26.8' LT	360.9'
PT-19	102+33.2	34.9' LT	360.1'
PT-20	102+32.4	52.0' LT	360.2'
PT-21	102+27.8	53.1' LT	360.3'
PT-22	102+09.5	24.2' LT	360.1'
PT-23	102+08.5	20.2' LT	358.5'
PT-24	102+08.5	1.0' LT	346.6'
PT-25	102+08.2	0.0' CT	346.6'
PT-26	102+07.3	0.5' RT	346.6'
PT-27	102+42.8	1.0' RT	345.9'
PT-28	102+42.8	0.0' CT	345.9'
PT-29	102+42.9	1.0' LT	345.9'
PT-30	102+68.8	1.0' RT	345.6'
PT-31	102+68.9	0.0' CT	345.6'
PT-32	102+68.9	1.0' LT	345.6'

STREAM CENTERLINE DATA ON SHEET 11

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SENN MP 0.87 BRIDGE SCOUR REPAIR

99% CONSTRUCTION PLANS
COUNTY ROAD PROJECT NO: 2159R

SCOUR COUNTERMEASURE PLAN

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TABLE 1
STREAMBED MATERIAL MIX

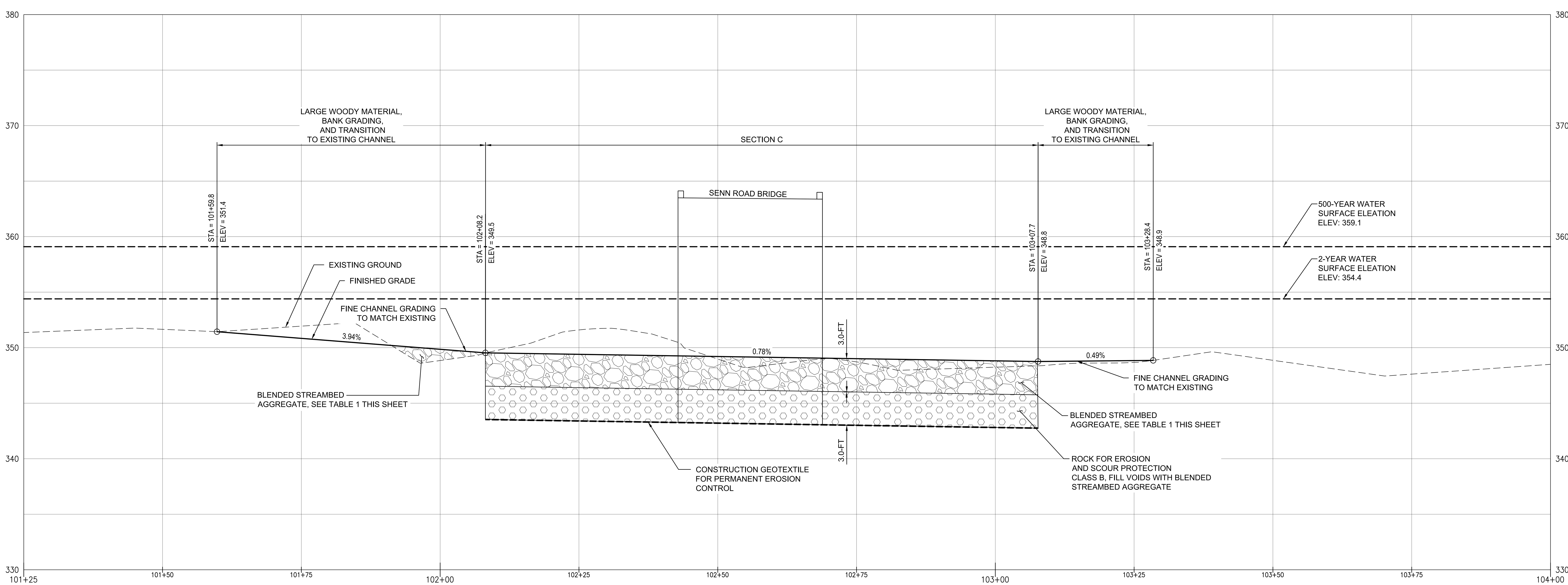
STREAMBED AGGREGATE	PERCENT BY VOLUME	WSDOT STANDARD SPECIFICATION
STREAMBED SEDIMENT	40%	9-03.11(1)
STREAMBED COBBLES 8-IN.	60%	9-03.11(4)

LEGEND

- ROCK FOR EROSION AND SCOUR PROTECTION CLASS B
- BLENDED STREAMBED AGGREGATE 290 TONS
- CONSTRUCTION GEOTEXTILE FOR PERMANENT EROSION CONTROL 861 SY

GENERAL NOTES:

1. ALL MATERIAL AND WORK SHALL BE IN ACCCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION, "STANDARD SPECIFICATIONS FOR ROAD, BRIDGE, AND MUNICIPAL CONSTRUCTION" 2026 EDITION AND SPECIAL PROVISIONS.
2. SEE SHEETS 9 AND 10 FOR TYPICAL SCOUR COUNTERMEASURE AND STREAM RECONSTRUCTION SECTION.
3. SEE SHEET 12 FOR LARGE WOODY MATERIAL DETAILS.



PROFILE VIEW
H: 1"=10', V: 1"=5'

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SCOUR COUNTERMEASURE PROFILE

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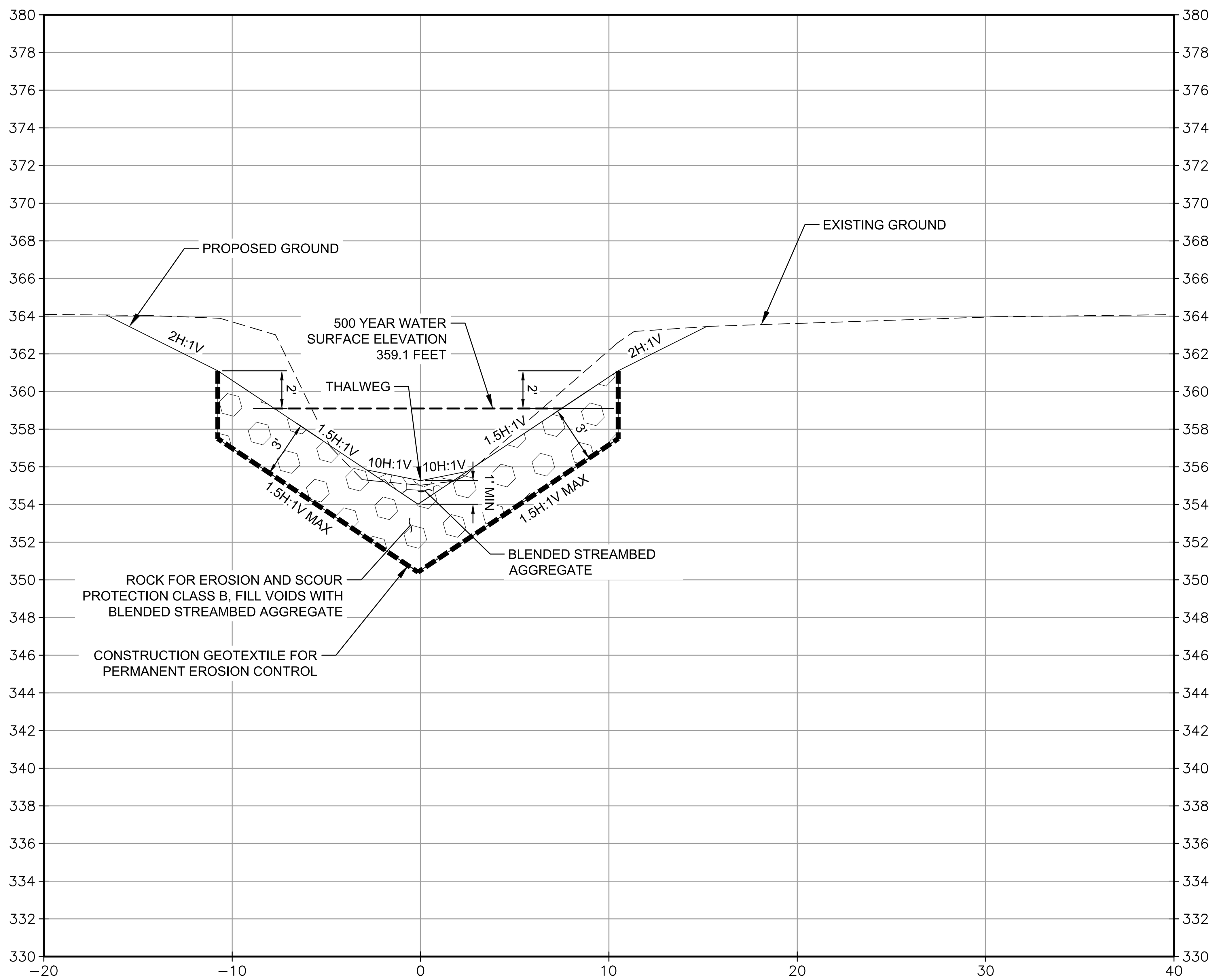
LEGEND

ROCK FOR EROSION AND SCOUR PROTECTION CLASS B

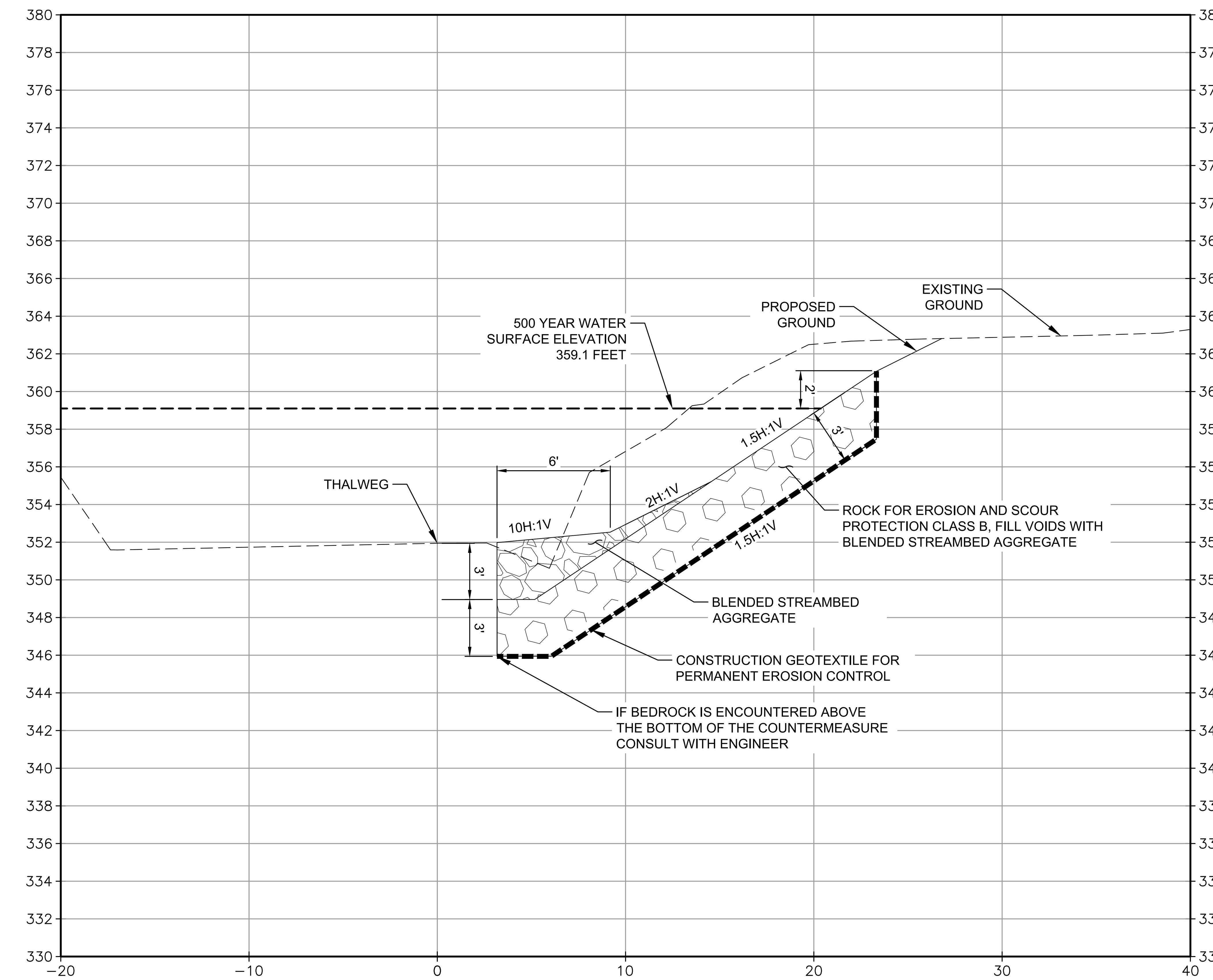
BLENDED STREAMBED AGGREGATE

CONSTRUCTION GEOTEXTILE FOR PERMANENT EROSION CONTROL



SECTION A - SCOUR COUNTERMEASURE TYPICAL SECTION ALONG BRIDGE APPROACH

SCALE: 1" = 5'



SECTION B - SCOUR COUNTERMEASURE TYPICAL TRANSITION SECTION

SCALE: 1" = 5'

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SCOUR REPAIR

99% CONSTRUCTION PLANS
COUNTY ROAD PROJECT NO: 2159R

SCOUR COUNTERMEASURE DETAILS

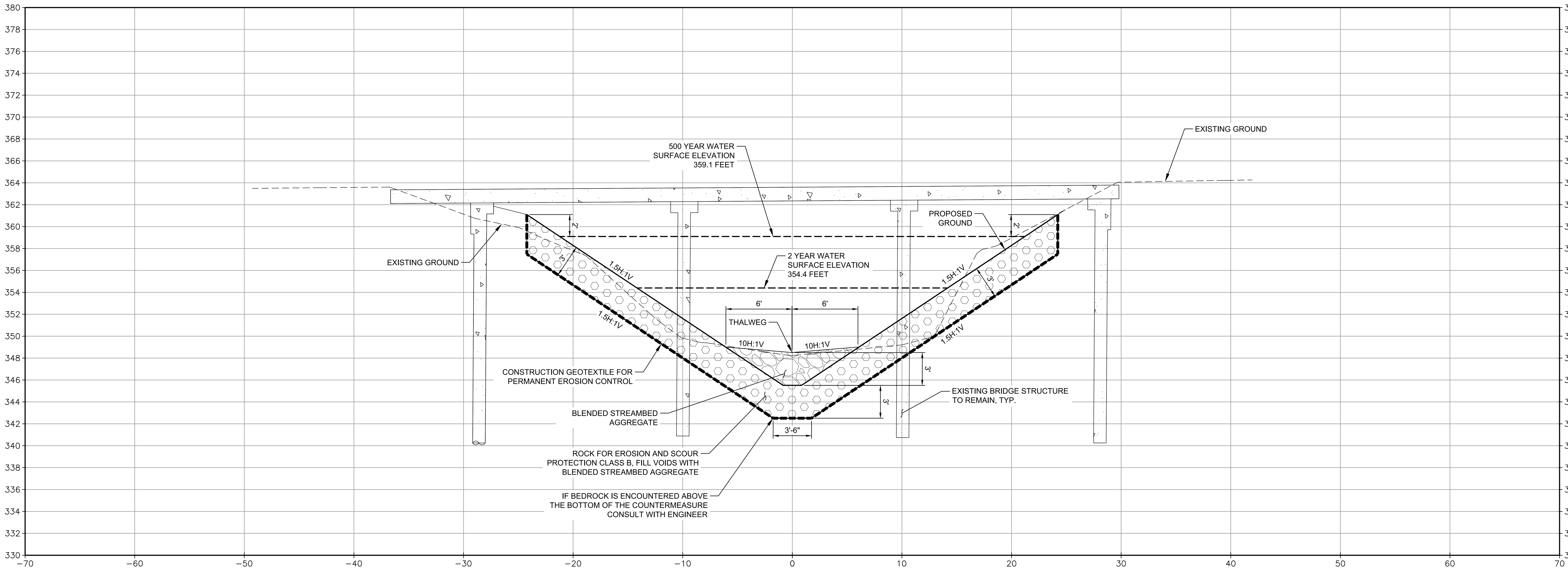
Osborn Consulting

GENERAL NOTES:

1. BLENDED STREAMBED AGGREGATE IS 40% STREAMBED SEDIMENT AND 60% 8-INCH COBBLES. SEE STANDARD SPECIFICATION 8-30 "STREAMS, RIVERS, AND WATERBODIES."

LEGEND

- ROCK FOR EROSION AND SCOUR PROTECTION CLASS B
- BLENDED STREAMBED AGGREGATE
- CONSTRUCTION GEOTEXTILE FOR PERMANENT EROSION CONTROL



SECTION C - TYPICAL SCOUR COUNTERMEASURE AND STREAM RECONSTRUCTION SECTION

SCALE: 1" = 5'

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SCOUR REPAIR

99% CONSTRUCTION PLANS
COUNTY ROAD PROJECT NO: 2159R
SCOUR COUNTERMEASURE DETAILS

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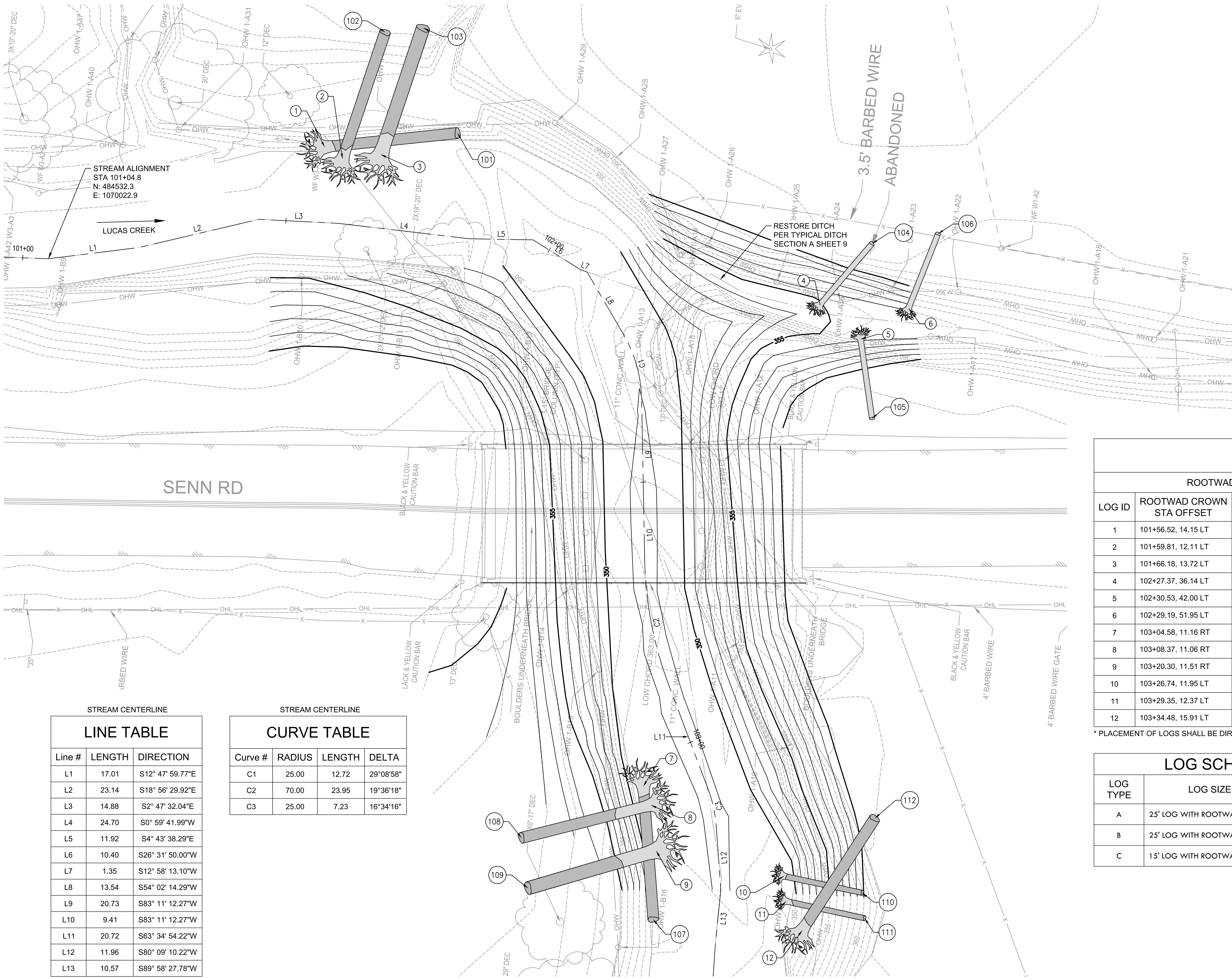
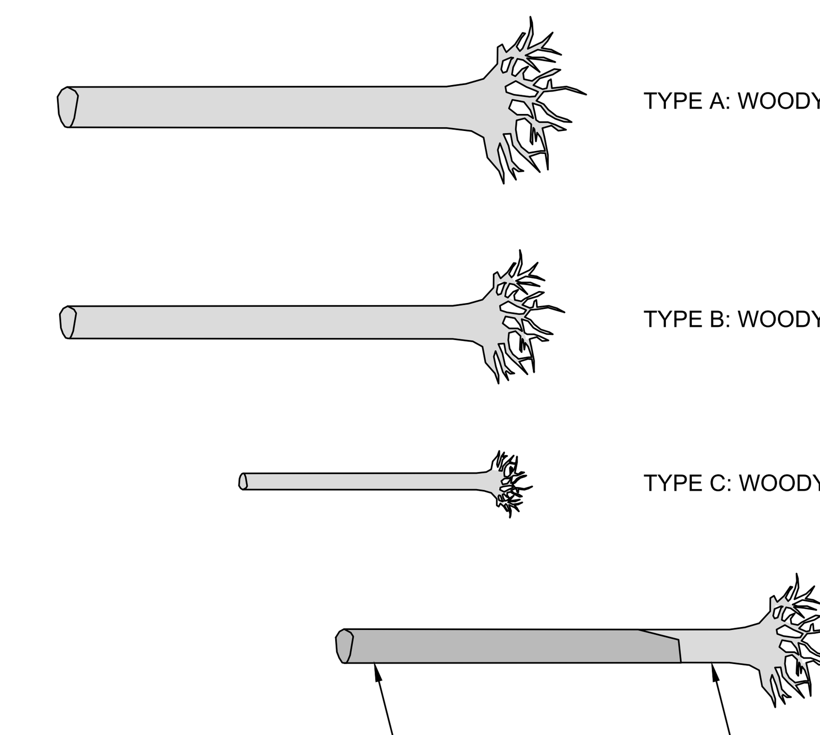
GENERAL NOTE:

1. ALL MATERIAL AND WORK SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION, "STANDARD SPECIFICATIONS FOR ROAD, BRIDGE, AND MUNICIPAL CONSTRUCTION" 2026 EDITION AND SPECIAL PROVISIONS.
2. SEE SHEET 12 FOR LARGE WOODY MATERIAL DETAILS.
3. PLACEMENT OF LOGS 4, 5, AND 6 SHALL BE FIELD DIRECTED BY THE ENGINEER. INSTALL ANCHORS PER DETAIL 3 SHEET 12.



LEGEND



TYPE A: WOODY MATERIAL - 25' LOG WITH ROOTWAD DIA 30"

TYPE B: WOODY MATERIAL - 25' LOG WITH ROOTWAD DIA 24"

TYPE C: WOODY MATERIAL - 15' LOG WITH ROOTWAD DIA 12"

BURIED LOG

EXPOSED LOG

LOG LAYOUT TABLE

ROOTWAD			LOG END			
LOG ID	ROOTWAD CROWN STA OFFSET	ROOTWAD CROWN ELEVATION	POINT #	LOG END STA OFFSET	LOG END ELEVATION	LWM TYPE AND DETAIL
1	101+56.52, 14.15 LT	352.73	101	101+79.98, 19.40 LT	351.60	WOODY MATERIAL - LOG TYPE B; DETAIL 1
2	101+59.81, 12.11 LT	353.20	102	101+64.31, 36.56 LT	354.10	WOODY MATERIAL - LOG TYPE B; DETAIL 1
3	101+66.18, 13.72 LT	353.58	103	101+71.31, 38.19 LT	353.90	WOODY MATERIAL - LOG TYPE A; DETAIL 1
4	102+27.37, 36.14 LT	356.50	104	102+24.68, 49.24 LT	364.90	WOODY MATERIAL - LOG TYPE C*
5	102+30.53, 42.00 LT	357.50	105	102+38.91, 43.03 LT	364.00	WOODY MATERIAL - LOG TYPE C*
6	102+29.19, 51.95 LT	357.70	106	102+25.61, 61.38 LT	364.60	WOODY MATERIAL - LOG TYPE C*
7	103+04.58, 11.16 RT	351.10	107	103+35.21, 12.05 RT	349.70	WOODY MATERIAL - LOG TYPE B; DETAIL 1
8	103+08.37, 11.06 RT	352.00	108	103+05.56, 35.90 RT	353.30	WOODY MATERIAL - LOG TYPE B; DETAIL 1
9	103+20.30, 11.51 RT	351.80	109	103+31.71, 35.92 RT	352.40	WOODY MATERIAL - LOG TYPE A; DETAIL 1
10	103+26.74, 11.95 LT	350.40	110	103+28.44, 26.48 LT	349.40	WOODY MATERIAL - LOG TYPE C; DETAIL 2
11	103+29.35, 12.37 LT	350.50	111	103+31.06, 27.29 LT	350.00	WOODY MATERIAL - LOG TYPE C; DETAIL 2
12	103+34.48, 15.91 LT	351.70	112	103+17.15, 29.71 LT	351.90	WOODY MATERIAL - LOG TYPE B; DETAIL 2

* PLACEMENT OF LOGS SHALL BE DIRECTED IN THE FIELD BY THE ENGINEER.

LOG SCHEDULE

LOG TYPE	LOG SIZE	QUANTITY
A	25' LOG WITH ROOTWAD DBH 30"	2
B	25' LOG WITH ROOTWAD DBH 24"	5
C	15' LOG WITH ROOTWAD DBH 12"	5

10 0 10 20

SCALE IN FEET

STREAM CENTERLINE

LINE TABLE

Line #	LENGTH	DIRECTION
L1	17.01	S12° 47' 59.77"E
L2	23.14	S18° 56' 29.92"E
L3	14.88	S2° 47' 32.04"E
L4	24.70	S0° 59' 41.99"W
L5	11.92	S4° 43' 38.29"E
L6	10.40	S26° 31' 50.00"W
L7	1.35	S12° 58' 13.10"W
L8	13.54	S54° 02' 14.29"W
L9	20.73	S83° 11' 12.27"W
L10	9.41	S83° 11' 12.27"W
L11	20.72	S63° 34' 54.22"W
L12	11.96	S80° 09' 10.22"W
L13	10.57	S89° 58' 27.78"W

STREAM CENTERLINE

CURVE TABLE

Curve #	RADIUS	LENGTH	DELTA
C1	25.00	12.72	29°08'58"
C2	70.00	23.95	19°36'18"
C3	25.00	7.23	16°34'16"

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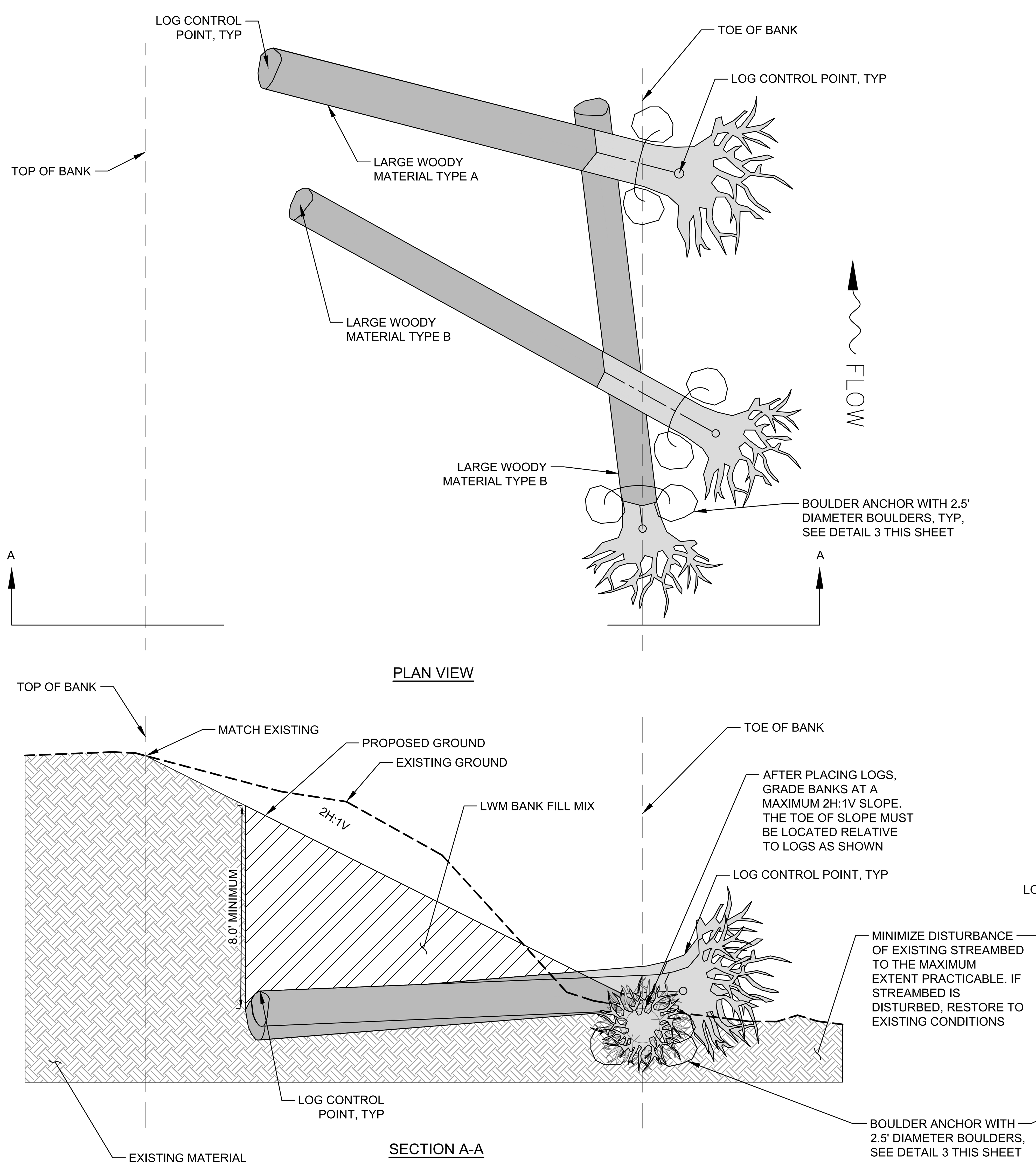
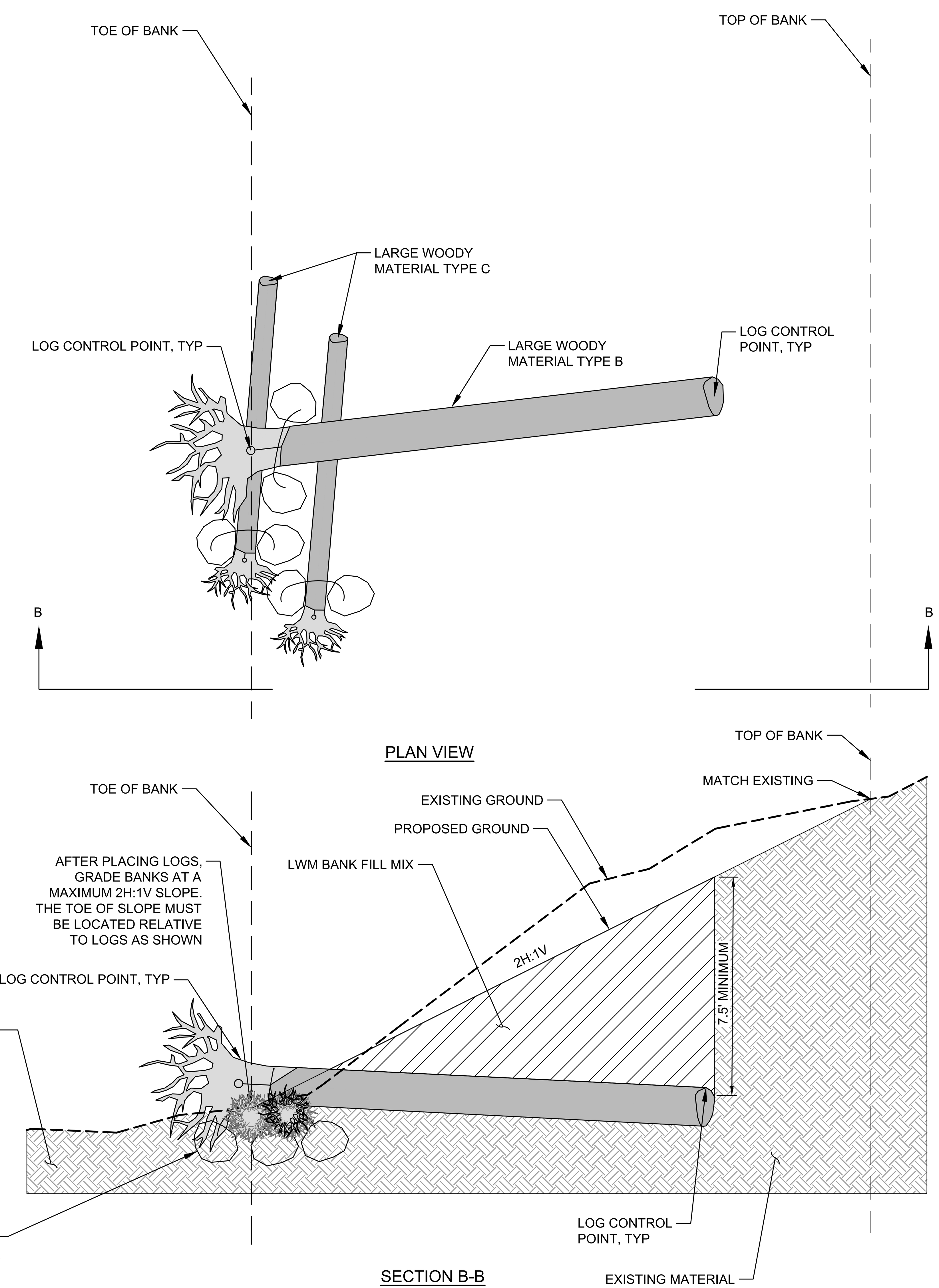
99% CONSTRUCTION PLANS

COUNTY ROAD PROJECT NO: 2159R

STREAM RECONSTRUCTION PLAN





1 / 11 LARGE WOODY MATERIAL (LWM) LOG JAM - DETAIL 1

NOT TO SCALE

2 / 11 LARGE WOODY MATERIAL (LWM) LOG JAM - DETAIL 2

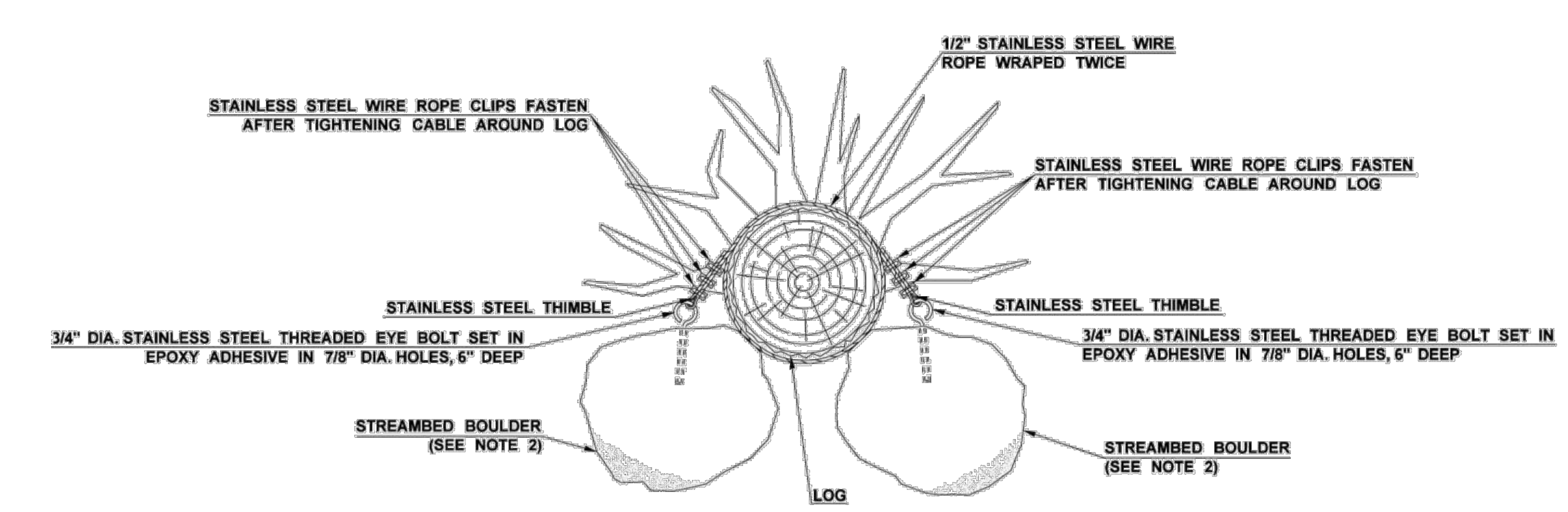
NOT TO SCALE

STREAM RECONSTRUCTION NOTES:

1. DETAILS SHOWN ON THIS SHEET MAY BE ROTATED OR MIRRORED. SEE SHEET 11 FOR LARGE WOODY MATERIAL (LWM) ORIENTATIONS.
2. SEE LWM LAYOUT TABLE AND LWM SCHEDULE ON SHEET 11 FOR LOG CONTROL POINTS, TOP OF LOG ELEVATIONS, AND LOG DIMENSIONS.
3. SEE SPECIAL PROVISIONS 8-33 "LOG STRUCTURES" FOR LWM.
4. LOCATIONS AND ORIENTATIONS OF STREAMBED FEATURES ARE APPROXIMATE AND SHALL BE STAKED PER PLAN BY THE CONTRACTOR AND APPROVED BY THE FIELD STREAM DESIGN ENGINEER. FINAL LOCATION AND ORIENTATION OF THESE STREAMBED FEATURES SHALL BE APPROVED BY THE FIELD STREAM DESIGN ENGINEER.
5. BLENDED STREAMBED AGGREGATE IS 25% 4-INCH COBBLES AND 75% STREAMBED SEDIMENT. SEE STANDARD SPECIFICATIONS 8-30 "STREAM, RIVERS, AND WATERBODIES."
6. ALL LWM DETAIL SECTIONS ON THIS SHEET ARE LOOKING DOWNSTREAM.
7. LWM BANK FILL MIX IS 50% BLENDED STREAMBED AGGREGATE AND 50% ROCK FOR EROSION CONTROL AND SCOUR PROTECTION CLASS B.

LEGEND:

(hatch)	LWM BANK FILL MIX
(hatch)	EXISTING MATERIAL



NOTES:

1. SEE SPECIAL PROVISIONS "WOODY MATERIAL" FOR DESCRIPTION OF MATERIALS AND CONSTRUCTION REQUIREMENTS.
2. SEE STANDARD SPECIFICATIONS 8-30 "WATER CROSSINGS" FOR BOULDER DESCRIPTION.

3 / 11 BOULDER ANCHOR - DETAIL 3

NOT TO SCALE

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SCOUR REPAIR

99% CONSTRUCTION PLANS
COUNTY ROAD PROJECT NO: 2159R

STREAM RECONSTRUCTION DETAILS

SHEET 12 OF 15

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Appendix F: Revetment Rock Sizing Calculations

Particle size for which 30% is finer by weight, ft (m)	d30	1.522	ft	
Local depth of flow, ft (m)	y	9.2	ft	
Safety factor (must be > 1.0)	Sf	1.4		
Stability coefficient (for blanket thickness = d100 or 1.5d50, whichever is greater, and uniformity ratio d85/d15 = 1.7 to 5.2)	Cs	0.3	<-- select which Cs to use	
	angular rock	0.3		
	rounded rock	0.375		
Velocity distribution coefficient	Cv	1.08	<-- select which Cv to use	
for straight channels or the inside of bends		1.00		Countermeasure not on bed. 2D model used
for the outside of bends (1.0 for Rc/W > 26)	Cv = 1.283 - 0.2log(Rc/W)	1.08		
downstream from concrete channels		1.25		
at the end of dikes		1.25		
Blanket thickness coefficient given as a function of the uniformity ratio d85/d15. CT = 1.0 is recommended because it is based on very limited data.	CT	1		
Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s (m/s)	Vdes	12.50	<-- select which Vdes to use	
For natural channels	Vdes = Vavg(1.74 – 0.52log(Rc/W))	13.02	ft/s	
	Vdes = Vavg for Rc/W > 26	7.82	ft/s	Countermeasure not on bed. 2D model used
For trapezoidal channels	Vdes = Vavg (1.71 – 0.78 log (Rc/W))	12.50	ft/s	
	Vdes = Vavg for Rc/W > 8	7.82	ft/s	
	Rc/W	1.39		
Channel cross-sectional average velocity, ft/s (m/s)	Vavg	7.82	ft/s	
Side slope correction factor	K1(see equation on right)	0.72		
bank angle in degrees	θ	33.70	degrees	1.5:1 slope = 33.7 degrees
Centerline radius of curvature of channel bend, ft (m)	Rc	64	ft	
Width of water surface at upstream end of channel bend, ft (m)	W	46	ft	
Specific gravity of riprap	Sg	2.65		
Acceleration due to gravity	g	32.2	ft/s2	
	d50	1.826	ft	

SMS Results - 500yr upstream observation line

Reach	Station	Vel_Mag_ft_p Ave	Water_Depth_ft Max
reach_2	364.7	9.5	9.2
reach_2	**438.14**	**6.14**	**9.6**

FHWA HEC 23 Volume 2 Design Guideline 4.2.2 (PDF page 74)

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{(V_{des})}{\sqrt{K_1(S_g - 1)gy}}\right]^{2.5} \qquad (4.1)$$

where:

- d_{30} = Particle size for which 30% is finer by weight, ft (m)
- y = Local depth of flow, ft (m)
- S_f = Safety factor (must be > 1.0)
- C_S = Stability coefficient (for blanket thickness = d_{100} or $1.5d_{50}$, whichever is greater, and uniformity ratio d_{85}/d_{15} = 1.7 to 5.2)
  - = 0.30 for angular rock
  - = 0.375 for rounded rock
- C_V = Velocity distribution coefficient
  - = 1.0 for straight channels or the inside of bends
  - = 1.283 - 0.2log(R_c/W) for the outside of bends (1.0 for R_c/W > 26)
  - = 1.25 downstream from concrete channels
  - = 1.25 at the end of dikes
- C_T = Blanket thickness coefficient given as a function of the uniformity ratio d_{85}/d_{15}. C_T = 1.0 is recommended because it is based on very limited data.
- V_{des} = Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s (m/s)
  - For natural channels, $V_{des} = V_{avg}(1.74 - 0.52\log(R_c/W))$
    $V_{des} = V_{avg}$ for $R_c/W > 26$
  - For trapezoidal channels, $V_{des} = V_{avg}\,(1.71 - 0.78\log(R_c/W))$
    $V_{des} = V_{avg}$ for $R_c/W > 8$
- V_{avg} = Channel cross-sectional average velocity, ft/s (m/s)
- K_1 = Side slope correction factor

$$K_1 = \sqrt{1 - \left(\frac{\sin(\theta - 14^\circ)}{\sin(32^\circ)}\right)^{1.6}}$$

  where: θ is the bank angle in degrees
- R_c = Centerline radius of curvature of channel bend, ft (m)
- W = Width of water surface at upstream end of channel bend, ft (m)
- S_g = Specific gravity of riprap (usually taken as 2.65)
- g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

$$d_{50} = 1.20\,d_{30}$$

Appendix G: Pier Rock Sizing Calculations

Particle size for which 50% is finer by weight, ft (m)	d50	1.85	ft
Design velocity for local conditions at the pier, ft/s (m/s)	Vdes	16.88	ft/s
Specific gravity of riprap (usually taken as 2.65)	Sg	2.65	
Acceleration due to gravity	g	32.2	ft/s2
Shape factor equal to	K1	1.5	<-- select which K1 to use
1.5 for round-nose piers		1.5	
1.7 for square-faced piers		1.7	
Vmax		11.25	ft/s

FHWA HEC 23 Volume 2 Design Guideline 11.3 (PDF pag

11.3 SIZING ROCK RIPRAP AT BRIDGE PIERS

To determine the required size of stone for ripr recommends using the rearranged Isbash Administration's Hydraulic Engineering Circular 2001) to solve for the median stone diameter:

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g}$$

where:
d_{50} = Particle size for which 50%
V_{des} = Design velocity for local co
S_g = Specific gravity of riprap (us
g = Acceleration due to gravity,

It is important that the velocity used in Equation the immediate vicinity of the bridge pier inclu bridge. If the cross-section or channel average multiplied by factors that are a function of the sh channel:

$$V_{des} = K_1 K_2 V_{avg}$$

If a velocity is distribution available from stream tu model or directly from a 2-D model, then only the pie maximum velocity in the active channel V_{max} is ofter the highest velocity could impact any pier.

$$V_{des} = K_1 V_{max}$$

where:
V_{des} = Local velocity at pier, ft/s (m/s)
K_1 = Shape factor equal to 1.5 for r piers
K_2 = Velocity adjustment factor for for a pier near the bank in a s the main current of flow aroun
V_{avg} = Channel average velocity at th
V_{max} = Maximum velocity in the active