

TONGUE RIVER WATERSHED PLAN

Appendix D-4: Conceptual Design Report



Typical Rock Arch Rapids Photo

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Significant effort has been invested in developing the preliminary design work for the Tongue River Channel Stabilization project, in order to derive a reasonable construction cost estimate as well as to evaluate alternative costs and benefits as required by National Watershed Program Manual and Handbook (USDA, 2014). Given the rapid progression of channel incision and widening in the project reach, even in years of relatively low spring runoff peak flows, a survey and final design will need to be completed immediately after spring runoff and prior to initiation of construction in mid-summer. In addition to an updated survey and related grading, the following activities will be taken to move the current preliminary design to a final design:

- Additional work to measure geomorphic parameters on the reference reach. A few of the current metrics, such as riffle length, seem odd (too short) so additional sections of the MB Forest River will be surveyed to confirm final design parameters.
- The current preliminary design has some compound pools, but the majority of meander bends have a single pool. For meanders that are more than 5 times the bankfull width, the final design will incorporate compound pools.
- Willow is well known to work effectively for live staking and encapsulated soil lift walls. Available riparian stands, including those on all of the sediment deposits on the upper end of the Renwick Reservoir, have a ~70% dogwood component. NRCS intends to do field trials in October 2021 to determine if dogwood will perform similarly to willow in both an encapsulated soil lift placement and as live stakes, at the project site.
- In the fall of 2020, NRCS installed a gauge just upstream of the project at the Country Road 6 bridge with 15-minute stage recording both onsite and transmitted to a website. The intent is to utilize this information to time field visits to collect suspended sediment and bedload data, in order to build FLOWSED/POWERSED models for final design work. In addition, pebble counts upstream of the CR6 bridge, where higher percentages of rounded gravels make up the active channel bed, will be completed. That could influence final cross section design. In addition, flow measurements will assist with design of the dewatering pumps.

Design Objectives

The overall design goal of the PL-566 project is to stabilize active erosion in the incised channel reach and halt the upstream progression of severe channel incision. Design objectives to achieve that that goal focus on development of a project that is cost effective, minimizes the need for future maintenance, limits potential for negative short-term impacts to Northern Pearl Dace and other aquatic species, and can be permitted under U.S. Army Corps of Engineers Nationwide Permit 27: Aquatic Habitat Restoration. Specifically, design objectives are to:

1. Restore ecologically self-sustaining function to the river channel, including levee removal to restore floodplain connectivity at the bankfull flow and cross section, planform, and profile characteristic of stable natural river channels in the area. Resulting channel hydraulic characteristics will allow beaver dams to survive spring runoff events, restoring natural density and longevity of dams.
2. Maintain balanced sediment transport through the reach, with degradation generally balanced with aggradation. Riverbank erosion rates will be comparable to the MB Forest River reference reach 3-years post construction.
3. Restore native vegetation to riverbanks and floodplains, to the extent practical.

Future monitoring by NRCS, post construction, would focus on resurveys of cross section and profile, in addition to measurements of bed chains and bank pins, to document erosion rates in the reach. Natural rates of bank erosion, meander migration, avulsions, and chute cutoffs are expected and are tolerable given land use in the floodplain (nearly all is in existing conservation easements). The only situation that would drive follow up maintenance work is if the river began to flank or undercut the arch rock ramp, or if rocks making up the rock arch rapids shifted in a manner that prevented fish passage.

Reference Conditions

Regional Hydraulic Geometry

As outlined in NEH Part 654 TS-5, regional hydraulic geometry relationships provide a method to validate bankfull parameters when indicators are not clear in the field. Given the rapid rates of incision, widening, and upper slope landslides on the project reach, both assessment and design were highly reliant on regional hydraulic geometry relationships. Development of regional hydraulic geometry relationships utilized for channel assessment and design presented in Appendix D-1, Section 1 and Figures 8-16. Results applied to the project reach are as follows:

$$A_{bkf} = 2.62 (DA)^{0.77} = 2.62 (63)^{0.77} = 64.0 \text{ sqft}$$

$$W_{bkf} = 5.4 (DA)^{0.41} = 5.40 (63)^{0.406} = 29.0 \text{ ft}$$

$$D_{bkf} = 0.47 (DA)^{0.41} = 0.47 (63)^{0.369} = 2.2 \text{ ft}$$

The above values matched both the riffle sections of old meanders cutoff by levees in the project reach, and proportioned values from the reference reach, therefore were determined to be suitable for proposed channel riffle geometry.

Middle Branch Forest River Reference Reach

Procedures detailed in NEH Part 654 Chapter 11 were utilized to develop reference reach parameters for use in the proposed channel design. NRCS evaluated numerous locations on in the Little Pembina River, Willow Creek, Park River, and Forest River systems for use in establishing reference conditions. The only site found to be acceptable, was the Middle Branch Forest River site, as shown in Appendix D-1 Figure 7. Rationale for selection of the MB Forest River reference reach are as follows:

- Similar physiographic province to the project site, which is defined as precipitation/runoff relations due to precipitation/elevation, lithology, and land use (USDA, 2007). Lithology is the most limiting due to project reach located at lower end of unique Pembina Escarpment, which is identified in geologic cross section in Appendix D-1 Figure 19. This area generally follows a 5-mile-wide swath along Hwy 32 from Appendix D-1 Figure 7. Precipitation/runoff and precipitation/elevation have small sensitivity as annual average precipitation, elevations, soils, and watershed land use do not vary considerably within a 100-mile radius.
- Valley type for both the project and reference reaches is U-BR-BC (Unconfined-BedRock-Bedrock Controlled). The bedrock is Pierre Shale formation, which is outcrop along the Pembina Escarpment in this area. The valley type criteria is very similar to lithology, and results in same applicable swath along Hwy 32.
- Same stream type, the Tongue River is a C4 upstream of the incised reach, which has undergone a channel evolution process of $C \rightarrow G \rightarrow F \rightarrow Bc$ (NRCS, 2020) due to lateral confinement generated incision. The Middle Branch Forest River reference reach site also classifies as a C4 stream type.
- Similar to upstream portions of the project reach, the reference reach has a forested riparian area with scattered areas of herbaceous vegetation. Beaver dams have been observed in both reaches, although in the project reach they are present for only a short time period prior to failing due to spring runoff events.
- Similar bed material between the project and reference reaches; a combination of shale derived particles and rounded gravel materials. Given the larger drainage area, the MB Forest gradation is overall larger, which acceptable as dimensionless parameters will be used. The one concern is that the reference reach does have a higher number of rounded boulders in the riverbed than does the project reach. The project reach has a higher density of large instream woody debris, however, so the design relies on those large roughness elements performing in a comparable manner.
- The project and reference reach are both below major watershed dams (Senator Young and Matecjek, respectively). Therefore, this similarity is positive to replicate watershed conditions and sediment transport.

Detailed surveys of the reference reach were completed in 2016 and 2020, which determined cross section, profile, and bed particle dimensions. Planform dimensions were measured from 2019 NAIP aerial image. A summary of cross section dimensions are shown in Figures 1-3, including Worksheets 2-2, 2-3, and 2-4a. Plan, profile, and particle dimensions are summarized in Figures 4-6, including Worksheets 2-4b, and 3-14. Critical parameters of the reference reach, i.e. bankfull depth, width, and area, were validated from regional curves as described in previous section.

The particle distribution is for one of two active bed pebble counts. The two riffles had varied results; the D_{50} ranged from fine gravel (~0.3 inches) to very coarse gravel (1.7 inches). For sediment competence calculations mid-values were adopted, D_{50} value represented coarse gravel (1.3 inches). Bar samples were indeterminate during surveys due to high water surfaces. Therefore, largest bar particle was determined from active channel D_{95} , in order to avoid influence of large boulder erratics. Sediment competence summary is shown in Figure 6; the result is generally stable for both entrainment and competence as calculated depth, slope, or movable particle match existing conditions. Therefore, the reference reach site has appropriate dimensionless parameters for designing a stable channel through the project reach.

Tongue River Project Reach Geomorphic Design Target Values

Procedures detailed in NEH Part 654 Chapter 11 were followed to develop dimensionless ratios for design. The reference reach drainage area is 122 square miles, while the project reach is 63 square miles. The dimensionless parameters calculated from reference reach were then multiplied back by the appropriate parameters for the project reach. The proposed Tongue River design target values are summarized in Figures 7 and 8, representing Worksheets 2-4a and 2-4b. Entrainment and competence calculations are summarized in Figure 9, representing Worksheet 3-14. Computations indicate the proposed channel will be stable if geomorphic target values are achieved in the design.

Preliminary Design

The master design table, including existing reach, proposed design reach, and reference reach data are summarized in Figures 10 – 15.

Pattern

As described in Appendix D-3, multiple factors made Priority 1 restoration, applied largely within the current incised channel top of bank extents, the preferred design approach for this project. The over-widened channel allows some flexibility in designing the pattern within it, however, to minimize disturbance to mature trees in the riparian zone it was desirable to generally center the new channel within the old one in most locations. The design increases the overall bankfull channel length within the project reach (4+25 to 98+50) by 1,201 feet via reactivation of 4 old channel meanders currently cutoff by levees (at 12+00, 26+00, 76+00, and 83+00). As a result, the project will increase of sinuosity from 1.56 to 1.78 over the project reach. A comparison of the preliminary design pattern results to the reference reach derived geomorphic design targets is provided in Table 1. Averages for the preliminary design are provided where AutoCAD Civil 3D is able to generate values from an alignment object.

Table 1: Pattern Preliminary Design vs. Target Geomorphic Values

Variable	Preliminary Design	Geomorphic Target
Minimum Curve Radius	40.7 ft	43.9 ft
Maximum Curve Radius	106.9 ft	120.6 ft
Average Curve Radius	66.7 ft	80.4 ft
Minimum Arc Length	43.7 ft	127.9 ft
Maximum Arc Length	174.8 ft	277.8 ft
Average Arc Length	98.6 ft	171.8 ft
Minimum Belt Width	85.1 ft	90.0 ft
Maximum Belt Width	228.5 ft	200.0 ft

Minimum Linear Wavelength	65.0 ft	146.2 ft
Maximum Linear Wavelength	256.6 ft	343.6 ft

There is currently no protection (riprap or otherwise) on the earth fill slopes below the Highway 89 bridge, or the adjacent approach fill slopes, likely because of the long span provided with the bridge. The north side of the fill slope below the bridge, behind the pier on the downstream side, has eroded to a vertical face as channel incision has progressed (to be repaired during construction). The river channel to either side of the bridge is relatively straight currently, and that was not changed in the design to avoid any increased risk of erosion. Likewise, the old meander on the south side of the channel, just downstream of the bridge, is not being purposely reactivated to avoid the potential for erosion on the road fill, although hydrology will be restore the riparian wetlands in that area with the channel bed elevation raise. As a result of the straighter reach around the bridge, minimum values for belt width, linear wavelength, and arc length are lower than the geomorphic targets for those variables. Although curve radii on the proposed channel fall within minimum and maximum target values, the average is somewhat lower than target, which can be considered in the final design if additional efforts on the reference reach don't indicate a similarly lower average. For the purpose of the preliminary design, a relatively short section of reference reach was surveyed.

Profile

The bankfull slope for the proposed channel is 0.0027 ft/ft, as shown on pages 4-11 of the preliminary design drawings (Appendix 3). That slope matches that of the upstream channel riffle-riffle slope as well as the grade between old river meanders cutoff by levee construction in the project reach. The extended upstream profile (2,000 feet between project upstream extents and County Road 6 bridge) range is measured as 0.0027 to 0.0029 ft/ft, which has small variability due to alignment measured from aerial photos. This upstream slope is shown in Appendix D-1 Figure 30. This extended profile measurement similarly matches proposed channel slope.

The elevation at the upper end of the rock arch rapids was set so that the riffle-riffle slope through the proposed project reach would overlay on the existing upstream channel, as shown on sheet 11 of the drawings. The profile was designed with the riffle sections set 3.0 below the bankfull grade line, running at a slope of 0.0027 ft/ft. Center of pools coincided with the center of the meander bend, with long arc length and shorter radius bends being assigned the deeper pool depths.

Table 2: Profile Preliminary Design vs. Target Geomorphic Values

Variable	Preliminary Design Range	Geomorphic Target Range
Riffle Slope	0.0027 ft/ft	0.001 – 0.006 ft/ft
Max Riffle Depth	3.0 ft	2.2 – 3.6 ft
Riffle Length	9.0- 73.0 ft	8.8 – 73.1 ft
Pool Max Depth	3.8 – 5.5 ft	3.8 – 5.5 ft
Individual Pool Length	12.0 – 172.0 ft	14.5 – 217.5 ft
Pool to Pool Spacing	61.0 - 252.0 ft	72.5 – 246.5 ft
Max Glide Depth	3.1 – 3.8 ft	2.6 – 5.2 ft
Max Run Depth	3.2 - 3.4 ft	3.2 – 4.4 ft

Although the riffle lengths in the preliminary design fit into the geomorphic target range from the reference reach, the lower end of the range does appear oddly short, so that will be investigated further for the final design. In addition, the pool to pool spacing near the bridge got a little to close, which will be remedied. Likewise, the max glide depth (downstream end of glide) and max run depth (upstream end of run) will be increased in some locations. For a preliminary design and generating quantities for a planning level construction cost estimate, however, the profile was determined to be adequate.

The 0.0027 ft/ft constructed channel slope intercepts the existing channel bottom at 105+80, however channel fill was terminated at 98+50. While it can be a risk to rely on the channel naturally filling, even with structures in place to encourage aggradation, this was determined to be a lower risk than disturbance to any of the vegetation in the overbank area between 98+00 and 101+00, in addition to 102+50 and 111+00. This are would be very

prone to chute cutoffs if it did not have long-term beaver dams located through it (as well as upstream). While channel migration is a natural process, it is preferred that the newly riparian zone have time to establish and mature prior to this happening.

Cross Section

Cross sections for riffles, runs, pools, and glides were developed directly from the geomorphic targets described previously and shown on page 12 of Appendix 3. Cross sectional dimensions were very close to what was measured in the abandoned meander channels behind the levees. As the maximum depths vary for runs, glides, and pools, side slopes were kept constant, therefore only the top width and area changed. Point bar slopes for pools remain constant as well, at 0.18 ft/ft.

Table 3: Cross Section Preliminary Design vs. Target Geomorphic Values

Variable	Preliminary Design Range	Geomorphic Target Range
Riffle Xsec Area	64.0 sqft	64.0 sqft
Riffle Width	29.0 ft	29.0 ft
Run Xsec Area	61.6 - 67.2 sqft	57.5 - 85 sqft
Run Width	27.4 - 28.0 ft	28.0 - 34.0 ft
Pool Xsec Area	59.5 - 116.5 sqft	67.4 - 99.3 sqft
Pool Width	27.1 - 37.3 ft	22.0 - 31.3 ft
Glide Xsec Area	58.6 - 78.7 sqft	46.9 - 74.4 sqft
Glide Width	27.7 - 29.8 ft	25.0 - 29.0 ft

Sections were applied as follows, for the purpose of the corridor grading in Civil3D:

- Riffle – both ends had the typical riffle section applied
- Run – upstream end typical riffle section, downstream end typical run section
- Pool – upstream end typical run section, middle typical pool (by depth), downstream end typical glide section
- Glide – upstream end typical glide, downstream end typical riffle

In locations where the bankfull elevation sat slightly below the adjacent floodplain, the preliminary design assumes a 6-foot horizontal bankfull bench with a 2H:1V slope up to current grade. Currently 1H:1V slopes below the Highway 89 bridge are holding well, as are even steeper vegetated side slopes along the river channel, which is the basis of the 2H:1V slope selection. Final design will incorporate slope stability modeling to confirm the grades. Existing steep, unstable slopes on the south side will have as wide a bankfull bench that can be provided (based on meander geometry targets), with a minimum width of 6 feet, placed up against them. The remainder of the slope will be left as is, and over time those are expected to slump to a stable angle of repose onto the floodplain. In a few areas bankfull is slightly above (less than a foot) the elevation of the adjacent floodplain; in those locations a minimum 10 ft wide bench at the bankfull elevation will be constructed before sloping back down to the floodplain.

Channel Stability / Channel Bed Material

The most significant challenge for river restoration on the Tongue River is the weak and highly erodible portion of the channel bed made up of shale derived particles. As described in Appendix D-1, more stable reaches of the Tongue River both up and downstream of the project reach have a higher percentage of rounded gravel particles in the bed material (~20-30%) than does the project reach (~10%), based on ocular estimate. The MB Forest River reference reach similarly has a substantial shale component (~70-80%). Pebble counts were completed utilizing the method outlined in NEH654 TS13A at 42+00 to represent project reach conditions and 102+00 to represent stable channel conditions upstream. Riffle (active bed) and composite (as described in TS3E-6) were completed at both sites, with the results shown. A bar sample was taken, using field wet sieving procedures, as described in NEH654 Table 11-5 at 42+00. In addition, a 5-gallon bucket of material from a gravel bar at 58+00 was taken to a lab and sieved. Tabular results are summarized in Table 4 and gradation plots are provided in Figure 16.

Table 4: Channel Bed Material Summary

Variable	42+00 Riffle	42+00 Composite	42+00 Bar Wet Sieve in Field	58+00 Bar Lab Sieve Analysis	102+00 Riffle	102+00 Composite	102+00 Pool
D16	1.00 mm	2.53 mm	0.00 mm	1.09 mm	2.60 mm	1.86 mm	0.94 mm
D35	5.13	5.28	4.20	2.85	5.84	4.85	1.95
D50	8.66	8.76	6.88	4.64	7.86	7.01	3.87
D84	20.20	21.24	15.59	12.02	14.06	13.81	10.06
D95	29.18	30.21	21.87	17.52	19.30	20.40	16.00
D100	45.00	90.00	30.00	30.00	32.00	45.00	22.60
Silt/Clay	4%	0%	0%	0%	0%	0%	2%
Sand	17%	12%	19%	28%	13%	17%	34%
Gravel	78%	87%	81%	72%	87%	83%	64%
Cobble	0%	1%	0%	0%	0%	0%	0%

The fact that channel bed materials at 42+00, representing the severely incised channel reach, are coarser than at 102+00 where the channel is only minimally incised, is logical given that the higher shear stress in the incised channel reduced the ability of fine grain materials to settle out. The fact that beaver dams exist only seasonally in the incised reach but are present for consecutive years in the upstream reach, also contributes to more fines in the stable channel reach. Pool gradations are finer than the composite, which is finer than the riffle, which is also to be expected. Point bars on pools tend to hold the gravel size classes, while the deep portion of the pool on outer banks is largely sand. As summarized in Appendix D-1, Figure 31, the existing conditions bed stability analysis found strong entrainment degradation potential, however also indicated the existing channel was generally stable for sediment competence. Therefore, historic vertical erosion has most likely occurred during flood events, but not during non-flood event years. These indications have been verified with annual cross section surveys, bed chains, and bank pins that found generally stable bed during the survey timeframe without flood events in the past ~5 years, but still sloughing banks that widen the channel.

The Geologic Investigation Report location in Appendix 1 provides soil mechanics data on the shale riverbed particles after they have been broken down into a soil like material over time due to the weight of overlying riverine deposits, land leveling activities on the floodplain, and historical tillage/planting activities on the floodplain. Construction activities will have the same effect on shale particles which, if left unreinforced, would cause the river channel to incise in even minor flow events after construction. The design, therefore, calls for a 2-foot thick channel lining of custom graded gravel that would under riffles, run, glides, and point bars. The bottoms of pools would not be lined, to allow them to deepen if needed for energy dissipation. The placed material is not intended to be permanent, it will mobilize above bankfull flows, move downstream, and be replaced by similar materials from upstream. Placement of the coarser material is critical to avoid immediate re-incision of placed fill material. Bed stability calculations, documented in Figure 9, shows that competence at bankfull flow is maintained as the predicted largest movable particle is 73.5mm (2.9 inches), while the D100 in Table 7 is 3 inches. Therefore, this design qualifies for an alluvial channel, not threshold channel, with stable balance of sediment competence for neither long term aggradation or degradation. The borrow material to be used for fill, will have to be compacted adequately in-place using rammer or pad-foot vibratory roller equipment. The lean clay material at optimum moisture will need to be impacted with enough force to remove air voids as much as possible.

The existing bed material is shale based, which easily erodes and breaks down, and detailed in Appendix D-1 as major contributing factor to recent incision. Therefore, existing active bed median particle size ($D_{50} \sim 0.3$ inches) is slight; in order to provide entrainment and competence stability, the active bed particle requirements include $D_{50} = 1$ inch, and $D_{100} = 2.9$ inches. The remaining select gravel gradation followed similarly to reference reach, however slightly smaller due to scaling. The reference reach D_{50} is approximately 30% larger than project reach design D_{50} , therefore channel gradation in Figure 4 was reduced by approximately 30% at other increments. Also, the reference reach had boulders which greatly increased D_{100} ; boulders are not included in proposed bed material as they are not natural to the reach, however D_{95} still matched scaled reference reach. The channel bed

material is entirely within gravel spectrum, ranging from very fine to very course. The resulting gradation is summarized in table 5 below.

Table 5: Select gravel gradation for bed material

Sieve Size	% Passing
3"	100%
2"	70-85%
1-1/2"	60-65%
1"	48-63%
3/4"	43-48%
3/8"	33-38%
#4	20-25%
#8	13-18%
#10	4-9%
#30	<3%

Grade Control

Rock Arch Rapids

The primary grade control feature is a rock ramp at the downstream extent of the project (station 2+70 to 4+50), which includes riprap base layer, weir boulders vanes, sheet pile wall, and plunge pool. The rock ramp details are included in Appendix 3, pages 19-20. The rock ramp brings the upstream critically incised channel up to a level where bankfull flow matches the floodplain, with a transition to the downstream reach designed for stability and natural ecological connection. The rock ramp is designed for stability at a 50-year flood flow (872 cfs for Alternative 2), which exceeds the 15-year practice lifespan for a PS-410 Grade Stabilization Structure but matches the 50-year Watershed Plan O&M Plan time period planned. The low flow design criteria is the 25% duration flow (5 cfs), which were calculated from regional regression equations developed by USGS (Williams-Sether and Gross, 2016). The sheet pile wall purpose is to act as a fail-safe countermeasure for channel incision that could be initiated by displacement of rock ramp components. In addition, the sheet pile wall will act as an impervious barrier to eliminate interstitial flow through riprap and maintain surface low flow environment for aquatic organisms. The rock sill out across the valley, sloped down toward the sheet pile wall, will minimize the risk of the sheet pile wall/rock ramp being flanked.

The rock ramp is designed as an arch rapids in order to direct flow to the center of the feature as flows come off arch in perpendicular direction to curvature. The weir angles from bank are ~25°, (Iowa Department of Natural Resources, 2018). The ramp is oriented straight without meanders to further direct flows to the center and reduce outside bend erosive currents. The importance of center directed flows are to concentrate low flows and reduce near bank stresses that could breach the structure and eliminate critical grade control for upstream river stabilization reach. The slope adopted is 5%, which is recommended not to exceed (Iowa Department of Natural Resources, 2018) while reducing project cost. Each weir drop is set at 0.7 feet, which provides a balance between rock stability, fish mobility through the drop, and resting zone between drops. The 0.7 feet drop is similar to other Red River Basin designs, i.e. Sand Hill River in Minnesota. Also, the weir drops are below maximum guidance (0.8 feet) which is set not exceed the physical mobility of fish species (Iowa Department of Natural Resources, 2018). The total ramp height of 9 feet with each drop of 0.7 feet results in 13 individual drops. The total ramp length of 180 feet and 13 drops results in ~14 feet boulder weir spacings, which is important for fish resting places between jumps when moving upstream.

Profile summary:

- Centerline slope: 5%
- Weir angles: ~25°
- Weir drops: 0.7 feet



- Boulder weir spacing: 14 feet

The high flow channel dimensions initiate from bankfull width, in order to align with upstream channel layout. The bankfull width is 29 feet, which is described in the previous section of this appendix. The base width and side slope adopted are 23 feet and 3H:1V, respectively, which are set to minimize unit discharge (Q/b) while maintaining stable slopes. The relatively large riprap ($D_{50} \sim 2$ feet) and boulders, needed for design flow stability cause considerable roughness at bankfull flow condition (271 cfs); with flow depth ~ 1.5 foot the approximate roughness- n value is 0.05 (U.S. Department of Interior, 2007). Using Manning's equation, the normal depth is 1.4 feet, and top width matches bankfull dimension of ~ 30 feet.

The high flow design (872 cfs) causes significant shear stress that is the primary criteria for selecting the significant volume of base stone for the structure. The U.S. Bureau of Reclamation (USBR) Rock Ramp Design Guide (U.S. Department of Interior, 2007) recommends same techniques as listed in NRCS Stone Sizing Criteria (USDA-NRCS, 2007), i.e. Abt and Johnson, ARS rock chute, and USACE Steep Slope Riprap. The Abt and Johnson as well as ARS rock chute techniques determine D_{50} at incipient motion, which both calculated as 0.9 feet. The USACE Steep Slope Riprap is design guide, which calculated D_{30} as 1.5 feet and D_{50} as 2.2 feet, therefore factor of safety to other techniques ~ 2.4 . This factor of safety is considered appropriate for base stone considering nearby rock sources will likely have some rounded features to rock. The North Dakota Department of Transportation (NDDOT) riprap gradation that matches is considered Grade II (NDDOT, 2019), and shown below in Table 6.

The base stone and boulder weirs extend up to and slightly beyond high flow design width. The relatively large riprap ($D_{50} \sim 2$ feet) and boulders, needed for design flow stability cause considerable roughness at this flow condition (872 cfs); with flow depth ~ 2.5 feet, the approximate roughness- n value remains 0.05 (U.S. Department of Interior, 2007). Using Manning's equation, the normal depth is less than 2.6 feet, and top width would be 40.5 feet. Therefore, the width beyond bankfull is 6 feet on each side of the channel. However, the design assumed Cut-off sill (keyway) length is 14.5 feet, based on half of bankfull width (Iowa Department of Natural Resources, 2018). This keyway rises slightly above design water surface elevation and provides stability against higher flows from washing around the weirs.

Minimum boulder size dimension is based on USACE habitat boulder design technique (USDA-NRCS, 2007), which is based on slope (0.05), specific gravity (2.8), and assumed high flow depth (~ 3.5 feet at low flow notch). The resulting minimum boulder dimension is 2 feet. This is similar to minimum boulder thickness (1.5 feet) from Restoration Toolbox (Iowa Department of Natural Resources, 2018). However, the regional rock pit will produce typical thickness ~ 3 feet to match other boulder dimensions of width ~ 3 feet and length ~ 5 feet (Iowa Department of Natural Resources, 2018). Additional thickness allows more embeddedness to resist turning and tumbling due to concentrated flow, ice, and debris; boulder protrusion should not be increased and still follow toolbox guidance of 0.5 feet to 1.5 feet (Iowa Department of Natural Resources, 2018). Boulder dimensions, spacing, and placement details are summarized below.

High flow cross section and rock summary:

- Flow: Design = 872 cfs (50-year flood), Bankfull = 271 cfs
- Mannings N: Design = 0.05, Bankfull = 0.05
- Channel base width: 23 feet
- Side slope above base width: 3H:1V
- Bankfull dimensions: Width ~ 29 feet, depth ~ 1.4 foot
- Keyway length: 14.5 feet on each side of bankfull width
- Base stone: NDDOT Grade II Riprap ($D_{50} = 2.1$ feet)
- Boulder dimensions: Length ~ 5 feet, width ~ 3 feet, and depth ~ 3 feet
- Boulder protrusion: 0.5 – 1.5 feet with less protrusion towards center more at banks of the channel

Table 6: NDDOT Riprap gradations

**Table 256-01
Riprap Gradations**

Grade I		Grade II		Grade III	
Size (Inches)	Percent Smaller	Size (Inches)	Percent Smaller	Size (Inches)	Percent Smaller
20	100	28	100	36	100
18	70-90	22	30-80	29	55-85
15	40-60	16	20-50	24	35-50
10	0-10	10	0-5	10	10-15
6	0-2	6	0-2	6	0-2

Due to the relatively small low flow design (5 cfs), the low flow channel selection is triangular with 4H:1V side slope. The relatively large riprap ($D_{50} \sim 2$ feet) and boulders, needed for high flow stability, cause considerable roughness at this low flow condition; with flow depth < 1 foot, the approximate roughness- n value is 0.08 (U.S. Department of Interior, 2007). Using Manning's equation, the normal depth is 0.8 feet, with 6 feet top width. This low flow channel will be a notch in the middle of the high flow channel. Each low flow boulder weir should include a fish gap measuring 0.3 to 0.6 feet wide (Iowa Department of Natural Resources, 2018). With such a small low flow channel and large boulders, this gap will most likely be in bottom of V-notches. The boulders should be snug on bases and open 0.3 to 0.6 feet at the top of rocks, which more angular boulders could be set aside for this purpose.

The riprap and boulders allow for large voids, which could allow for all or majority of low flow through voids as interstitial flow. Interstitial flow is undesirable at low flow to maintain riverine conditions for fish migration and aquatic invertebrate habitat. Therefore, the sheet pile will be impermeable, plus surrounding choke stone will be used to seal upstream portion of rapids. Choke stone varies in size from 3/8" to 6" (Iowa Department of Natural Resources, 2018), which is mix of ~80% gravel and ~20% cobbles to fill voids of base stone layer.

The sheet pile wall was designed in accordance with the NRCS National Engineering Handbook, Part 654 – Stream Restoration Design, Technical Supplement 14R: Design and Use of Sheet Pile Walls in Stream Restoration and Stabilization Projects (August 2007). This document directly references the Steel Sheet Piling Design Manual, which was therefore the primary design reference. The in-situ soils at the site were identified as Pierre Shale during soil explorations of the stream bed as well as the planned borrow areas, see Appendix 1- Geologic Investigation Report. Although no testing of the in-situ soils at the depth of wall was completed, engineering soil properties were obtained from geologic investigation for the Matejcek Dam rehabilitation effort. Although these sites are located approximately 35 miles apart, a review of geological maps for the state indicates that both sites are characterized as being part of the Cretaceous Pierre Formation. Thus, the material properties obtained for the Matejcek Dam site were considered to be valid as design values for the sheet pile wall. On site geologic investigation and soil mechanics testing will be conducted to support the sheet pile wall analysis during the final design phase.

For construction of the rock arch rapids, the wall height is required to be 9-feet high. The embedment depth of the wall was then determined using the methods presented in the Steel Sheet Piling Design Manual for cohesive soils. As indicated in the analysis, the negative pressure exerted by the cohesive soil at the top of the wall was ignored due to the assumed saturated soil conditions. Thus, the pressure of the backfill was assumed to act against the entire height of the wall. The saturated soil unit weight is an assumed value based upon the CH soil classification and the laboratory obtained dry unit weight. The wall was designed without accounting for the effects of the rock fill associated with the rock arch rapids. Instead, the design of the wall assumed no fill on the face of the wall. This condition is the worst-case future condition, should the rock ramp erode in an extreme event (for example breach of one of the upstream dams or a flood in excess of the 50-yr RI). In addition, this condition will exist during construction given that the sheet pile wall be will installed prior to construction of the rock arch rapids.

The required embedment depth of the wall was determined to be 24'. Therefore, the required panel length is the wall height plus the embedment depth: $9' + 24' = 33'$. Given that sheet panel lengths are typically provided in 5-

foot increments, 35-foot panels will be used. Discussions with suppliers of sheeting panels resulted in the specification of PZ27 ASTM A572 Grade 50 panels as these are common, readily available sections that meet the minimum material requirements by design (sectional modulus, cross sectional area, yield strength). The density of the compacted shale material is expected to prevent installation through traditionally vibratory methods. The SPT count of the material is assumed to be in the range of 30-40 range based upon drilling completed at Matejcek Dam. Thus, installation is recommended by use of a crane with an impact hammer. The previously mentioned PZ27 panels are typically used in dense clay applications given their durability and resistance to the impacts encountered during installation.

Low flow summary:

- Flow: 5 cfs (25% duration)
- Mannings N: 0.08
- V-notch channel: 0.8 feet deep, 6 feet wide, 4H:1V side slope
- Upstream choke stone: D_{50} : 3 inches

The riprap lined plunge pool follows Design Note No. 6 (DN-6) (USDA-SCS, 1986) with design flow (872 cfs) for dimensions and rock sizing. The design note assumed pipe outlet, which differs from the rock ramp outlet. However, the pipe invert above water surface set to typical boulder thickness (3 feet) is conservative but reasonable assumption. The same NDDOT Grade II riprap was initially assumed and verified as stable with the analysis. Velocity calculations assuming pipe flow and area were modified with flow dimensions described previously in this section. Otherwise, the calculations are appropriate to calculate critical plunge pool dimensions, which are summarized in bullets below. Generally, the DN-6 calculated dimensions aligned better with step pool characteristics (U.S. Department of Interior, 2007) identified in Appendix D-4 Figure 17, i.e. short distance to maximum scour depth (L_1) with natural curvature vs. trapezoidal section with long L_1 . The Scour depth (S) is calculated as 4 feet from DN-6, which was validated by recommended equations from USBR (U.S. Department of Interior, 2007). The pool length (L_2) is calculated from DN-6 as 17 feet.

Plunge pool summary:

- Flow: 872 cfs (50-year flood)
- Distance to maximum scour depth (L_1): 4 feet
- Scour depth (S): 4 feet
- Pool Length (L_2): 17 feet

Cobble Patches

Cobble patches will be installed, as a countermeasure to localized excess incision, at 12 top of riffle locations scattered through the project reach as shown in Appendix 3, pages 4-11. As shown on page 22, cobble patch dimensions are 4 foot, 30 feet wide, and 10 feet long. A factor of safety is estimated, and listed in Table 9 below, based on incipient motion from NEH Part 654 Chapters 8 and 11 (NRCS, 2007). The critical parameter for resisting motion is 50% passing diameter (D_{50}), which is estimated to be 4 inches. The cobble key resistance to motion shear stress (1.5 lb/ft²) is compared to hydraulic model shear stress at flood events, or acting force on cobbles, are compared to resistance to motion and factor of safety calculated. The maximum shear stress of 26 hydraulic model cross sections within the project reach and factor of safeties are documented in Table 7. The gradation selected is commonly available as “railroad ballast” material from aggregate suppliers in the area.

Table 7: Cobble key erosion analysis for flood events

	Flood Cobble Key Erosion Analysis					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Shear Stress (lb/ft ²)	0.78	0.99	1.06	1.17	1.34	1.75

Factor of Safety	1.9	1.5	1.4	1.3	1.1	0.9
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Rock Cross Vanes

The erodible nature of channel bed material and depth of fine-grained fill material placed in the channel does generate concern for incision, despite the geomorphic based analyses that indicate stability. Rock cross vanes will therefore be constructed at two locations upstream of the rock ramp as additional countermeasures against incision. One will be located at 555+31, just downstream of the Highway 89 bridge crossing, which is the location of the highest channel shear stress/velocity, and one at 86+25 between that cross vane and the upstream end of the project. The top of vanes will be on grade with the bottom of the bankfull channel, as shown on Appendix 3, page 21. Steel sheet pile cutoff walls will be driven through the placed fill material in the channel bottom, and into native riverbed materials. Cutoff sills to prevent lateral flanking will be installed 20 ft to both sides of the constructed bankfull channel. Rock portion of the cross vanes designed to be stable to the 25-year peak flow and buried steel sheet pile cutoff walls to the 50-year peak flow, see supporting computations in Appendix 2. In addition, two cross vanes without sheet pile cutoff walls will be placed above the end of the channel restoration project, at 98+50 and 101+75. The purpose of these structures is to encourage 1-2 of natural aggradation as shown on Appendix 3, page 11 in the ~700 feet upstream of the end of channel restoration.

Debris Collectors

In addition to the two cross vanes noted above, two debris collector structures will be installed in the ~700 feet of upstream channel where natural aggradation is desired. Debris collectors are a combination of ballasted large woody debris and a beaver dam analogue style structure, as shown in Appendix 3, page 22. They are intended to be temporary in nature, as documented in Appendix 2 supporting computations, and quickly augmented or supplanted by natural beaver dams or large woody debris jams. Natural beaver dams in this area have been in place for several years, and if present during construction will not be disturbed by the project. In this event, one or both debris collectors may be eliminated during construction by NRCS as unnecessary, and the upstream cross vane may be relocated to ensure any natural beaver dam is not subject to additional scour forces.

Bank Protection

Type 1 bank protection will consist of ballasted large woody debris at the toe, underlain by cobble material, and overlain by encapsulated soil lifts with live cuttings and transplanted live clumps as shown in Appendix 3, page 17. The Type 1 protection will be placed on 6,960 feet of channel, along the outer banks of meander bends as shown on pages 3-11, where erosive velocities will be the highest. Structural elements will be stable to the 10-year peak flow, if overlying soil remains in place, which matches the likely lifespan of the large wood. The outer layer of coir fabric will also provide protection against erosion at the 10-year peak flow. That same fabric is incorporated into the Type 2 bank protection treatment shown on page 18. Ultimately bank protection will rely on establishment of mature vegetation on the banks and on the adjacent floodplain, which is being initially encouraged by live staking, clump transplants, and live stems placed horizontally in the encapsulated soil lifts. Beaver browse, particularly during the initial years of establishment is a concern, however substantial natural regeneration of trees and shrubs is also expected in this area. A natural level of erosion and meander migration is acceptable in terms of both design objectives and landowner expectations.

Floodplain Grading

Existing levees will be excavated to the elevation of the surrounding floodplain. Topsoil will be stripped and stockpiled for spreading over the floodplain at the completion of the project, and remaining materials (Pierre shale similar to floodplain and channel bed soils) will be used as fill for the channel construction work. Additional fill material will be derived from the proposed floodplain excavations, as shown in Appendix 3, page 3. Excavations are planned to have 6H:1V side slopes with a flat bottom.

Floodplain excavations have multiple benefits, including flood control, wetland ecological improvements, and local supply of fill needed for the project. One of the PL-566 goals is to increase overall flood retention in the Red River Basin, and to reduce flood damages on downstream cropland in the Tongue River watershed. Raising the river channel bottom up will reconnect the river with the adjacent floodplain. None of the floodplain on the

upstream end of project reach is currently in cropland. The old pastures adjacent to Hwy 89 are partially in permanent USFWS easements, and the remainder of the floodplain is forested or other perennial vegetation. Therefore, these areas can act to provide floodwater retention simply as the result of raising the channel bottom and restoring the natural cross sectional area. However, given the space available in the old fields and the need for fill material in the channel, the potential for varying excavating pools for additional retention is present.

Shallow floodplain soils (0-6 feet deep) are considered alluvial silty clay loam, as found from Web Soil Survey (WSS) and documented in the Geologic Investigations Report (Appendix 1). Alluvium are deposits left by flowing streams, typically fine suspended sediments traveling through the floodplain during floods that fall out due to shallow depths and high roughness causing low velocity. Soil investigations were completed in the fields, which validated shallow silty clay loam, with deeper weathered shale composition. No gravel seams were found, which were hypothesized if the stream channel migrated throughout the floodplain. However, particle mixing, discontinuous gravel at facet changes, and limited point soil investigations don't disprove gravel deposits in the existing floodplain. Therefore, no inferences could be made about the stream bed material prior to incision.

Alternative 1: Large floodplain excavations

Large floodplain excavations objective is to maximize floodplain water storage during flood events while maintaining forest areas and gaining wetland ecological benefits. The excavations are situated partially on USFWS easement areas, and remainder on privately owned land. The total volume of excavations is 190,000 cubic yards (CY), with each excavation summarized in below bullets. However, excavated soils beyond required random channel fill will need to be spoiled off-site, at additional cost. Peak flood flow reduction is significant below the project reach; detailed results are summarized in Appendix D-8 with critical economic frequencies reductions at next bridge (Hwy 32) resulting in 31% (10-year) and 34% (25-year flood). Wetland benefits consider area and functional capacity units and summarized in Appendix D-8; wetland acreage estimate is summarized in below bullets.

- West side of Hwy 89
 - Volume: 52,000 CY or 32 acre-feet
 - Bottom area: 4 acres
 - Depth: average 17 feet
 - Invert elevation: 1108
 - Wetland area: 4.7 acres
- East side of Hwy 89
 - Volume: 138,000 CY or 85.5 acre-feet
 - Bottom area: 8.3 acres
 - Depth: average 10 feet
 - Invert elevation: 1107
 - Wetland area: 8.2 acres

Alternative 2: Small floodplain excavations

Small floodplain excavations objective is to excavate only the volume of material required for channel fill. Outcome remains a lower floodplain water storage during flood events and wetland ecological benefits. The excavations are situated entirely on privately owned land, and adjacent to USFWS easement areas. The total volume of excavations is 67,700 cubic yards (CY), with each excavation summarized in below bullets. Each excavation was sized to ensure construction equipment would not need to haul fill across Highway 89. Peak flood flow reduction is noteworthy below the project reach; detailed results are summarized in Appendix D-8 with critical economic frequencies reductions at next bridge (Hwy 32) resulting in 20% (10-year) and 9% (25-year flood). Wetland benefits consider area and functional capacity units and summarized in Appendix D-8; wetland acreage estimate is summarized in below bullets.

- West side of Hwy 89
 - Volume: 16,300 CY or 10.1 acre-feet
 - Bottom area: 2.3 acres
 - Depth: average 4 feet
 - Invert elevation: 1121.8
 - Wetland area: 2.6 acres
- East side of Hwy 89

- Volume: 51,400 CY or 31.9 acre-feet
- Bottom area: 3.1 acres
- Depth: average 7 feet
- Invert elevation: 1111.0
- Wetland area: 3.5 acres

Revegetation

Current vegetation at the project site was assessed on May 18-19 2021 and is described in detail within Appendix D-7. The project area consists of four habitat types: tame grassland, upland forest, river/stream, and native riparian forest. Two-thirds of the tame grass is in poor condition consisting of predominantly bromegrass with a heavy infestation of common tansy. The remaining tame grass, currently enrolled in the Conservation Reserve Program (CRP), is in fair condition and is hayed in a rotating schedule as the contract allows. The upland forest is in good condition and is dominated by bur oak, green ash, and young American elm species. The riparian forest habitat ranges from fair to good condition; basswood dominates the upper story, however bur oak, boxelder maple, balsam poplar and many other diverse species are present. In the mid-story/shrub layer, chokecherry, young American elm, balsam poplar and willow species are common. The American elm naturally regenerates in the forest but rarely reaches maturity due to Dutch elm disease. Many areas of the understory have abundant desirable species of forbs such as wild ginger, columbine, violets, and American vetch; however invasive species and noxious weeds are also present such as smooth bromegrass and Canadian thistle.

Suitable plant materials were determined by soil type. Soils in the project area are within the Major Land Resource Area (MLRA) 56 - Red River Valley of the North. These soils formed from the glaciolacustrine materials of glacial lake Agassiz. The project is at the far west boundary of MLRA 56 where the Tongue River cut through the sand deltas and beach ridges of the glacial lake. Over 80% of the soils within the AA are mapped as Fairdale silty clay loam, 2-6% slope and 0-2% soil map units. The Fairdale soil series consists of deep, moderately well drained soils that formed in calcareous, stratified alluvium that is occasionally flooded. There is also a small percentage of Rauville silty clay loam 0-1 % slope soil. Rauville soil floods more frequently but does not have dissimilar existing vegetation to the Fairdale units. The primary tree and grass soil planning unit for the project is therefore Fairdale silty clay loam (I585B)

This Fairdale soil has very few limitations for tree or vegetative plantings. Herbaceous plant materials appropriate for the Fairdale silty clay loam soil are within the Loamy Overflow Ecological Site Condition Group (NRCS FOTG). Woody plant materials will be selected from species suited to Conservation Tree and Shrub Group 1K (NRCS FOTG). This tree and shrub group is only slightly limited by moderately high levels of calcium carbonate.

The desired future condition for the riparian and floodplain environment within the project area is to restore, maximize and maintain the native riparian vegetation. Revegetation and renovation will occur within four planting zones as shown on the conceptual design drawings in Appendix 3, pages 24 and 25, and is also incorporated into bank protection treatment drawings on pages 17 and 18.

Zone 1 consists of the immediate riverbank along both sides of the 9,425 feet river restoration reach. Out of the total 18,850 feet of riverbank, 37% of the highest shear stress areas will have Type 1 protection (toe wood with encapsulated soil lifts above), 49% will have the Type 2 protection (coir fabric with live stake plantings), and the remaining 14% are point bar areas that are naturally unvegetated. Stability design and a detailed description of the Type 1 and 2 bank treatments is described above, under bank protection, and outlined in the project drawings. Vegetative materials to be utilized on the 6,960 feet of Type 1 protection include grass seed, below the coir fabric on the exposed face of the encapsulated soils lifts, live willow and dogwood cuttings, and transplanted mature live willow and dogwood clumps. Vegetative materials to be utilized on the 9,310 feet of Type 2 protection include grass seed below the coir fabric and live willow and dogwood stakes. There are ample areas where live stakes may be harvested in the Tongue River watershed, particularly on sediment deposits on the upstream ends of Olson, Senator Young, and Renwick Dams. The horizontal layers of cuttings to be placed within the encapsulated lifts above the toe wood placement, in the Type 1 protection, may be acquired in one of two ways by the construction contractor:

- a) Harvesting the winter prior to construction, and then storing in cold storage prior to the project, below 41 degrees F and above 90% humidity. This would allow the convenience of harvesting in winter, when reservoirs are iced over and trucks/trailers have good access to stands.
- b) Harvesting live stakes/cutting immediately prior to construction. Access would be more difficult, particularly if it is wet summer, and leaves may need to be hand stripped prior to placement. In Iowa, success with encapsulated soil lift construction utilizing live branches without stripped leaves has occurred. ND NRCS staff will be completing some trials in late summer/fall of 2021 to see if this is likely to work in the Tongue River watershed.

The live stakes for the Type 2 protection will be harvested in the winter after construction and placed the following spring, as soon as the ground thaws to the point that can be done. Likewise, the willow clump transplants for the Type 2 protection will be completed early spring following construction. The bid package will provide a list of locations suitable for local harvesting of materials, that can be utilized by the contractor, and these alternatives will be explained at the site showing. Grass seed used for the bank protection treatments will be a cool season mix.

Zone 2 consists of 16.6 acres to be restored to native riparian woody vegetation. In mid-July, existing tame grass vegetation will be mowed with a rotary mower. In early August, just before construction commences, a broad-spectrum systemic herbicide will be applied to the tame grass. Post construction conditions in November will vary between heavily compacted construction routes, lightly disturbed existing woody vegetation, and lightly disturbed tame grass. All areas except the undisturbed woodland, will be disked, harrow-packed, and seeded to a temporary cover of oats in the fall after construction. Construction routes and drop points are expected to be heavily compacted and will be thoroughly tilled to a depth of at least 6 inches by disking and harrow-packing. In the spring following construction, this zone will be seeded to a non-competitive native grass mixture of slender wheatgrass, little bluestem, sideoats grama and blue grama to reduce herbaceous competition. After this seeding operation, trees, shrubs, and prairie cordgrass plugs will be hand planted in 8 feet by 8 feet grid using planting bars, spades, and augers. Tree and shrub species will include basswood, bur oak, boxelder, hawthorn, chokecherry, woods rose, snowberry and false indigo. Lower areas of this zone and wetlands will be planted to species more tolerant of wet conditions such as prairie cordgrass plugs, willows and aspen. Tree tubes with stakes will be applied to non-suckering hardwoods by hand labor, to prevent deer browse. All planting operations will be completed by May 30.

Zone 3 consists of 27.5 acres to be restored to native herbaceous vegetation, which includes the floodplain excavations being completed to generate fill for the project. In mid-July, existing tame grass vegetation will be mowed with a rotary mower. In early August, just before construction commences, a broad-spectrum systemic herbicide will be applied to the tame grass. Post construction conditions in November will vary between heavily compacted construction routes and herbicide-desiccated tame grass. All areas will be disked, harrow-packed, and seeded to a temporary cover of oats after construction. Construction routes and drop points are expected to be heavily compacted and will be thoroughly tilled to a depth of at least 6 inches by disking and harrow-packing. The following spring, the floodplain excavation sites will be seeded to a 3-4 species native grass/forb mixture tolerant of wet conditions such as prairie cordgrass seed, switchgrass, western wheatgrass and Canada milkvetch. The remaining acres will be seeded to a diverse mixture of 6-10 species of native grasses and forbs designed to benefit wildlife and pollinators. This mixture will include green needle grass, western wheatgrass, big bluestem, little bluestem, switchgrass, American vetch, and purple prairie clover. One mechanical weed control treatment will be performed 1-3 months after seeding and clipping will occur in the following two years. Zone 3 grass seeding will be completed prior to June 15th.

Zone 4 consists of 11.1 acres to be restored to the existing CRP grass mixture. Post construction conditions will vary between heavily compacted construction routes and lightly disturbed grass currently in the CRP program. Areas of existing grass still in good condition will not receive any treatment. All areas negatively impacted by construction will be disked, harrowed, and seeded to a temporary cover of oats or barley in the fall after construction. The following spring, the area will be seeded to a mix comparable to the existing CRP mixture of native and introduced wheatgrasses and legumes. One mechanical weed control will be performed 1-3 months after seeding.

During the final design phase of the project NRCS will develop detailed construction specifications for the project and provide the contractor with an exact listing for quantities and types of grass seed, trees, shrubs, and tree shelters/stakes to be supplied. Trees and shrubs will be conservation grade and the contractor will be required to supply the NRCS inspector with seed tags for grass and forbs. NRCS will be onsite to provide quality control during all aspects of the revegetation work.

Operation and Maintenance

Given the objectives of the project, maintenance of the reconstructed channel profile is the primary long-term concern. Sediment delivery as a result of meander migration and channel bank erosion is expected in the reach, at naturally occurring levels (similar to those on the MB Forest River reference reach). Therefore, the only O&M potential would be at the rock arch rapids structure. MN DNR has installed a number of this style of grade control structure on the east side of the Red River Basin with minimal maintenance needs reported, however if large boulders were to dislodge in a significant flood event the rapids could become impassable for aquatic species. As a result, the Pembina WRD will acquire an O&M easement to access the rock arch rapids and be responsible for maintenance on the structure should it ever become necessary. A conservative assumption of 8 hours of onsite excavator work, 42 tons of riprap, and 12 tons of large boulders at flood events exceeding the 20-year recurrence interval was made for watershed plan economics (\$ 5,671 in current year dollars).

Hydraulic Analysis

Existing conditions hydrologic model for the Tongue River base was first developed in HEC-HMS computer program in 2011. The study results were presented in Red River of the North Hydraulic Modeling – Phase 2 (USACE, 2013). The USACE hydrology (HEC-HMS) model is modified for refinement of features and parameters, i.e. split subbasins, geospatial curve number analysis, unit hydrograph, and reach routing enhancements. Then, calibration was completed for June 2002 and May 2013 flood events. Finally, synthetic rainfall events were developed for 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals at critical watershed 4-day duration.

The hydraulic model (HEC-RAS) began in 2016 with survey and development of mixed 1-Dimensional (1-D) and 2-Dimensional (2-D) features. The model extends along the Tongue River from 127th Ave. NE to tailwater of Renwick Dam. HEC-HMS subbasin results are input to HEC-RAS model; tributary inflows are applied to 1-D storage areas, while Tongue River is modeled with 1-D cross sections and floodplain 2-D areas. The model was calibrated to 2002 event and validated with 2013 event. Finally, synthetic events are applied to hydraulic model. Additional existing condition details are summarized in Existing Conditions Hydraulics and Hydrology Report in Appendix D-2.

The two alternative project details were applied to the existing conditions hydraulic (HEC-RAS) model, including raised channel thalweg and floodplain excavations. Floodplain excavation details are summarized in previous report section. Flood damage reductions below the project are summarized in Appendix D-8, which showed variable peak flow reductions, inundated areas, and duration reductions in total and for cropland in the entire watershed. However, there are hydraulic model results within the project reach that are important that are summarized in tables below. The three locations include upper project reach (above Hwy 89), lower project reach (below Hwy 89), and average project reach. Flow results are documented at Hwy 89 (upper reach) and rock rapids (lower reach). Velocity represents average cross section velocity in feet/sec (fps); which is taken as average of nineteen (upper reach) and seven (lower reach) cross sections. Shear represents average cross section shear stress in lb/sq ft (psf); which is taken as average of nineteen (upper reach) and seven (lower reach) cross sections. Table 8 is summary of Alternative 1 and table 9 summary of Alternative 2 hydraulic model results. In summary, with floodplain re-connectivity and additional storage there is a significant reduction for peak flow, velocity, and shear stress for all events.

- Alternative 1 – range of average project reach hydraulic changes
 - Peak flow reduction: 0-35%
 - Cross section velocity reduction: 1-13%
 - Cross section shear stress reduction: 47-59%
- Alternative 2– range of average project reach hydraulic changes
 - Peak flow reduction: 0-24%
 - Cross section velocity reduction: 1-13%

- Cross section shear stress reduction: 46-59%

Table 8: Alternative 1 project reach hydraulic property results (flow, channel velocity, and cross section shear stress)

Location	Event	Existing Conditions	Alternative 1:		Existing Conditions	Alternative 1:		Existing Conditions	Alternative 1:	
		Flow (cfs)	Flow (cfs)	% Change	Velocity (fps)	Velocity (fps)	% Change	Shear (psf)	Shear (psf)	% Change
Tongue River - Upper Project (Above Hwy 89)	2-year	302	301	0%	3.8	4.0	4%	0.39	0.24	-37%
	5-year	489	451	-8%	4.5	4.6	3%	0.49	0.24	-51%
	10-year	690	542	-21%	5.0	5.1	1%	0.56	0.29	-48%
	25-year	1,009	797	-21%	5.7	5.6	-3%	0.66	0.33	-49%
	50-year	1,294	1,180	-9%	6.2	5.8	-7%	0.66	0.34	-49%
	100-year	1,609	1,546	-4%	6.7	5.9	-11%	0.64	0.36	-43%
Tongue River - Lower Project (Below Hwy 89)	2-year	303	303	0%	4.4	4.1	-7%	0.48	0.19	-60%
	5-year	493	400	-19%	5.1	4.7	-8%	0.59	0.22	-63%
	10-year	696	433	-38%	5.7	4.9	-14%	0.71	0.24	-66%
	25-year	1,020	523	-49%	6.6	5.5	-17%	0.87	0.30	-66%
	50-year	1,310	776	-41%	7.2	6.1	-15%	1.00	0.38	-62%
	100-year	1,629	1,030	-37%	7.8	6.6	-15%	0.90	0.45	-50%
Tongue River - Project Average	2-year	302	302	0%	4.1	4.0	-1%	0.43	0.22	-50%
	5-year	491	425	-13%	4.8	4.7	-3%	0.54	0.23	-58%
	10-year	693	487	-30%	5.4	5.0	-7%	0.63	0.27	-58%
	25-year	1,014	660	-35%	6.1	5.5	-10%	0.76	0.31	-59%
	50-year	1,302	978	-25%	6.7	5.9	-11%	0.83	0.36	-57%
	100-year	1,619	1,288	-20%	7.2	6.3	-13%	0.77	0.41	-47%

Table 9: Alternative 2 project reach hydraulic property results (flow, channel velocity, and cross section shear stress)

Location	Event	Existing Conditions	Alternative 2:		Existing Conditions	Alternative 2:		Existing Conditions	Alternative 2:	
		Flow (cfs)	Flow (cfs)	% Change	Velocity (fps)	Velocity (fps)	% Change	Shear (psf)	Shear (psf)	% Change
Tongue River - Upper Project (Above Hwy 89)	2-year	302	301	0%	3.8	4.0	4%	0.39	0.24	-37%
	5-year	489	460	-6%	4.5	4.6	3%	0.49	0.24	-51%
	10-year	690	598	-13%	5.0	5.1	1%	0.56	0.29	-48%
	25-year	1,009	981	-3%	5.7	5.5	-4%	0.66	0.33	-49%
	50-year	1,294	1,270	-2%	6.2	5.8	-7%	0.66	0.34	-49%
	100-year	1,609	1,584	-2%	6.7	5.9	-11%	0.64	0.36	-43%
Tongue River - Lower Project (Below Hwy 89)	2-year	303	303	0%	4.4	4.1	-7%	0.48	0.19	-60%
	5-year	493	402	-18%	5.1	4.7	-8%	0.59	0.22	-62%
	10-year	696	456	-35%	5.7	5.1	-12%	0.71	0.25	-65%
	25-year	1,020	674	-34%	6.6	5.8	-12%	0.87	0.30	-66%
	50-year	1,310	869	-34%	7.2	6.2	-13%	1.00	0.40	-60%
	100-year	1,629	1,094	-33%	7.8	6.7	-15%	0.90	0.46	-49%
Tongue River - Project Average	2-year	302	302	0%	4.1	4.0	-1%	0.43	0.22	-50%
	5-year	491	431	-12%	4.8	4.7	-2%	0.54	0.23	-57%
	10-year	693	527	-24%	5.4	5.1	-6%	0.63	0.27	-57%
	25-year	1,014	828	-18%	6.1	5.6	-8%	0.76	0.31	-59%
	50-year	1,302	1,070	-18%	6.7	6.0	-11%	0.83	0.37	-55%
	100-year	1,619	1,339	-17%	7.2	6.3	-13%	0.77	0.41	-46%

Since this is a conceptual plan a hydraulic model including sediment transport (Flowsed/Powered) has not been completed, this is planned as part of the final design. The six USGS gauges described in Appendix D-1 to analyze regional hydraulic geometry were analyzed for sediment transport measurements. Two of the gauges (Pembina River and Little South Pembina River) included Suspended Sediment Concentration (SSC) measurements in mg/l. Therefore, a best fit line was plotted correlating flow to SSC; which bankfull value will be used to develop regional SSC relating drainage area and SSC at bankfull levels. The Pembina River gauge (DA = 3,350 mi²) bankfull SSC is 1,294 mg/l with sand/fine split of 10/90%. The Little South Pembina River gauge (DA ~177 mi²) bankfull SSC is 2546 mg/l with sand/fine split of 19/81%. Other gauges in the region will be considered along with NRCS SSC field

measurements at temporary gauge installed in 2020 at 127th Ave NE Bridge for final design sediment transport hydraulic modeling.

Conceptual Construction Plan

Design of a dewatering plan on this project is challenging, due to the high fill of Highway 89 across the valley. Consideration was given to boring a culvert below the highway fill, to allow construction of a temporary bypass channel, however the culvert cost alone would approach \$500,000. The highly erodible floodplain materials generate concern that any type of flood flow would cause a massive gully to form, effectively causing the channel to braid upstream of the highway. The proposed dewatering plan therefore relies on 8 temporary in-channel dams and bypass pumps, with portable pipeline to be placed on the south side of the river, as shown on Appendix 3, pages 2, 23, and 24. Upstream of the dam, a fish screen will be placed across the channel, and aquatic species will be collected and relocated out of each dewatering section prior to removing flows. Preliminary design computations for the dewatering system are provided in Appendix 2, which is sized for the 25% probability flow between August and November, when the project would take place. Adequate fuel for a pump run time of at least 5 days would be required to be maintained onsite, stored in double walled fuel tanks located outside of the bankfull channel. Pumps would have float activated start/stop controls given that flows during the planned construction window are typically very low; so that if a sudden rainstorm occurs in off hours the secondary pump will start up. The project would start with construction of the rock arch rapids, and then proceed upstream as described on page 24. Construction access routes and stockpile/staging areas for each segment of construction are shown on pages 23-24.

Pollution control during is of concern, particularly since northern pearl dace are reportedly sensitive to fine sediment. The initial flow of water into the newly constructed channel segment will generate sediment, however the fact that most of the channel will be have imported gravel bed material will minimize that concern. In addition, the bank protection elements will also help to minimize sediment delivery. Spring runoff in the year following construction, prior to establishment of vegetation, is also of concern but is addressed through planting of temporary cover as described.

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Bankfull VELOCITY & DISCHARGE Estimates						
Stream:	MBForest_Reference		Location:	Reach - DS_Matecjek_2020_Elev		
Date:	6/22/2020	Stream Type:	C4	Valley Type:	U-BR-BC	
Observers:	JP, CF, RS		HUC:	9020308		
INPUT VARIABLES			OUTPUT VARIABLES			
Bankfull Riffle Cross-Sectional AREA	99.60	A_{bkf} (ft ²)	Bankfull Riffle Mean DEPTH	2.42	d_{bkf} (ft)	
Bankfull Riffle WIDTH	41.16	W_{bkf} (ft)	Wetted PERMIMETER $\sim (2 * d_{bkf}) + W_{bkf}$	43.77	W_p (ft)	
D_{84} at Riffle	181.49	Dia. (mm)	D_{84} (mm) / 304.8	0.60	D_{84} (ft)	
Bankfull SLOPE	0.0037	S_{bkf} (ft / ft)	Hydraulic RADIUS A_{bkf} / W_p	2.28	R (ft)	
Gravitational Acceleration	32.2	g (ft / sec ²)	Relative Roughness $R(ft) / D_{84} (ft)$	3.83	R / D_{84}	
Drainage Area	122.0	DA (mi ²)	Shear Velocity $u^* = (gRS)^{1/2}$	0.521	u^* (ft/sec)	
ESTIMATION METHODS			Bankfull VELOCITY		Bankfull DISCHARGE	
1. Friction Factor / Relative Roughness $u = [2.83 + 5.66 * \text{Log} \{ R / D_{84} \}] u^*$			3.19	ft / sec	317.66	cfs
2. Roughness Coefficient: a) Manning's n from Friction Factor / Relative Roughness (Figs. 2-18, 2-19) $u = 1.49 * R^{2/3} * S^{1/2} / n$ $n = 0.038$			4.10	ft / sec	408.36	cfs
2. Roughness Coefficient: b) Manning's n from Stream Type (Fig. 2-20) $u = 1.49 * R^{2/3} * S^{1/2} / n$ $n = 0.031$			5.05	ft / sec	502.58	cfs
2. Roughness Coefficient: c) Manning's n from Jarrett (USGS): Note: This equation is applicable to steep, step/pool, high boundary roughness, cobble- and boulder-dominated stream systems; i.e., for Stream Types A1, A2, A3, B1, B2, B3, C2 & E3 $u = 1.49 * R^{2/3} * S^{1/2} / n$ $n = 0.39 * S^{0.38} * R^{-0.16}$ $n = 0.041$			3.84	ft / sec	382.76	cfs
3. Other Methods (Hey, Darcy-Weisbach, Chezy C, etc.) Manning's Limerinos $n=0.0451$			3.47	ft / sec	345.51	cfs
3. Other Methods (Hey, Darcy-Weisbach, Chezy C, etc.) Darcy-Weisbach (Leopold, Wolman and Miller)			3.40	ft / sec	338.30	cfs
4. Continuity Equations: a) Regional Curves Return Period for Bankfull Discharge $Q = 1.6$ year $u = Q / A$			2.20	ft / sec	222.00	cfs
4. Continuity Equations: b) USGS Gage Data $u = Q / A$				ft / sec	224.00	cfs
Protrusion Height Options for the D_{84} Term in the Relative Roughness Relation (R/D_{84}) – Estimation Method 1						
Option 1. For sand-bed channels: Measure 100 "protrusion heights" of sand dunes from the downstream side of feature to the top of feature. Substitute the D_{84} sand dune protrusion height in ft for the D_{84} term in method 1.						
Option 2. For boulder-dominated channels: Measure 100 "protrusion heights" of boulders on the sides from the bed elevation to the top of the rock on that side. Substitute the D_{84} boulder protrusion height in ft for the D_{84} term in method 1.						
Option 3. For bedrock-dominated channels: Measure 100 "protrusion heights" of rock separations, steps, joints or uplifted surfaces above channel bed elevation. Substitute the D_{84} bedrock protrusion height in ft for the D_{84} term in method 1.						
Option 4. For log-influenced channels: Measure "protrusion heights" proportionate to channel width of log diameters or the height of the log on upstream side if embedded. Substitute the D_{84} protrusion height in ft for the D_{84} term in method 1.						
Appendix D-4 Figure 1 Reference Reach						
Bankfull Flow Estimates						



Stream:	MB Forest River - Reference Reach		
Basin:	Drainage Area:	acres	122 mi ²
Location:	48.223 Lat / 97.894 Long		
Twp.&Rge:	155 / 56	Sec.&Qtr.:	NW 30
Cross-Section Monuments (Lat./Long.):	Riffle 610	Date:	6/1/2020
Observers:	Fisher, Petersen, and all		
	Landscape Type: VI, U-BR-I		

Bankfull Width (W_{bkf}) The surface width of the stream at bankfull stage elevation, in a riffle section.	39.4	ft
Bankfull Mean Depth (d_{bkf}) Mean depth of the stream channel cross-section, at bankfull stage elevation, in a riffle section ($d_{bkf} = A_{bkf} / W_{bkf}$).	2.60	ft
Bankfull Cross-Sectional Area (A_{bkf}) Area of the stream channel cross-section, at bankfull stage elevation, in a riffle section.	102.5	ft ²
Width/Depth Ratio (W_{bkf} / d_{bkf}) <i>Bankfull Width</i> divided by <i>Bankfull Mean Depth</i> , in a riffle section.	15.2	ft/ft
Bankfull Maximum Depth (d_{max}) Maximum depth of the bankfull channel cross-section, or distance between the bankfull stage and Thalweg elevations, in a riffle section.	3.37	ft
Flood-Prone Area Width (W_{fpa}) Width of the channel at an elevation that is twice the <i>Bankfull Maximum Depth</i> , measured perpendicular to the fall line of the valley in a riffle section.	125.00	ft
Entrenchment Ratio (ER) The <i>Flood-Prone Area Width</i> divided by <i>Bankfull Width</i> (W_{fpa} / W_{bkf}), in a riffle section.	3.17	ft/ft
Channel Materials (Particle Size Index D_{50}) The D_{50} particle size index represents the median or dominant diameter of channel materials, as sampled proportionately from the channel surface between the bankfull stage and Thalweg elevations.	41	mm
Average Water Surface Slope (S) The elevation difference of water surface measurements over the stream length between two similar bed features (e.g., start of riffle to start of last riffle) for several riffle-pool or step-pool sequences, representing channel gradient.	0.0034	ft/ft
Channel Sinuosity (k) An index of channel pattern determined from stream length divided by valley length (SL / VL), or from valley slope divided by average water surface slope (S_{val} / S).	1.45	ft/ft

Stream Type	C4	See Classification Key (Figure 2-35)
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Appendix D-4 Figure 2 Reference Reach Bankfull Parameters and Classification

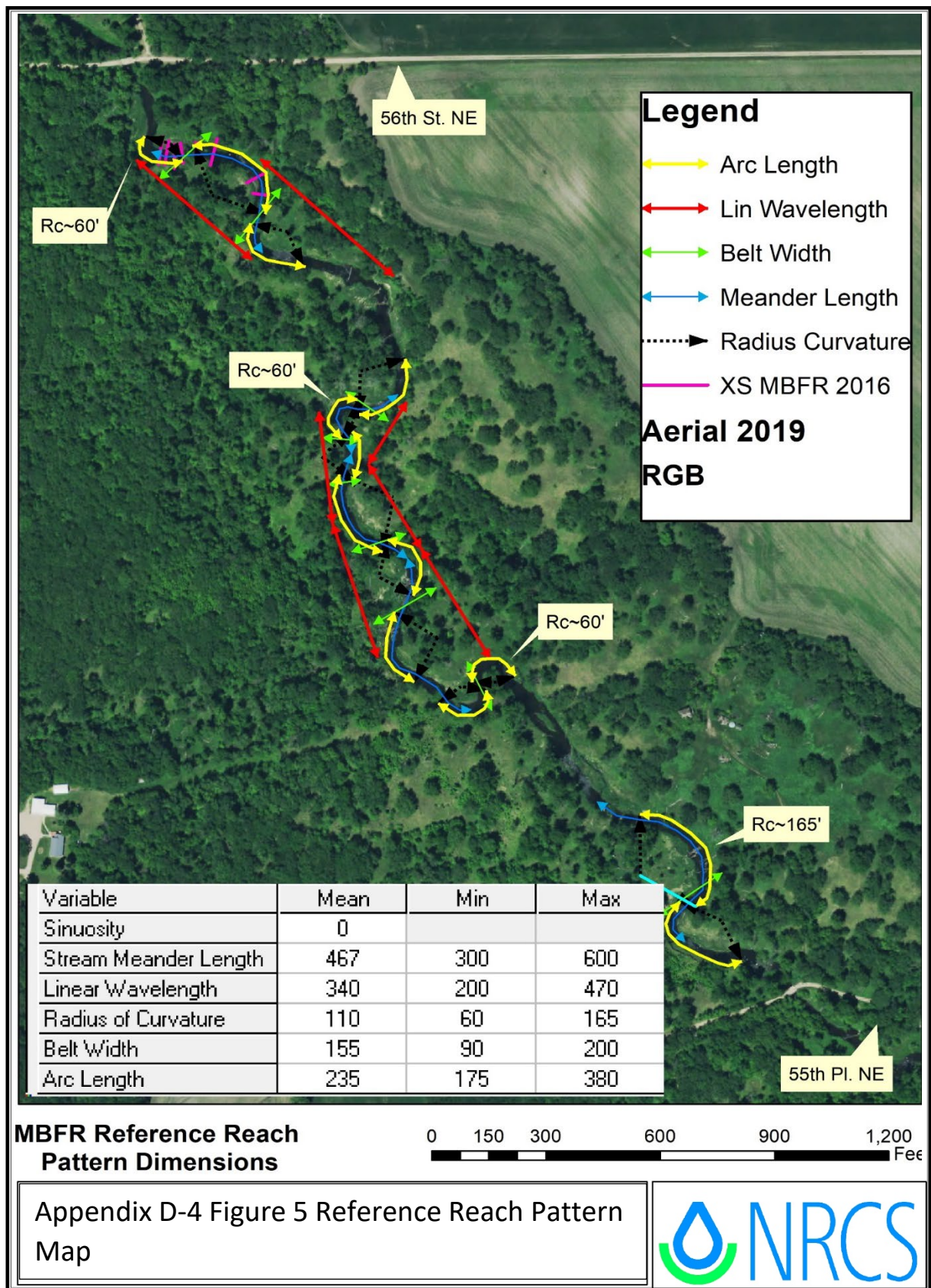




Stream: Middle Branch Forest River		Location: Reach - DS_Matecjek_2020_Elev						
Observers: JP, CF, RS		Date: 03/16/21	Valley Type: U-BR-BC Stream Type: C4					
Hydraulics	River Reach Summary Data.....2							
	Streamflow: Estimated Mean Velocity at Bankfull Stage (U_{bkt})	3.2 ft/sec	Estimation Method U/U*					
	Streamflow: Estimated Discharge at Bankfull Stage (Q_{bkt})	318 cfs	Drainage Area 122 mi ²					
Channel Pattern	Geometry			Dimensionless Geometry Ratios				
	Linear Wavelength (λ)	340	200	470 ft	Linear Wavelength to Riffle Width (λ / W_{bkt})	8.57	5.04	11.85
	Stream Meander Length (L_m)	467	300	600 ft	Stream Meander Length Ratio (L_m / W_{bkt})	11.77	7.56	15.12
	Radius of Curvature (R_c)	110	60	165 ft	Radius of Curvature to Riffle Width (R_c / W_{bkt})	2.77	1.51	4.16
	Belt Width (W_{bit})	155	90	200 ft	Meander Width Ratio (W_{bit} / W_{bkt})	3.91	2.27	5.04
	Arc Length (L_a)	235	175	380 ft	Arc Length to Riffle Width (L_a / W_{bkt})	5.92	4.41	9.58
	Riffle Length (L_r)	35	12	90 ft	Riffle Length to Riffle Width (L_r / W_{bkt})	1.26	0.30	2.52
	Individual Pool Length (L_p)	100	20	300 ft	Individual Pool Length to Riffle Width (L_p / W_{bkt})	2.50	0.50	7.50
	Pool to Pool Spacing (P_s)	235	100	340 ft	Pool to Pool Spacing to Riffle Width (P_s / W_{bkt})	5.88	2.50	8.50
Channel Profile	Valley Slope (S_{val})	0.005	ft/ft	Average Water Surface Slope (S)	0.0034	ft/ft	Sinuosity (S_{val} / S)	1.47
	Stream Length (SL)	1726	ft	Valley Length (VL)	1262	ft	Sinuosity (SL / VL)	1.45
	Low Bank Height (LBH)	start: 2.6 ft end: 2.6 ft		Max Depth (d_{max})	start: 2.6 ft end: 2.6 ft		Bank-Height Ratio (BHR) (LBH / d_{max})	start: 1 end: 1
	Facet Slopes			Dimensionless Facet Slope Ratios				
	Riffle Slope (S_{rif})	0.005	0.001	0.008 ft/ft	Riffle Slope to Average Water Surface Slope (S_{rif} / S)	1.32	0.29	2.35
	Run Slope (S_{run})	0.004	0.002	0.008 ft/ft	Run Slope to Average Water Surface Slope (S_{run} / S)	1.18	0.59	2.35
	Pool Slope (S_p)	0.001	0.000	0.002 ft/ft	Pool Slope to Average Water Surface Slope (S_p / S)	0.29	0.00	0.59
	Glide Slope (S_g)	0.002	0.000	0.003 ft/ft	Glide Slope to Average Water Surface Slope (S_g / S)	0.59	0.00	0.88
	Step Slope (S_s)			ft/ft	Step Slope to Average Water Surface Slope (S_s / S)			
	Max Depths^a			Dimensionless Depth Ratios				
	Max Riffle Depth (d_{maxrif})	3.54	2.6	4.20 ft	Max Riffle Depth to Mean Riffle Depth (d_{maxrif} / d_{bkt})	1.37	1.01	1.63
	Max Run Depth (d_{maxrun})	4	3.8	5.2 ft	Max Run Depth to Mean Riffle Depth (d_{maxrun} / d_{bkt})	1.55	1.47	2.02
	Max Pool Depth (d_{maxp})	5.30	4.40	6.50 ft	Max Pool Depth to Mean Riffle Depth (d_{maxp} / d_{bkt})	2.05	1.71	2.52
	Max Glide Depth (d_{maxg})	3.76	2.60	5.20 ft	Max Glide Depth to Mean Riffle Depth (d_{maxg} / d_{bkt})	1.46	1.01	2.02
	Max Step Depth (d_{maxs})			ft	Max Step Depth to Mean Riffle Depth (d_{maxs} / d_{bkt})			
Channel Materials	Reach^b			Reach^b			Protrusion Height^d	
	% Silt/Clay		6		D_{16}	2		mm
	% Sand		15		D_{35}	10		mm
	% Gravel		53		D_{50}	32		mm
	% Cobble		18		D_{84}	60		mm
	% Boulder		8		D_{95}	94		mm
	% Bedrock		0		D_{100}	1200		mm

Appendix D-4 Figure 4 Reference Reach
Morphological Relations and Ratios Continued





Stream:	MBForest_Reference		Stream Type:	C 4	
Location:	DS_Matecjek_2020		Valley Type:	U-BR-BC	
Observers:	JP, CF, RS		Date:	03/16/2021	
Enter Required Information for Existing Condition					
32.0	D_{50}	Median particle size of riffle bed material (mm)			
40.7	$D_{50}^{\wedge}$	Median particle size of bar or sub-pavement sample (mm)			
0.307	D_{max}	Largest particle from bar sample (ft)	93.7	(mm)	304.8 mm/ft
0.00300	S	Existing bankfull water surface slope (ft/ft)			
2.60	d	Existing bankfull mean depth (ft)			
1.65	$\gamma_s - \gamma/\gamma$	Immersed specific gravity of sediment			

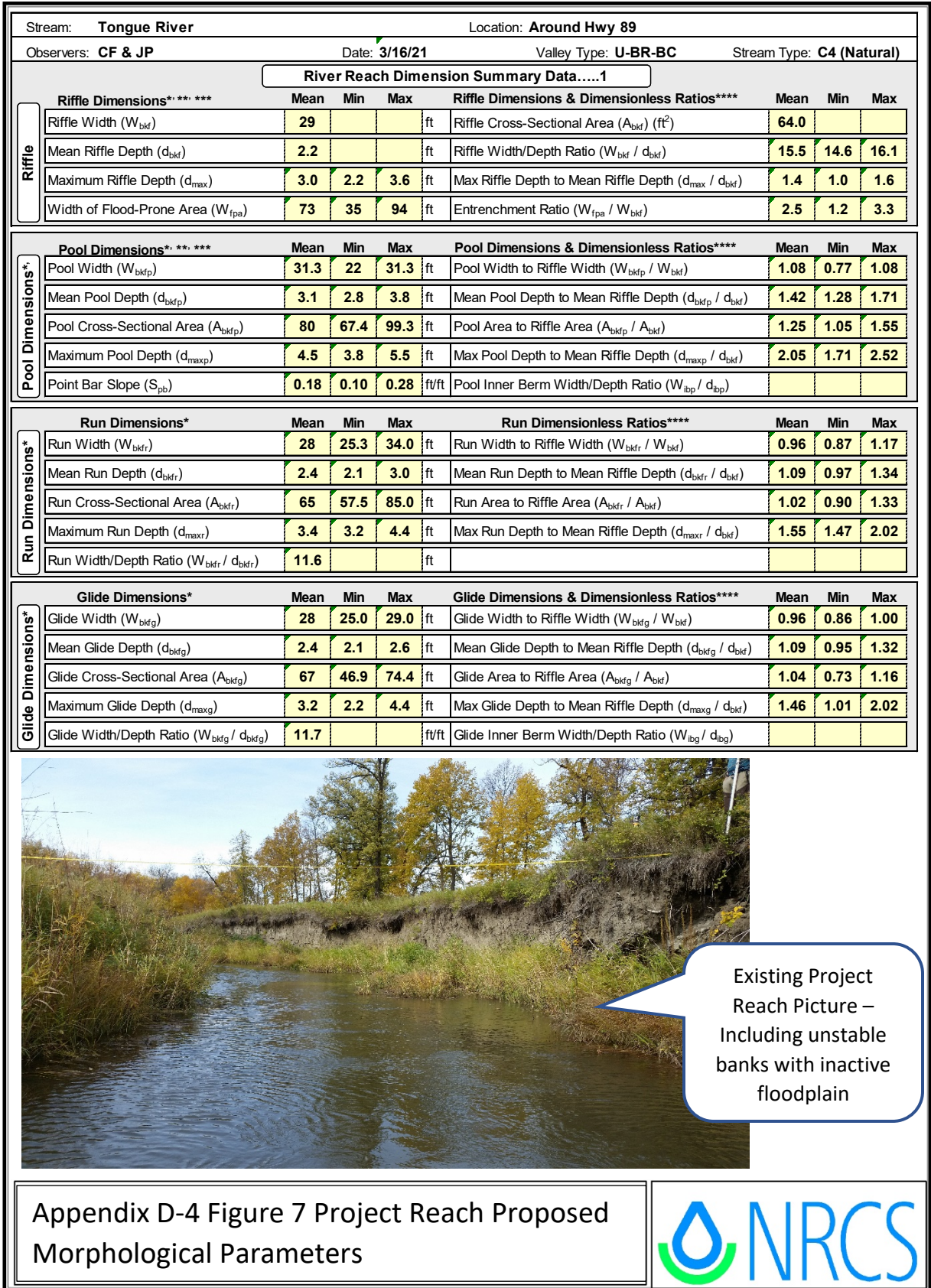
Select the Appropriate Equation and Calculate Critical Dimensionless Shear Stress					
0.79	$D_{50}/D_{50}^{\wedge}$	Range: 3 – 7	Use EQUATION 1: $\tau^* = 0.0834 (D_{50}/D_{50}^{\wedge})^{-0.872}$		
2.93	D_{max}/D_{50}	Range: 1.3 – 3.0	Use EQUATION 2: $\tau^* = 0.0384 (D_{max}/D_{50})^{-0.887}$		
0.015	τ^*	Bankfull Dimensionless Shear Stress	EQUATION USED:	2	

Calculate Bankfull Mean Depth Required for Entrainment of Largest Particle in Bar Sample					
2.50	d	Required bankfull mean depth (ft)	$d = \frac{\tau^* (\gamma_s - 1) D_{max}}{S}$ (use D_{max} in ft)		
Calculate Bankfull Water Surface Slope Required for Entrainment of Largest Particle in Bar Sample					
0.00289	S	Required bankfull water surface slope (ft/ft)	$S = \frac{\tau^* (\gamma_s - 1) D_{max}}{d}$ (use D_{max} in ft)		
Check: ☒ Stable ☐ Aggrading ☐ Degrading					

Sediment Competence Using Dimensional Shear Stress					
0.487	Bankfull shear stress $\tau = \gamma d S$ (lbs/ft ²) (substitute hydraulic radius, R, with mean depth, d) $\gamma = 62.4$, d = existing depth, S = existing slope				
Shields	CO	Predicted largest moveable particle size (mm) at bankfull shear stress τ (Figure 3-11)			
36.82	89.51				
Shields	CO	Predicted shear stress required to initiate movement of measured D_{max} (mm) (Figure 3-11)			
1.193	0.518				
Shields	CO	Predicted mean depth required to initiate movement of measured D_{max} (mm) $d = \frac{\tau}{\gamma S}$			
6.37	2.77				
Shields	CO	Predicted slope required to initiate movement of measured D_{max} (mm) $S = \frac{\tau}{\gamma d}$			
0.0074	0.0032				
Check: ☒ Stable ☐ Aggrading ☐ Degrading					

Appendix D-4 Figure 6 Reference Reach Sediment Competence Calculation for Bed Stability





Appendix D-4 Figure 7 Project Reach Proposed Morphological Parameters



Stream: Tongue River		Location: Around Hwy 89										
Observers: JP, CF		Date: 9/2/21	Valley Type: U-BR-BC									
		Stream Type: C4										
River Reach Summary Data.....2												
Hydraulics	Streamflow: Estimated Mean Velocity at Bankfull Stage (u_{bkt})		4.1 ft/sec									
	Streamflow: Estimated Discharge at Bankfull Stage (Q_{bkt})		263 cfs									
		Estimation Method	U/U*									
		Drainage Area	63 mi ²									
Channel Pattern	Geometry			Dimensionless Geometry Ratios								
	Mean	Min	Max	Mean	Min	Max						
	Linear Wavelength (λ)	248.6	146.2	343.6	ft	Linear Wavelength to Riffle Width (λ / W_{bkt})	8.57					
	Stream Meander Length (L_m)	341.4	219.3	438.6	ft	Stream Meander Length Ratio (L_m / W_{bkt})	11.77					
	Radius of Curvature (R_c)	80.4	43.9	120.6	ft	Radius of Curvature to Riffle Width (R_c / W_{bkt})	2.77					
	Belt Width (W_{bkt})	113.3	65.79	146.2	ft	Meander Width Ratio (W_{bkt} / W_{bkt})	3.91					
	Arc Length (L_a)	171.8	127.9	277.8	ft	Arc Length to Riffle Width (L_a / W_{bkt})	5.92					
	Riffle Length (L_r)	36.6	8.8	73.1	ft	Riffle Length to Riffle Width (L_r / W_{bkt})	1.26					
	Individual Pool Length (L_p)	72.5	14.5	217.5	ft	Individual Pool Length to Riffle Width (L_p / W_{bkt})	2.50					
Pool to Pool Spacing (P_s)	170.4	72.5	246.5	ft	Pool to Pool Spacing to Riffle Width (P_s / W_{bkt})	5.88						
Channel Profile	Valley Slope (S_{val})		0.005	ft/ft	Average Water Surface Slope (S)		0.0027	ft/ft	Sinuosity (S_{val} / S)		1.67	
	Stream Length (SL)		4750	ft	Valley Length (VL)		2850	ft	Sinuosity (SL / VL)		1.6667	
	Low Bank Height (LBH)	start: 2.7 ft end: 2.7 ft	Max Depth (d_{max})	start: 2.7 ft end: 2.7 ft	Bank-Height Ratio (BHR) (LBH / d_{max})		start: 1 end: 1					
	Facet Slopes			Dimensionless Facet Slope Ratios			Mean			Min		
	Riffle Slope (S_{rif})			Riffle Slope to Average Water Surface Slope (S_{rif} / S)			1.32			0.29		
	Run Slope (S_{run})			Run Slope to Average Water Surface Slope (S_{run} / S)			1.18			0.59		
	Pool Slope (S_p)			Pool Slope to Average Water Surface Slope (S_p / S)			0.29			0.00		
	Glide Slope (S_g)			Glide Slope to Average Water Surface Slope (S_g / S)			0.59			0.00		
	Step Slope (S_s)			Step Slope to Average Water Surface Slope (S_s / S)			0.00			0.00		
	Max Depths^a			Dimensionless Depth Ratios			Mean			Min		
	Max Riffle Depth (d_{maxrif})			Max Riffle Depth to Mean Riffle Depth (d_{maxrif} / d_{bkt})			1.37			1.01		
	Max Run Depth (d_{maxrun})			Max Run Depth to Mean Riffle Depth (d_{maxrun} / d_{bkt})			1.55			1.47		
	Max Pool Depth (d_{maxp})			Max Pool Depth to Mean Riffle Depth (d_{maxp} / d_{bkt})			2.05			1.71		
	Max Glide Depth (d_{maxg})			Max Glide Depth to Mean Riffle Depth (d_{maxg} / d_{bkt})			1.46			1.01		
	Max Step Depth (d_{maxs})			Max Step Depth to Mean Riffle Depth (d_{maxs} / d_{bkt})			0			0		
Channel Materials	Reach^b			Riffle^c			Bar			Protrusion Height^d		
	% Silt/Clay			D ₁₆	2.6	2.6						
	% Sand			D ₃₅	9	9						
	% Gravel			D ₅₀	25	25						
	% Cobble			D ₈₄	50	50						
	% Boulder			D ₉₅	72	72						
	% Bedrock			D ₁₀₀	74	74						
Appendix D-4 Figure 8 Project Reach Proposed Morphological Parameters Continued												



Stream:	Tongue Project		Stream Type:	
Location:	Design		Valley Type:	
Observers:			Date:	
Enter Required Information for Existing Condition				
25.0	D_{50}	Median particle size of riffle bed material (mm)		
25.0	$\hat{D}_{50}$	Median particle size of bar or sub-pavement sample (mm)		
0.243	D_{max}	Largest particle from bar sample (ft)	74	(mm) 304.8 mm/ft
0.00270	S	Existing bankfull water surface slope (ft/ft)		
2.21	d	Existing bankfull mean depth (ft)		
1.65	$\gamma_s - \gamma/\gamma$	Immersed specific gravity of sediment		

Select the Appropriate Equation and Calculate Critical Dimensionless Shear Stress				
1.00	$D_{50}/\hat{D}_{50}$	Range: 3 – 7	Use EQUATION 1: $\tau^* = 0.0834 (D_{50}/\hat{D}_{50})^{-0.872}$	
2.96	D_{max}/D_{50}	Range: 1.3 – 3.0	Use EQUATION 2: $\tau^* = 0.0384 (D_{max}/D_{50})^{-0.887}$	
0.015	τ^*	Bankfull Dimensionless Shear Stress	EQUATION USED:	2

Calculate Bankfull Mean Depth Required for Entrainment of Largest Particle in Bar Sample				
2.18	d	Required bankfull mean depth (ft)	$d = \frac{\tau^* (\gamma_s - 1) D_{max}}{S}$ (use D_{max} in ft)	
Calculate Bankfull Water Surface Slope Required for Entrainment of Largest Particle in Bar Sample				
0.00266	S	Required bankfull water surface slope (ft/ft)	$S = \frac{\tau^* (\gamma_s - 1) D_{max}}{d}$ (use D_{max} in ft)	
Check: ☒ Stable ☐ Aggrading ☐ Degrading				

Sediment Competence Using Dimensional Shear Stress				
0.372	Bankfull shear stress $\tau = \gamma d S$ (lbs/ft ²) (substitute hydraulic radius, R, with mean depth, d) $\gamma = 62.4$, d = existing depth, S = existing slope			
Shields	CO	Predicted largest moveable particle size (mm) at bankfull shear stress τ (Figure 3-11)		
27.85	73.51			
Shields	CO	Predicted shear stress required to initiate movement of measured D_{max} (mm) (Figure 3-11)		
0.951	0.376			
Shields	CO	Predicted mean depth required to initiate movement of measured D_{max} (mm) $d = \frac{\tau}{\gamma S}$		
5.65	2.23	τ = predicted shear stress, $\gamma = 62.4$, S = existing slope		
Shields	CO	Predicted slope required to initiate movement of measured D_{max} (mm) $S = \frac{\tau}{\gamma d}$		
0.0069	0.0027	τ = predicted shear stress, $\gamma = 62.4$, d = existing depth		
Check: ☒ Stable ☐ Aggrading ☐ Degrading				

Appendix D-4 Figure 9 Proposed Project Reach
Sediment Competence Calculation for Bed Stability



Existing Reach Stream & Location:		Tongue River, Pembina County, ND		
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND		
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach
	1 Landscape/Valley Type	U-BR-BC	U-BR-BC	U-BR-BC
	2 Valley Width (W_{val})	800-1,400	800-1,400	800-1,000
	3 Stream Type	B4c/F	C4	C4
	4 Drainage Area, mi^2 (DA)	63	63	122
	5 Bankfull Discharge, cfs (Q_{bkf})	197	214	318
Riffle Dimensions	6 Riffle Width, ft (W_{bkf})	Mean: 34.6 Min: 20.0 Max: 42.0	Mean: 29.0 Min: 29.0 Max: 32.1	Mean: 40.0 Min: 37.7 Max: 41.5
	7 Riffle Mean Depth, ft (d_{bkf})	Mean: 2.00 Min: 1.00 Max: 2.50	Mean: 2.21 Min: 1.99 Max: 2.21	Mean: 2.58 Min: 2.40 Max: 3.20
	8 Riffle Width/Depth Ratio (W_{bkf}/d_{bkf})	Mean: 17.3 Min: 10.0 Max: 21.0	Mean: 15.5 Min: 14.6 Max: 16.1	Mean: 15.5 Min: 14.6 Max: 16.1
	9 Riffle Cross-Sectional Area, ft^2 (A_{bkf})	Mean: 68.9 Min: 50.0 Max: 75.0	Mean: 64.0	Mean: 115.0 Min: 100.0 Max: 130.0
	10 Riffle Maximum Depth (d_{max})	Mean: 2.90 Min: 2.00 Max: 3.50	Mean: 3.03 Min: 2.22 Max: 3.59	Mean: 3.54 Min: 2.60 Max: 4.20
	11 Riffle Maximum Depth to Riffle Mean Depth (d_{max}/d_{bkf})	Mean: 1.450 Min: 1.000 Max: 1.750	Mean: 1.372 Min: 1.008 Max: 1.628	Mean: 1.372 Min: 1.008 Max: 1.628
	12 Width of Flood-Prone Area at Elevation of $2 * d_{max}$, ft (W_{fpa})	Mean: 55.0 Min: 30.0 Max: 100.0	Mean: 220.0 Min: 180.0 Max: 280.0	Mean: 99.3 Min: 47.8 Max: 125.0
	13 Entrenchment Ratio (W_{fpa}/W_{bkf})	Mean: 1.6 Min: 0.9 Max: 2.9	Mean: 7.6 Min: 6.2 Max: 8.7	Mean: 2.5 Min: 1.2 Max: 3.3

Appendix D-4 Figure 10: Master Design Table (Riffle Dimensions)



Existing Reach Stream & Location:		Tongue River, Pembina County, ND		
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND		
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach
Pool Dimensions	21 Pool Width, ft (W_{bkfp})	Mean: Min: Max:	Mean: 31.3 Min: 22.5 Max: 31.3	Mean: 43.2 Min: 31.0 Max: 43.2
	22 Pool Width to Riffle Width (W_{bkfp}/W_{bkf})	Mean: Min: Max:	Mean: 1.080 Min: 0.775 Max: 1.080	Mean: 1.080 Min: 0.775 Max: 1.080
	23 Pool Mean Depth, ft (d_{bkfp})	Mean: Min: Max:	Mean: 3.14 Min: 2.82 Max: 3.76	Mean: 3.67 Min: 3.30 Max: 4.40
	24 Pool Mean Depth to Riffle Mean Depth (d_{bkfp}/d_{bkf})	Mean: Min: Max:	Mean: 1.422 Min: 1.279 Max: 1.705	Mean: 1.422 Min: 1.279 Max: 1.705
	25 Pool Width/Depth Ratio (W_{bkfp}/d_{bkfp})	Mean: Min: Max:	Mean: 11.8 Min: 7.0 Max: 13.1	Mean: 11.8 Min: 7.0 Max: 13.1
	26 Pool Cross-Sectional Area, ft ² (A_{bkfp})	Mean: Min: Max:	Mean: 80.1 Min: 67.4 Max: 99.3	Mean: 144.0 Min: 121.2 Max: 178.5
	27 Pool Area to Riffle Area (A_{bkfp}/A_{bkf})	Mean: Min: Max:	Mean: 1.252 Min: 1.054 Max: 1.552	Mean: 1.252 Min: 1.054 Max: 1.552
	28 Pool Maximum Depth (d_{maxp})	Mean: Min: Max:	Mean: 4.53 Min: 3.76 Max: 5.56	Mean: 5.30 Min: 4.40 Max: 6.50
	29 Pool Maximum Depth to Riffle Mean Depth (d_{maxp}/d_{bkf})	Mean: Min: Max:	Mean: 2.054 Min: 1.705 Max: 2.519	Mean: 2.054 Min: 1.705 Max: 2.519
	30 Point Bar Slope (S_{pb})	Mean: Min: Max:	Mean: 0.180 Min: 0.100 Max: 0.280	Mean: 0.180 Min: 0.100 Max: 0.280
Appendix D-4 Figure 11: Master Design Table Continued (Pool Dimensions) 				



Existing Reach Stream & Location:		Tongue River, Pembina County, ND		
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND		
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach
Run Dimensions	38 Run Width, ft (W_{bkfr})	Mean: Min: Max:	Mean: 27.7 Min: 25.3 Max: 34.0	Mean: 38.2 Min: 34.9 Max: 46.9
	39 Run Width to Riffle Width (W_{bkfr}/W_{bkf})	Mean: Min: Max:	Mean: 0.956 Min: 0.873 Max: 1.173	Mean: 0.956 Min: 0.873 Max: 1.173
	40 Run Mean Depth, ft (d_{bkfr})	Mean: Min: Max:	Mean: 2.40 Min: 2.13 Max: 2.96	Mean: 2.80 Min: 2.49 Max: 3.46
	41 Run Mean Depth to Riffle Mean Depth (d_{bkfr}/d_{bkf})	Mean: Min: Max:	Mean: 1.085 Min: 0.965 Max: 1.341	Mean: 1.085 Min: 0.965 Max: 1.341
	42 Run Width/Depth Ratio	Mean:	Mean: 13.7	Mean: 13.7
	43 Run Cross-Sectional Area, ft ² (A_{bkfr})	Mean: Min: Max:	Mean: 65.1 Min: 57.5 Max: 85.0	Mean: 117.0 Min: 103.3 Max: 152.7
	44 Run Area to Riffle Area (A_{bkfr}/A_{bkf})	Mean: Min: Max:	Mean: 1.017 Min: 0.898 Max: 1.328	Mean: 1.017 Min: 0.898 Max: 1.328
	45 Run Maximum Depth (d_{maxr})	Mean: Min: Max:	Mean: 3.42 Min: 3.25 Max: 4.45	Mean: 4.00 Min: 3.80 Max: 5.20
	46 Run Maximum Depth to Riffle Mean Depth (d_{maxr}/d_{bkf})	Mean: Min: Max:	Mean: 1.550 Min: 1.473 Max: 2.016	Mean: 1.550 Min: 1.473 Max: 2.016
Glide Dimensions	47 Glide Width, ft (W_{bkfg})	Mean: Min: Max:	Mean: 27.9 Min: 25.0 Max: 29.0	Mean: 38.5 Min: 34.5 Max: 40.0
	48 Glide Width to Riffle Width (W_{bkfg}/W_{bkf})	Mean: Min: Max:	Mean: 0.963 Min: 0.864 Max: 1.000	Mean: 0.963 Min: 0.864 Max: 1.000
	49 Glide Mean Depth, ft (d_{bkfg})	Mean: Min: Max:	Mean: 2.40 Min: 2.09 Max: 2.91	Mean: 2.80 Min: 2.44 Max: 3.40
	50 Glide Mean Depth to Riffle Mean Depth (d_{bkfg}/d_{bkf})	Mean: Min: Max:	Mean: 1.085 Min: 0.946 Max: 1.318	Mean: 1.085 Min: 0.946 Max: 1.318
	51 Glide Width/Depth Ratio	Mean:	Mean: 13.8	Mean: 13.8
	52 Glide Cross-Sectional Area, ft ² (A_{bkfg})	Mean: Min: Max:	Mean: 66.8 Min: 46.9 Max: 74.4	Mean: 120.0 Min: 84.3 Max: 133.6
	53 Glide Area to Riffle Area (A_{bkfg}/A_{bkf})	Mean: Min: Max:	Mean: 1.043 Min: 0.733 Max: 1.162	Mean: 1.043 Min: 0.733 Max: 1.162
	54 Glide Maximum Depth (d_{maxg})	Mean: Min: Max:	Mean: 3.22 Min: 2.22 Max: 4.45	Mean: 3.76 Min: 2.60 Max: 5.20
	55 Glide Maximum Depth to Riffle Mean Depth (d_{maxg}/d_{bkf})	Mean: Min: Max:	Mean: 1.457 Min: 1.008 Max: 2.016	Mean: 1.457 Min: 1.008 Max: 2.016
Appendix D-4 Figure 12: Master Design Table Continued (Run/Glide Dimensions)				



Existing Reach Stream & Location:		Tongue River, Pembina County, ND		
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND		
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach
Channel Pattern	72 Linear Wavelength, ft (λ)	Mean: 280.0 Min: 100.0 Max: 450.0	Mean: 248.6 Min: 146.2 Max: 343.6	Mean: 340.0 Min: 200.0 Max: 470.0
	73 Linear Wavelength to Riffle Width (λ/W_{bkf})	Mean: 8.092 Min: 2.890 Max: 13.006	Mean: 8.571 Min: 5.042 Max: 11.848	Mean: 8.571 Min: 5.042 Max: 11.848
	74 Stream Meander Length, ft (L_m)	Mean: 230.0 Min: 100.0 Max: 400.0	Mean: 341.4 Min: 219.3 Max: 438.6	Mean: 467.0 Min: 300.0 Max: 600.0
	75 Stream Meander Length Ratio (L_m/W_{bkf})	Mean: 6.647 Min: 2.890 Max: 11.561	Mean: 11.772 Min: 7.562 Max: 15.125	Mean: 11.772 Min: 7.562 Max: 15.125
	76 Belt Width, ft (W_{blt})	Mean: 150.0 Min: 110.0 Max: 210.0	Mean: 113.3 Min: 65.8 Max: 146.2	Mean: 155.0 Min: 90.0 Max: 200.0
	77 Meander Width Ratio (W_{blt}/W_{bkf})	Mean: 4.335 Min: 3.179 Max: 6.069	Mean: 3.907 Min: 2.269 Max: 5.042	Mean: 3.907 Min: 2.269 Max: 5.042
	78 Radius of Curvature, ft (R_c)	Mean: 170.0 Min: 65.0 Max: 270.0	Mean: 80.4 Min: 43.9 Max: 120.6	Mean: 110.0 Min: 60.0 Max: 165.0
	79 Radius of Curvature to Riffle Width (R_c/W_{bkf})	Mean: 4.913 Min: 1.879 Max: 7.803	Mean: 2.773 Min: 1.512 Max: 4.159	Mean: 2.773 Min: 1.512 Max: 4.159
	80 Arc Length, ft (L_a)	Mean: 265.0 Min: 120.0 Max: 380.0	Mean: 171.8 Min: 127.9 Max: 277.8	Mean: 235.0 Min: 175.0 Max: 380.0
	81 Arc Length to Riffle Width (L_a/W_{bkf})	Mean: 7.659 Min: 3.468 Max: 10.983	Mean: 5.924 Min: 4.411 Max: 9.579	Mean: 5.924 Min: 4.411 Max: 9.579
	82 Riffle Length (L_r), ft	Mean: 50.0 Min: 10.0 Max: 100.0	Mean: 36.6 Min: 8.8 Max: 73.1	Mean: 35.0 Min: 12.0 Max: 90.0
	83 Riffle Length to Riffle Width (L_r/W_{bkf})	Mean: 1.445 Min: 0.289 Max: 2.890	Mean: 1.260 Min: 0.302 Max: 2.521	Mean: 1.260 Min: 0.302 Max: 2.521
	84 Individual Pool Length, ft (L_p)	Mean: 40.0 Min: 15.0 Max: 60.0	Mean: 72.5 Min: 14.5 Max: 217.5	Mean: 100.0 Min: 20.0 Max: 300.0
	85 Pool Length to Riffle Width (L_p/W_{bkf})	Mean: 1.156 Min: 0.434 Max: 1.734	Mean: 2.500 Min: 0.500 Max: 7.500	Mean: 2.500 Min: 0.500 Max: 7.500
	86 Pool-to-Pool Spacing, ft (P_s)	Mean: 100.0 Min: 30.0 Max: 150.0	Mean: 170.4 Min: 72.5 Max: 246.5	Mean: 235.0 Min: 100.0 Max: 340.0
	87 Pool-to-Pool Spacing to Riffle Width (P_s/W_{bkf})	Mean: 2.890 Min: 0.867 Max: 4.335	Mean: 5.875 Min: 2.500 Max: 8.500	Mean: 5.875 Min: 2.500 Max: 8.500
Appendix D-4 Figure 13: Master Design Table Continued (Channel Pattern)				



Existing Reach Stream & Location:		Tongue River, Pembina County, ND		
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND		
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach
Sinuosity and Slope	88 Stream Length (SL)	4750.0	9850.0	600.0
	89 Valley Length (VL)	2850.0	6000.0	400.0
	90 Valley Slope (S_{val})	0.0049	0.0049	0.0050
	91 Sinuosity (k)	SL/VL: 1.67 VS/S: 1.64	SL/VL: 1.64	SL/VL: 1.47 VS/S: 1.45
	92 Average Water Surface Slope (S)	0.0030	$S = S_{val}/k$ 0.0030	0.0034
Floodpl	93 Floodplain Width, ft (W_f)	Mean: 1000.00 Min: Max:	Mean: 1000.0 Min: Max:	Mean: 500.0 Min: 350.0 Max: 600.0
	97 Flood-Prone Area Width, ft (W_{fpa})	Mean: 43.0 Min: 40.0 Max: 50.0	Mean: 125.0 Min: 110.0 Max: 200.0	Mean: 125.0 Min: 110.0 Max: 200.0
Flood-Prone Area	98 Flood-Prone Area Surface Depth Limit, ft (d_{fpa})	Mean: 4 Min: Max:	Mean: 4.00 Min: Max:	Mean: 5.00 Min: Max:
	99 Low Bank Height (LBH)	Mean: 8.00 Min: 8.00 Max: 8.00	Mean: 2.10 Min: Max:	Mean: 2.60 Min: 2.60 Max: 2.60
Degree of Incision	100 Maximum Bankfull Depth (d_{max}) at Same Location as Low Bank Height (LBH) Measurement	Mean: 3.15 Min: 2.50 Max: 3.80	Mean: 2.10 Min: Max:	Mean: 2.60 Min: 2.60 Max: 2.60
	101 Bank-Height Ratio (LBH/ d_{max})	Mean: 2.65 Min: 2.11 Max: 3.20	Mean: 1.00 Min: Max:	Mean: 1.00 Min: 1.00 Max: 1.00
Channel Materials	123 Particle Size Distribution of Active Bed or Pavement Channel Materials			
	D_{16} (mm)	2.5	2.5	2.0
	D_{35} (mm)	5.3	9.0	10.0
	D_{50} (mm)	8.8	25.0	32.0
	D_{84} (mm)	21.2	50.0	60.0
	D_{95} (mm)	30.2	72.0	94.0
	D_{100} (mm)	32.0	74.0	1200.0
	124 Particle Size Distribution of Bar Material or Sub-pavement			
	D_{16} (mm)	0.0	0.0	
	D_{35} (mm)	4.2	9.0	
	D_{50} (mm)	6.9	25.0	40.7
	D_{84} (mm)	15.6	50.0	
	D_{95} (mm)	21.9	72.0	
	D_{max} (mm)	30.0	74.0	120.0

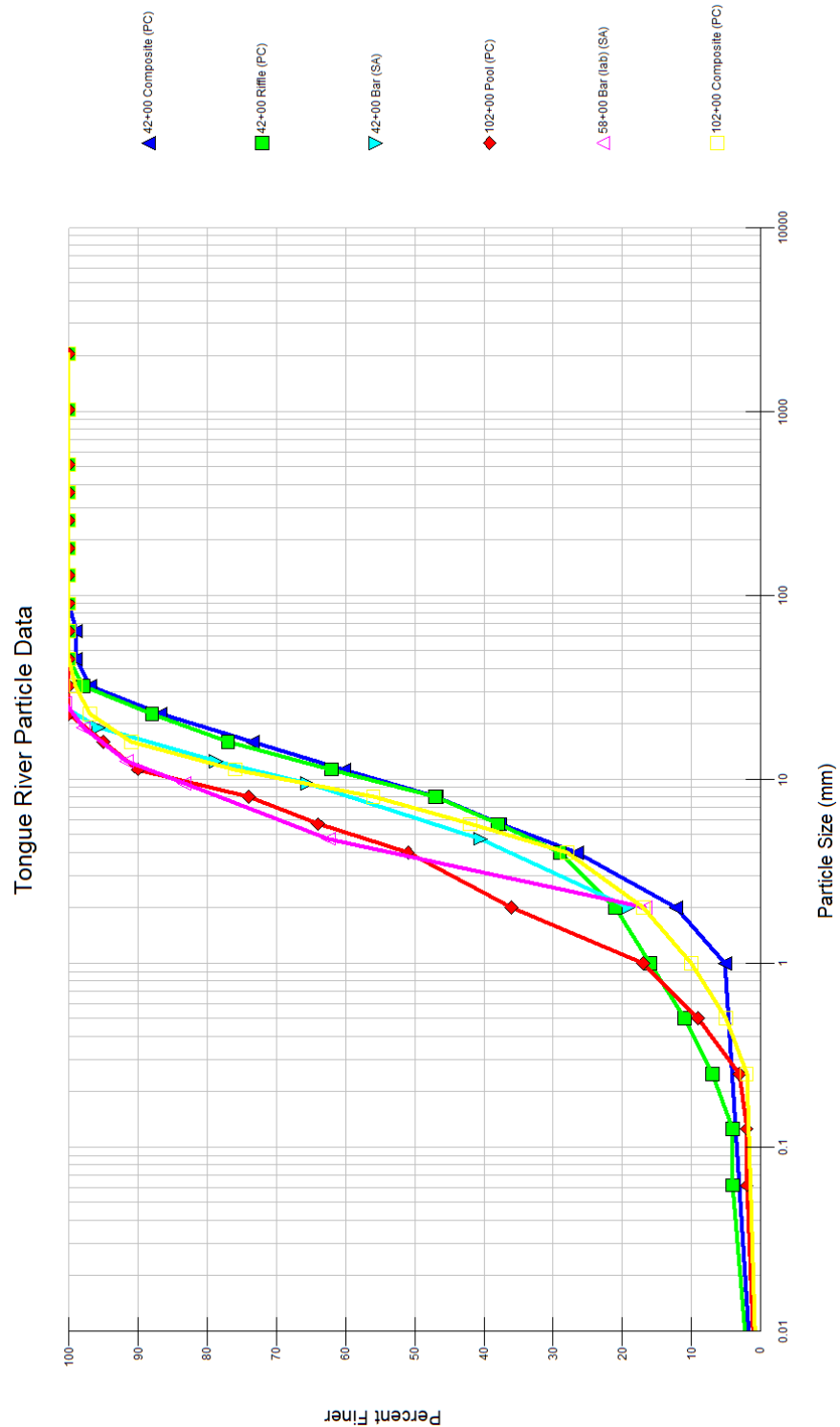
Appendix D-4 Figure 14: Master Design Table Continued (Sinuosity, Slope, Bed)



Existing Reach Stream & Location:		Tongue River, Pembina County, ND			
Reference Reach Stream & Location:		Middle Branch Forest River, Walsh County, ND			
Entry Number & Variable		Existing Reach	Proposed Design Reach	Reference Reach	
Hydraulics & Fluvial	125	Estimated Bankfull Mean Velocity, ft/sec ($\bar{U}_{bkf}$)	3.20	3.30	3.2
	126	Estimated Bankfull Discharge, cfs (Q_{bkf}); Compare with Regional Curve	136.0	136.0	318.0
Sediment Competence	133	Calculated bankfull shear stress value, lbs/ft ² (τ)	0.317	0.372	0.487
	134	Predicted largest moveable particle size (mm) at bankfull shear stress, (τ)	Shields: 23.6	Shields: 28	Shields: 36.8
			Colo.: 65.4	Colo.: 74	Colo.: 89.5
	135	Largest particle size to be moved (D_{max}) (mm) (see #124: Particle Size Distribution of Bar Material)	30	64	93.7
	136	Predicted shear stress required to initiate movement of D_{max} (mm)	Shields: 0.400	Shields: 0.827	Shields: 1.193
			Colo.: 0.110	Colo.: 0.308	Colo.: 0.518
	137	Predicted mean depth required to initiate movement of D_{max} (mm), $d = \tau/\gamma S$ (τ = predicted shear stress, γ = 62.4, S = existing or design slope)	Shields: 1.97	Shields: 4.91	Shields: 6.37
			Colo.: 1.97	Colo.: 1.83	Colo.: 6.37
	138	Predicted slope required to initiate movement of D_{max} (mm) $S = \tau/\gamma d$ (τ = predicted shear stress, γ = 62.4, d = existing or design depth)	Shields: 0.0041	Shields: 0.0060	Shields: 0.0074
			Colo.: 0.0011	Colo.: 0.0022	Colo.: 0.0032
139	Bankfull dimensionless shear stress (τ^*)	0.013	0.017	0.015	
140	Required bankfull mean depth d_{bkf} (ft) using dimensionless shear stress equation: $d_{bkf} = \tau^*(\gamma_s - 1)D_{max}/S$ (Note: D_{max} in ft)	0.64	2.14	2.50	
141	Required bankfull water surface slope S (ft) using dimensionless shear stress equation: $S = \tau^*(\gamma_s - 1)D_{max}/d_{bkf}$ (Note: D_{max} in ft)	0.0013	0.0026	0.0029	

Appendix D-4 Figure 15: Master Design Table Continued (Sediment Competence)





Appendix D-4 Figure 16: Tongue Riverbed Gradation Data



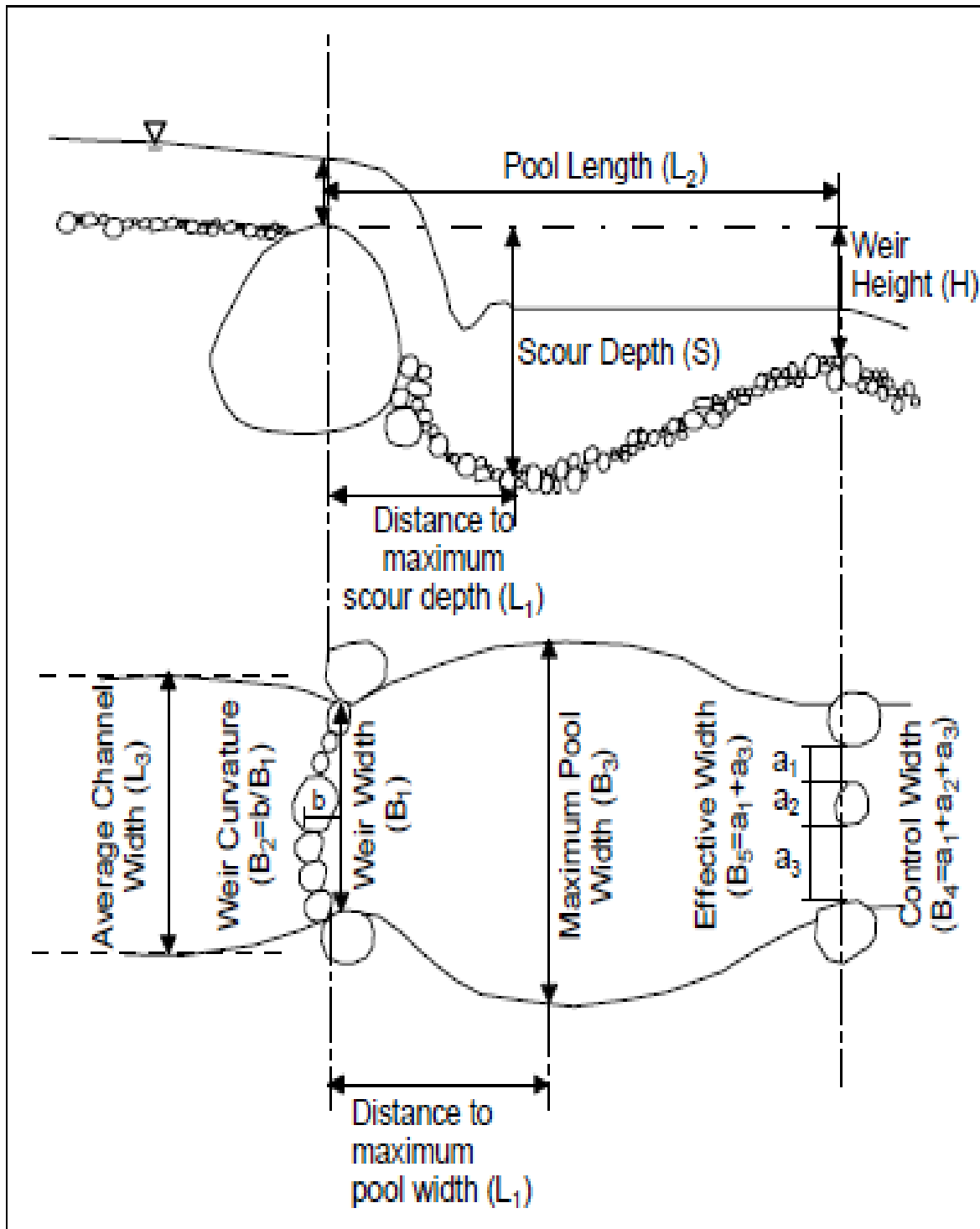


Figure 8.2– Step-pool characteristics (from Thomas et al. 2000)

Appendix D-4 Figure 17: Scour pool from USBR (U.S. Dept. of Interior, 2007)



TONGUE RIVER RESTORATION GEOTECHNICAL REPORT

Date: 9/2/2021

SUBJECT: Tongue River Restoration – Geotechnical Investigation

TRIP SYNOPSIS

On August 19th 2021 a soil investigation was conducted at the site of the Tongue River Restoration project near Mountain, ND along the Tongue River in Pembina County in North Dakota. On the 18th of August, four drilling locations were located within a proposed borrow site for the project. The four drilling locations would be drilled with a portable truck mounted drill rig. A drill depth of 16'-0" was set for the investigation. Tom Schanandore (myself) the investigation civil engineer and Tory Nygaard the engineering technician arrived onsite around 7:45 a.m. on the morning of the 19th to begin drilling. A planned test of in-place native soil density was conducted as well. Four tests were conducted using a nuclear density gauge. Soil samples of the soil pulled up by the drilling auger, were sampled in 5-gallon buckets and sample plastic bags. The four drilling holes were completed by noon on the 19th and then the soil density tests were conducted. One density test was conducted in the floor plain, near boring BH-03 and the other three were conducted along the Tongue River channel; one in the channel, with some sediment deposits and along the exposed surface of the river bank. Tory and myself left the site around 1:00 p.m.

PURPOSE

The purpose of the soil investigation was to acquire soil geotechnical data, pertaining to the Tongue River Restoration project. Although the site was previously drilled, the related logs and notes had been lost. The drilling holes were conducted within the proposed borrow areas for the project, which will be used for compacted fill along the river channel. Soil samples were taken for the purpose of determining compaction potential through a standard proctor test, Atterberg limits (PL, LL, PI) for classification and in-place density using a nuclear density gauge. .

LOCATION AND DESCRIPTION

The project is located in Pembina County, 4.5 miles northwest of Mountain, ND. The investigation was conducted specifically in the southwest quarter of section 28 and the southeast quarter of 29 in township 161N 56W.



Figure 1: Soil Investigation Map

PROJECT COORDINATION

ND one call was contacted two days prior to the start of the drilling. A week prior Tory and I were in contact and coordinated the arrival time and desired drilling start time. An investigation plan was developed prior to meeting on site, to estimate equipment need and time on-site.

EQUIPMENT

Truck mounted drilling rig (probe truck) with 4" diameter by 4'-0" long drilling auger was used to drill the four holes. 5-gallon buckets and plastic bags were used to sample the soil. Hand shovels were used to dig down past the topsoil in order to conduct the soil density tests.

GENERAL SITE GEOLOGY

The geologic processes that formed the landscape near the soil investigation site, include continental glacier expanse and retreat, as well as glacial Lake Agassiz. The controlling bedrock feature in this area is Pierre Shale, which was laid in the Cretaceous period and is very shallow in the vicinity of the Pembina Escarpment. The continental glacier and Lake Agassiz drained away ~10,000 years ago with warming global temperatures; which included massive rivers with high erosive power. Historic and recent erosion through the Tongue River valley brought the channel bottom to the controlling bedrock formation Pierre Shale.

BORING EXPLORATIONS AND SOIL DENSITY TESTS

Four boring holes were drilled during the soil investigation. Each boring was drilled to a depth of 16'-0" which corresponds to four separate drills and sample recoveries per boring hole. The soil recovery upon the auger bit was documented by field logbook as well as with photos, prior to



TONGUE RIVER RESTORATION GEOTECHNICAL REPORT

collecting soil for samples. The four borings were consistent in recovery, having a loose dry and dark gray color within the first auger flight (0'-4'); Stiffer, dry and dark gray color within the second auger flight (4'-8'); soft, very moist and gray color within the third auger flight (8'-12') and very soft, wet (saturated) and gray color within the fourth auger flight (12'-16'). See appendix for soil logs and descriptions. With the exception of the topsoil, all soil collected and viewed on site during the drilling was classified as a lean clay (CL) material.

The soil density tests were conducted using a Troxler nuclear eGauge. The test site within the flood plan was conducted near soil boring BH-03. 1'-0" of topsoil was removed prior to leveling the ground for the test. The second, third and fourth test sites were conducted in and near the river channel. The second was within the river channel, the second was conducted upon deposits within the channel and the third was conducted upon the exposed riverbank soil.

Boring Logs: (all soil classified as lean clay (CL) material)

Boring Number	Boring Depth Range	Description
BH-01	0'-4'	Loose, dry, little silt, dark gray color
	4'-8'	More stiff, few specs of sand/gravels, dark gray color
	8'-12'	Soft, moist, specs of gravel, gray color
	12'-16'	Very soft, wet/saturated, gray color, water at 13'
BH-02	0'-4'	Loose slightly silty, dry, dark gray color
	4'-8'	Soft, moist, specs of gravel, dark gray color
	8'-12'	Very soft, specs of sand, very moist, gray color
	12'-16'	Very soft, very moist, gray color, water at 12'
BH-03	0'-4'	Loose, dry, dark gray color
	4'-8'	Loose, dry, dark gray color
	8'-12'	Lost soil in hole
	12'-16'	Soft, moist, gray color, no runny water, not saturated
BH-04	0'-4'	Loose, dry, dark gray color
	4'-8'	Stiffer, dry, dark gray color
	8'-12'	Stiffer from 8'-9' dark gray color, transition to very soft and moist at 9'-12' gray color
	12'-16'	Very soft, wet, gray color, water at 13'

SOIL TESTING RESULTS

Soil Testing Results		
Atterberg Limits	Plastic Limit (PL)	38
	Liquid Limit (LL)	60
	Plasticity Index (PI)	22
Standard Proctor	Max. Soil Density	82 pcf
	Optimum Moisture	29 %



TONGUE RIVER RESTORATION GEOTECHNICAL REPORT

In-situ Soil Dry Density	T1 (near BH-03)	74.9 pcf
	T2 (at river channel)	74.3 pcf
	T3 (at river channel sediments)	80.0 pcf
	T4 (at riverbank)	67.0 pcf

Table 1: Soil Testing Results

GEOLOGIC CONSIDERATIONS FOR CONSTRUCTION

Borrow Material:

The borrow material collected during the soil investigation was tested for compaction capability (standard proctor) and for Atterberg limits. The resulting liquid limit (39) and plasticity index (22) classifies the soil as a lean clay (CL) material. The resulting proctor density potential resulted at a density of 82 lbs per cubic foot and an optimum moisture content of 29 percent. Although this material can be classified and acts mechanically as a lean clay (CL) material, historical geotechnical and geological understanding of the material also classifies it as bedrock/shale material. This material was expected to be found in this area of the Tongue River floodplain. The resulting soil density from the standard proctor test is low when compared to other lean clay materials. This shale/bedrock material historically has a range of in-situ soil densities ranging from 79 pcf to 125 pcf and soil moisture contents of 19 to 41 percent. More exploration and soil testing must be conducted to ensure that the material can be compacted to a more desirable density.

Per the borrows, the first 8'-0" of the borrow material was dry to slightly moist. This would allow for the excavation of the soil using scrapper equipment. Below 8'-0" in depth the lean clay material becomes very soft and very moist and at around 12'-0" becomes saturated with water. The top 8'-0" of soil would be adequate to use as fill since the moisture content at those depths would be below the optimum moisture content and a simple excavate and place, with water added as needed, could be performed rather easily. The one concern for excavation would be the ability of scrapper equipment to successfully traverse the site when the upper dryer lean clay material is removed. At this point long reach excavators could be used to excavate the dryer material down to the moister material. If the moist/wet deeper material is to be used, it would not be able to be placed immediately after excavation. The material would have to be dried/condition prior to placement. The proposed 6:1 side slopes of the excavation ponds (after excavation) should be adequate for slope stability.

Considerations for Placement of Compacted Fill:

The borrow material to be used for fill, will have to be compacted adequately in-place using rammer or pad-foot vibratory roller equipment. The lean clay material at optimum moisture will need to be impacted with enough force to remove air voids as much as possible. As discussed above, most of the excavated borrow material should be able to be brought directly to the fill site and placed and conditioned with water if needed. The moister material, if used, will have to be conditioned/dried prior to placement. A nearby collection/conditioning site will have to be established to conduct this conditioning process.

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From statics, the following conditions must be satisfied:

(1) $\Sigma F_H = 0$ In terms of areas: Area(O'A'A) + Area(CJE) - Area(BAFE) = 0

$$\frac{1}{2} \cdot P_A \cdot H + \frac{1}{2} \cdot (P_C + P_J) \cdot Z - P_C \cdot D = 0$$

Solve for Z:

$$Z := \frac{2 \cdot P_C \cdot D - P_A \cdot (H)}{P_C + P_J} = 4.34 \text{ ft}$$

(2) ΣM about any point is zero

$$\Sigma M_F := \frac{1}{2} \cdot P_A \cdot (H) \cdot \left(D + \frac{H}{3}\right) + \frac{1}{2} \cdot (P_C + P_J) \cdot Z \cdot \frac{Z}{3} - P_C \cdot D \cdot \frac{D}{2} = -0.067 \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

*Change "D" iteratively until ΣM_F is near zero

$$D_{\text{design1}} := D = 17.75 \text{ ft}$$

Since a factor of safety of 1.5 was applied to the cohesion, no increase in penetration is needed. Equilibrium should now be checked for the long term case of $c=0$ and $\phi=18^\circ$. The phi angle is a generic value provided by a geologist with familiarity with the Pierre Shale material in the region.

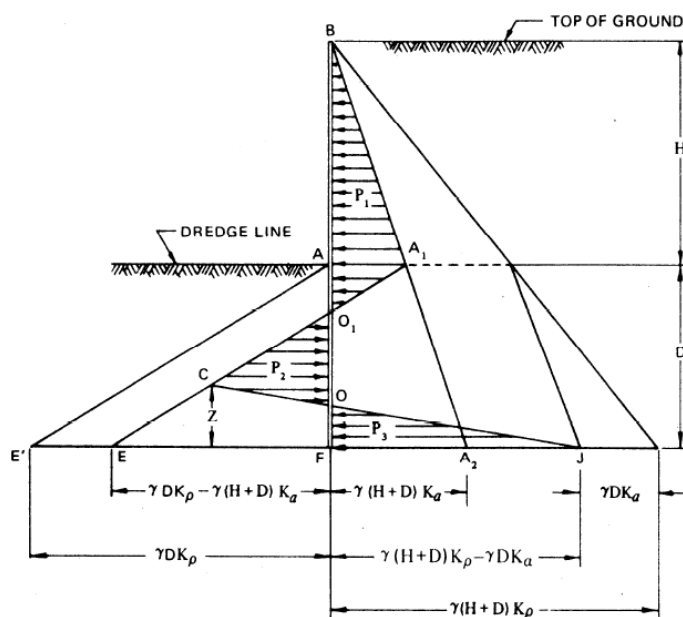
$$c := 0 \text{ psf} \quad \phi := 18 \text{ deg}$$

$$D := 18.43 \text{ ft}$$

$$K_a := \frac{1 - \sin(\phi)}{1 + \sin(\phi)} = 0.528 \quad \text{active lateral earth pressure coefficient}$$

$$K_p := \frac{1 + \sin(\phi)}{1 - \sin(\phi)} = 1.894 \quad \text{passive lateral earth pressure coefficient}$$

Conventional Assumed Pressure Diagram



Determine wall pressures

$$P_{A1} := \gamma_e \cdot H \cdot K_a = 593.847 \text{ psf}$$

$$P_{A2} := P_{A1} + \gamma_e \cdot D \cdot K_a = (1.81 \cdot 10^3) \text{ psf}$$

$$P_E := \gamma_e \cdot D \cdot K_p - P_{A2}$$

$$P_E = (2.554 \cdot 10^3) \text{ psf}$$

$$P_J := \gamma_e \cdot (H + D) \cdot K_p - \gamma_e \cdot D \cdot K_a$$

$$P_J = (5.279 \cdot 10^3) \text{ psf}$$

From statics, the following conditions must be satisfied:

(1) $\Sigma F_H = 0$ In terms of areas: $\text{Area}(\text{BAA}_1) + \text{Area}(\text{AA}_1\text{A}_2\text{F}) + \text{Area}(\text{ECJ}) - \text{Area}(\text{EA}_1\text{A}_2) = 0$

$$\frac{1}{2} \cdot H \cdot P_{A1} + (P_{A1} + P_{A2}) \cdot \frac{D}{2} + (P_E + P_J) \cdot \frac{Z}{2} - (P_E + P_{A2}) \cdot \frac{D}{2} = 0$$

Solve for Z

$$Z := \frac{(P_E - P_{A1}) \cdot D - H \cdot P_{A1}}{P_E + P_J} = 3.93 \text{ ft}$$

(2) ΣM about any point is zero

$$\Sigma M_F := \frac{1}{2} \cdot H \cdot P_{A1} \cdot \left(D + \frac{H}{3}\right) + P_{A1} \cdot \frac{D^2}{2} + (P_E + P_J) \cdot \frac{Z^2}{6} - (P_E + P_{A2}) \cdot \frac{D^2}{6} + (P_{A2} - P_{A1}) \cdot \frac{D^2}{6} = 65.512 \text{ lbf}$$

$$\Sigma M_F = 0.066 \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

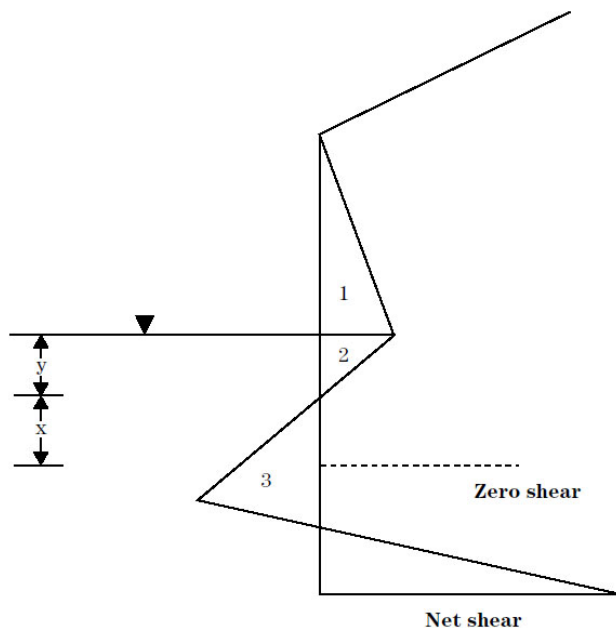
To determine the design embedment depth, increase D by 20 percent for a factor of safety

$$FS := 1.3 \quad \text{factor of safety}$$

$$D_{\text{design2}} := FS \cdot D = 23.959 \text{ ft} \quad \text{design embedment depth}$$

$$D_{\text{design}} := \max(D_{\text{design1}}, D_{\text{design2}}) = 23.959 \text{ ft} \quad \text{*maximum design embedment depth from different analyses}$$

Locate the point of zero shear



$$\gamma' := \gamma_e = 125 \text{ pcf}$$

$$y := \frac{P_{A1}}{\gamma' \cdot (K_p - K_a)} = 3.476 \text{ ft}$$

$$P_1 := \frac{1}{2} \cdot P_{A1} \cdot H = (2.672 \cdot 10^3) \frac{\text{lbf}}{\text{ft}}$$

$$P_2 := \frac{1}{2} \cdot P_{A1} \cdot y = (1.032 \cdot 10^3) \frac{\text{lbf}}{\text{ft}}$$

$$x := \sqrt{\frac{2 \cdot (P_1 + P_2)}{\gamma' \cdot (K_p - K_a)}} = 6.586 \text{ ft}$$

$$P_3 := \frac{1}{2} \cdot \gamma' \cdot (K_p - K_a) \cdot x^2 = (3.705 \cdot 10^3) \frac{\text{lbf}}{\text{ft}}$$

Solve for the maximum moment, which occurs at the point of zero shear.

$$l_1 := \frac{H}{3} + y + x = 13.062 \text{ ft}$$

$$l_2 := \frac{2 \cdot y}{3} + x = 8.903 \text{ ft}$$

$$l_3 := \frac{x}{3} = 2.195 \text{ ft}$$

$$M_{max} := P_1 \cdot l_1 + P_2 \cdot l_2 - P_3 \cdot l_3 = (3.596 \cdot 10^4) \frac{\text{ft} \cdot \text{lb}}{\text{ft}}$$

Determine the minimum section modulus per foot of wall

$$f_y := 50 \text{ ksi}$$

$$f_b := 0.5 \cdot f_y = 25 \text{ ksi}$$

$$S_{min} := \frac{M_{max}}{f_b} = 17.263 \frac{\text{in}^3}{\text{ft}}$$

Determine the minimum cross sectional area of sheet pile per foot of wall

$$V_{max} := \max(P_1, P_2, P_3) = 3704.549 \frac{\text{lb}}{\text{ft}}$$

$$f_v := 0.33 \cdot f_y = 16.5 \text{ ksi}$$

$$A_{min} := \frac{V_{max}}{f_v} = 0.225 \frac{\text{in}^2}{\text{ft}}$$

Determine required panel length; panels are provided in 5-foot increments

$$\text{Panel_Length} := H + D_{design} = 32.959 \text{ ft}$$

$$\text{Panel_Length} := 35 \text{ ft}$$

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Sizing and stability analysis for rock cross vanes and bank protection.

References

1. NRCS National Engineering Handbook, Part 654 Stream Restoration Design, Chapter 11 Rosgen Geomorphic Channel Design TS14B Scour Calculations, TS14C Stone Sizing Criteria, TS14G Grade Stabilization Techniques, 2007.
2. U.S. Bureau of Reclamation, Rock Weir Design Guidance, 2016.
3. OR NRCS Tech Note 25: Incorporation of Large Wood into Engineering Structures, 2001.

Hydraulic Conditions

From RAS model, most extreme hydraulic conditions in the project reach by event:

	Q2	Q10	Q25	Q50	
Flow (cfs)	301	598	981	1270	
Shear (psf)	0.24	0.29	0.33	0.40	
Velocity (fps)	4.1	5.1	6.6	7.2	
Max Depth (ft)	3.3	4.2	4.9	5.5	max, max D in project reach excl just upstr of bridge
Avg Depth (ft)	3.1	3.8	4.2	4.5	avg max D in project reach excl just upstrm of bridge

Cross Vanes

$$Wbkf := 29.0 \text{ ft}$$

$$W := \frac{Wbkf}{3} = 9.667 \text{ ft} \quad \text{throat distance and horiz distance for arms (Ref 1)}$$

$$A := 30 \text{ deg} \quad \text{arm angle from bank Ref 1 recommends 20-30 deg, I have always done 30 in narrower channels like this.}$$

$$AL := \frac{W}{\sin(A)} = 19.333 \text{ ft} \quad \text{arm length}$$

$$S := .03 \quad \text{ft/ft arm slope Ref 1 indicates 2-7%, staying with 3% to limit scour}$$

$$AD := AL \cdot S = 0.58 \text{ ft} \quad \text{arm elev drop}$$

Per Ref 2, above design parameters do not fall into those of vanes with observed failures. Note that all 4 cross vanes will be installed on straight channel reaches, and be set to that the top of the vane is on grade with the low point of the bankfull channel. Ref 1 guidance for key depth is 3h for gravel channels, but in this case our h is 0 so cannot apply any of the formulas provided for channel spanning structures. Compute scour on the arm of the rock vane, which will rise above the channel, per Kuhnle (TS14B-65) as if was a barb:

$$y := 3.4 \text{ ft} \quad \text{Q10, mean flow depth approaching arm} = 4.2 - 0.8 = 3.4 \text{ ft}$$

$$Lc := W = 9.667 \text{ ft} \quad \text{length spur crest perpendicular to flow}$$

$$x := \frac{Lc}{y} = 2.843 \quad \text{for } Lc/y \text{ between 1 and 25} \rightarrow a := 0.5$$

$$\phi := 45 \text{ deg} \quad \text{angle of repose, gravel channel bed}$$

$$\theta := 90 \text{ deg} \quad \text{side slope of rock- shot rock from the Warroad Pit is fairly square}$$

$$K1 := 0.75 \cdot \left(1 + \frac{2 \cdot \tan(\phi)}{\tan(\theta)} \right)^{.5} = 0.75$$

$$z := K1 \cdot \left(\frac{Lc}{y} \right)^a \cdot y = 4.3 \text{ ft} \quad \text{expected scour depth, Q10}$$

$$y := 4.1 \text{ ft} \quad \text{Q25, mean flow depth approaching arm} = 4.9 - 0.8 = 4.1 \text{ ft}$$

$$x := \frac{Lc}{y} = 2.358 \quad \text{still between 1 and 25}$$

$$z := K1 \cdot \left(\frac{Lc}{y} \right)^a \cdot y = 4.722 \text{ ft} \quad \text{expected scour depth, Q25}$$

$$y := 4.7 \text{ ft} \quad \text{Q50, mean flow depth approaching arm} = 5.5 - 0.8 = 4.7 \text{ ft}$$

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$$x := \frac{Lc}{y} = 2.057 \quad \text{still between 1 and 25}$$

$$z := K1 \cdot \left(\frac{Lc}{y} \right)^a \cdot y = 5.055 \text{ ft} \quad \text{expected scour depth, Q50}$$

Largest angular boulders available at the closest shot rock pit (Warroad MN) will be 3-4 ft so bottom of footer boulder will be 6-8 ft below bankfull. Adequate to the Q50, for scour, although we would typically design a cross vane to be stable at the Q10. Check boulder sizing- plan to just specify those large 1.5-2 ton boulders with DOT Class II riprap to fill voids.

TS14G-2:

$$q_{design} := \frac{598}{29} = 20.621 \quad \text{cfs/ft over weir at Q10}$$

$$D_{50} := 5.23 \cdot S^{.43} \cdot q_{design} = 23.876 \quad \text{inches}$$

$$q_{design} := \frac{981}{29} = 33.828 \quad \text{cfs/ft over weir at Q25}$$

$$D_{50} := 5.23 \cdot S^{.43} \cdot q_{design} = 39.168 \quad \text{inches}$$

$$q_{design} := \frac{1270}{29} = 43.793 \quad \text{cfs/ft over weir at Q50}$$

$$D_{50} := 5.23 \cdot S^{.43} \cdot q_{design} = 50.707 \quad \text{inches}$$

TS14G-5:

$$S_o := .0027$$

$$q_{design} := \frac{598}{29} = 20.621 \quad \text{cfs/ft over weir at Q10}$$

$$D_{50} := \left(\frac{q_{design}}{.52 \cdot (S_o)^{-1.5}} \right)^{\frac{1}{1.89}} = 0.064 \quad \text{ft} \quad \text{apparent this channel is too low of a slope to utilize this equation}$$

TS14G-6:

$$\tau := 1.816 \quad \text{kg/sqm bankfull shear stress, proposed channel} = 0.372 \text{ psf} \times 4.882$$

$$D_{min} := .1724 \cdot \ln(\tau) + .6349 = 0.738 \quad \text{m} \times 39.37 = 29.1 \text{ in}$$

By comparison, using Dmin = 0.75 x D50 from TS14G-2 would yield:

$$D_{min} := 0.75 \cdot 23.876 = 17.907 \quad \text{in Q10}$$

$$D_{min} := 0.75 \cdot 39.168 = 29.376 \quad \text{in Q25}$$

$$D_{min} := 0.75 \cdot 50.707 = 38.03 \quad \text{in Q50}$$

The 36-48" boulders proposed look good, for perhaps all but Q50- ok. Typical design is for Q10.

Ref 2 doesn't provide any different methods that would apply in this instance. Actual gradation is not particularly relevant to a cross vane, given it is best constructed with large square boulders anyway. Per rock arch rapids design notes, ND DOT Class II riprap is sufficient so using that between the larger boulders is acceptable. The 2'x3'x4' rock noted in the drawings appears to be adequate for stability to at least a Q25, which is acceptable.

Sheet pile proposed to be installed as a cutoff through the fill material and into undisturbed riverbed at 55+31 and 86+25. Simply used the ratio that was computed for the sheet pile wall at the rock arch ramp for this preliminary design, which was ~25% above ground, 75% below ground.

55+31 SP wall will be 4 ft above existing ground, so need min. 12 ft additional = 16 ft

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86+25 SP wall will be 5.3 ft above existing ground, so need min. 15.9 ft additional = 21.2 ft

Note: sheet pile cutoffs below rock cross vanes used with success on urban incised river restoration projects, in very erodible fine grained soils, in CA on projects by West Consultants.

Bank Protection

Type 1 planned to be used in highest shear stress/velocity areas on outer banks of meander bends. Goal is to stabilize bank long enough for mature woody vegetation to become established. There is ~20 of sound, toppled over trees with large rootwads in the river channel due to both landslides on the upper slope and bank erosion. In addition, levee removal will result in another 20-30 mature trees being removed. The contractor can either purchase those from the landowner or source other trees from offsite and haul them in with sidedumps. Tree row clearing in croplands is not unusual in the area and will be an additional source of trees.

$S_L := 0.79$ $S_S := 2.7$ $C_{DRW} := 1.2$ $C_{DB} := 0.3$ $p := 0.3$ $D_{RW} := 4. \text{ ft}$ $L_{RW} := 1.5 \text{ ft}$ $N_{RW} := 6$ $D_L := 1.17 \text{ ft}$ $L := 18 \text{ ft}$ $N_L := 12$ $D_B := 3.56 \text{ ft}$ $N := 6$ $L_B := 14 \text{ ft}$ $N_B := 6$ $O := 110 \text{ pcf}$ $H := 2$ $V := 6.2 \text{ fps}$ $f := 0.9$ $A := 40 \text{ sqft}$	Specific gravity, typical NE ND hardwoods (basswood, ash) Specific gravity, rock Rootwad drag coefficient Boulder drag coefficient Proportion of voids in rootwad (visual observation, fallen trees in project reach). Rootwad diameter (typical of fallen trees in project reach) Rootwad length (typical of fallen trees in project reach) Number of rootwads Log diameter (typical of available trees) Log length Number of logs Boulder diameter (as if round) Number of boulders Log length to be buried Number of boulders to be buried Soil density Average soil depth over logs Q10 velocity * 1.22 to adjust for bend radius. NEH 654, Chapter 8. R/W = 107) / 29 = 3.6 max and 41/29 = 1.4 minimum. W/D ratio is 29/2.2 = 13, so use bottom chart in Figure 8-1. 107 degree bend radius indicates ~ 1.22 V _{bend} / V _{avg} . Friction factor, channel bed Projection of all elements perpendicular to flow	
$V_{RW} := 3.14 \cdot \left(\frac{D_{RW}}{2} \right)^2 \cdot L_{RW} \cdot (1 - p) \cdot N_{RW} = 79.128 \text{ cft}$	Volume of rootwads	
$V_T := 3.14 \cdot L \cdot \left(\frac{D_L}{2} \right)^2 = 19.343 \text{ cft}$	Volume of logs	
$V_R := 3.14 \cdot \frac{(D_B)^3}{6} = 23.612 \text{ cft}$	Volume of rock, weight: $Rock := V_R \cdot S_S \cdot \frac{62.4}{2000} = 1.989 \text{ tons}$	
$F_G := (V_{RW} + V_T) \cdot S_L \cdot 62.4 = 4854.205 \text{ lbs}$	Weight of logs & rootwads	
$F_B := (V_{RW} + V_T) \cdot 62.4 = 6144.563 \text{ lbs}$	Buoyant force on logs & rootwads (immersed)	
$W_S := L_B \cdot D_L \cdot H \cdot (O - 62.4) = 1559.376 \text{ lbs}$	Soil weight (immersed)	
$W_R := V_R \cdot (S_S - 1) \cdot 62.4 = 2504.736 \text{ lbs}$	Rock weight (immersed)	
$F_F := \frac{V^2}{2 \cdot 32.2} \cdot A \cdot 62.4 \cdot C_{DRW} = 1787.818 \text{ lbs}$	Sliding force on structure	
$F_{U1} := (F_G + W_S + W_R - F_B) \cdot f = 2496.378 \text{ lbs}$	Frictional resistance at base of logs, if soil stays	

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$$F_{U2} := (F_G + W_R - F_B) \cdot f = 1092.94 \quad \text{lbs} \quad \text{Frictional resistance at base of logs, if soil erodes}$$

$$FS_{B1} := \frac{(F_G + W_S + W_R)}{F_B} = 1.451 \quad \text{Factor of safety against buoyancy if soil remains in place}$$

$$FS_{B2} := \frac{(F_G + W_R)}{F_B} = 1.198 \quad \text{Factor of safety against buoyancy if soil erodes}$$

$$FS_{S1} := \frac{F_{U1}}{F_F} = 1.396 \quad \text{Factor of safety against sliding if soil remains in place}$$

$$FS_{S2} := \frac{F_{U2}}{F_F} = 0.611 \quad \text{Factor of safety against sliding if soil erodes}$$

At Q25:

$$V := 8 \quad \text{fps} \quad \text{Q25 velocity} \times 1.22$$

$$F_F := \frac{V^2}{2 \cdot 32.2} \cdot A \cdot 62.4 \cdot C_{DRW} = 2976.596 \quad \text{lbs} \quad \text{Sliding force on structure}$$

$$FS_{S1} := \frac{F_{U1}}{F_F} = 0.839 \quad \text{Factor of safety against sliding if soil remains in place}$$

$$FS_{S2} := \frac{F_{U2}}{F_F} = 0.367 \quad \text{Factor of safety against sliding if soil erodes}$$

At Q50:

$$V := 8.8 \quad \text{fps} \quad \text{Q50 velocity} \times 1.22$$

$$F_F := \frac{V^2}{2 \cdot 32.2} \cdot A \cdot 62.4 \cdot C_{DRW} = 3601.681 \quad \text{lbs} \quad \text{Sliding force on structure}$$

$$FS_{S1} := \frac{F_{U1}}{F_F} = 0.693 \quad \text{Factor of safety against sliding if soil remains in place}$$

$$FS_{S2} := \frac{F_{U2}}{F_F} = 0.303 \quad \text{Factor of safety against sliding if soil erodes}$$

PS580 design criteria is FS 1.2 min. at the Q10 which the design meets if soil over logs remains in place. Hence the decision to utilize encapsulated soil lifts- most conservative bank treatment above LWD, good success in MN and IA, although one of the more expensive to install. Proposed outer fabric layer on lifts is GEOCOIR 700 or equal, which is manufacturer rated for avg velocity of 10 ft/sec. Observed to have performed well in MT on rivers that experience ice flows during spring runoff. Major risk factor to this bank protection treatment is heavy beaver browse on the live cuttings before they can establish. Hence the decision to also do live lump transplants which tend to be more resilient.

Type 2 bank protection is in lower shear stress/velocity areas, mainly banks on riffles, upper portion of runs, lower portion glides. Same GEOCOIR 700 fabric over cobble/tops of trees matrix at toe with live stakes. Same concern with beaver browse exists.

Debris Collectors

Several individuals on the planning team were really wanted to use beaver dam analogues in this project. The fact that large natural beaver dams have been observed to fail in relatively minor spring runoff years is evidence that high velocity in the incised channel makes this technique impractical. In the upstream area where the goal is to encourage natural aggradation of 1-2.5 feet, they can be incorporated. That said, there are two beaver dams that have been in place in that reach for several years in a row. The need for debris collectors and the cross vane at 101+75 would be evaluated based on beaver dam locations in that reach during construction. Design reflects

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planned structures if no dams were present.

$N_{RW} := 2$ Number of rootwads

$N_L := 2$ Number of logs

$L := 50$ ft Log length

$N := 3$ Number of boulders

$L_B := 0$ ft Log length to be buried

$V := 4.1$ fps Q2 velocity (these are intended to be short lifespan structures)

$A := 72$ sqft Projection of all elements perpendicular to flow

$F_G := (V_{RW} + V_T) \cdot S_L \cdot 62.4 = 4854.205$ lbs Weight of logs & rootwads

$F_B := (V_{RW} + V_T) \cdot 62.4 = 6144.563$ lbs Buoyant force on logs & rootwads (immersed)

$W_R := V_R \cdot (S_S - 1) \cdot 62.4 = 2504.736$ lbs Rock weight (immersed)

$F_F := \frac{V^2}{2 \cdot 32.2} \cdot A \cdot 62.4 \cdot C_{DRW} = 1407.279$ lbs Sliding force on structure

$F_{U2} := (F_G + W_R - F_B) \cdot f = 1092.94$ lbs Frictional resistance, due to weight

$FS_{B2} := \frac{(F_G + W_R)}{F_B} = 1.198$ Factor of safety against buoyancy

$A_{BUR} := 10$ sqft Approximate area of rootwads & boulders, buried in bed, banks

$LB := 60$ psf/ft Assumed allowable lateral bearing pressure, bed & banks

$b := 1$ ft Avg burial depth, rootwads & boulders (considering bed scour)

$F_x := A_{BUR} \cdot LB \cdot b = 600$ lbs Frictional resistance, due to burial

$FS_{S2} := \frac{(F_{U2} + F_x)}{F_F} = 1.203$ Factor of safety against sliding

Sliding FS does not credit resistance of mature tree so logs should remain stable at Q2. These would be placed ~2 miles upstream of the private bridge, therefore potential for full size trees moving that far is minimal.

Check stability of " dia buried 2.5' into the bed with 5.5' exposed, at 3 ft spacing:

$C_{DRW} := 0.4$ Assume 40% porosity for beaver dam or debris jam on posts

$A := 16.5$ sqft 3 ft spacing by 5.5 ft high debris

$F_F := \frac{V^2}{2 \cdot 32.2} \cdot A \cdot 62.4 \cdot C_{DRW} = 107.5$ lbs Lateral force on post

$b := 2.5$ ft Burial depth

$A_{BUR} := 1.04$ sqft 5/12 x 2.5, area of post below ground

$F_x := A_{BUR} \cdot LB \cdot b = 156$ lbs Lateral resistance of post

$SF_x := \frac{F_x}{F_F} = 1.451$ Factor of safety, tipping due to soils failure

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$$F_B := 575 \quad \text{psi} \quad \text{Allowable bending, Hem-Fir \#2 posts}$$

$$F'_B := \frac{F_F}{\frac{5}{12} \cdot \frac{5}{12}} = 619.203 \quad \text{psi}$$

$$SF_B := \frac{F_B}{F'_B} = 0.929 \quad \text{Factor of safety, bending}$$

Typical that beaver dam analogue structures do not pass stability analysis. In this case relying on backwater effect of trees/boulders to remain stable at even a 2-yr flood. These are not critical structures to the project, however, so ok.

Size bypass pipeline and pump for dewatering river reaches for project construction. Considered a larger culvert for gravity diversion, but not practical/cost effective without boring through the Hwy 89 embankment on north side of the valley- fill height ~30 ft. A dewatering system utilizing portable pumps was developed, as shown on the drawings, using 8 locations for temporary pump sites. Temporary pipe will need to be placed and snaked between mature trees on the riverbank above the construction extents. In locations of active landslides, pipe will run up a steep grade to the upper terrace. Concrete block will be used for temporary thrust blocks. A check valve will be placed at the pump, obviously, to avoid it draining when the pump(s) shutoff. Portability of pipe is a key consideration- important that a small bobcat be able to transport and place it. Total length of temporary pipe will range from 800 to 1,400 ft. Maximum elevation rise to get above slide areas is 53 ft (site #4 to #3). For hydraulics, simply apply that elevation head at the outlet end of the pipeline to be conservative (in reality it will be at an intermediate point). Plan to just use a standard foot valve on the suction line, and then specify a portable welded wire screen 20 ft upstream, to be installed upstream to exclude fish from the intake.

Hydrology:

No gauges available on the Tongue River, other than immediately below Renwick Dam which is not usable for determining low flows (construction project anticipated August to November). NRCS installed one in early 2021, but for now the best data source available is USGS Report SIR2015-5184 "Regression equations to estimate seasonal flow duration, n-day high-flow frequency, and n-day low-flow frequency at sites in North Dakota using data through water year 2009."

Table 4. Regression equations for seasonal flow duration estimates in North Dakota.

[ft³/s, cubic foot per second; chi², used to measure goodness of fit and to test hypotheses and obtain confidence intervals for the variance of a normally distributed variable; Q_p, duration flow for x exceedance probability; ≤, less than or equal to; STREAMLENGTH, sum of length of all mapped streams, in miles; COMPRAT, compactness ratio, dimensionless; DRNAREA, drainage area, in square miles; ISOLAKEDA, percentage of basin drainage area that drains to isolated lakes and ponds, in percent; MINBELEV, minimum basin elevation, in feet (North American Vertical Datum of 1988 [NAVD 88]); PRECIP, mean annual precipitation, in inches (normal period 1961–90); SLOPERAT, slope ratio, dimensionless]

Regression equation	Range of explanatory variables	Censor value, in ft ³ /s	Number of stations	Number of stations censored	Estimated residual standard error, in log units	Chi ²
Hydrologic zone A						
$\log Q_{p90} = -5.422 + 1.646 \log(\text{STREAMLENGTH})$		0.100	23	14	0.9885	13.49
$\log Q_{p75} = -6.118 + 2.110 \log(\text{STREAMLENGTH})$		0.100	23	12	0.9370	21.62
$\log Q_{p50} = -4.099 + 1.437 \log(\text{STREAMLENGTH}) + 2.276 \log(\text{COMPRAT})$	$13.91 \leq \text{STREAMLENGTH} \leq 3.373; 1.89 \leq \text{COMPRAT} \leq 4.77$	0.500	23	6	0.3545	47.78
$\log Q_{p25} = -2.507 + 1.257 \log(\text{STREAMLENGTH}) + 1.350 \log(\text{COMPRAT})$		0.500	23	1	0.3393	47.18
$\log Q_{p10} = -1.320 + 1.029 \log(\text{STREAMLENGTH}) + 1.455 \log(\text{COMPRAT})$		0.500	23	0	0.2813	47.60

$SL := 112$ STREAMLENGTH (miles), determined from GIS analysis

$CR := 2.85$ COMPRAT, determined from GIS analysis

$Q_{90} := 10^{-5.422 + 1.646 \cdot \log(SL)} = 0.009$ cfs (90% probability Q will exceed, March-September)

$Q_{75} := 10^{-6.118 + 2.11 \cdot \log(SL)} = 0.016$ cfs (75% probability Q will exceed, March-September)

$Q_{50} := 10^{-4.099 + 1.437 \cdot \log(SL) + 2.276 \cdot \log(CR)} = 0.76$ cfs (50% probability Q will exceed, March-September)

$Q_{25} := 10^{-2.507 + 1.257 \cdot \log(SL) + 1.35 \cdot \log(CR)} = 4.818$ cfs (25% probability Q will exceed, March-September)

$Q_{10} := 10^{-1.32 + (1.209) \cdot \log(SL) + 1.455 \cdot \log(CR)} = 65.964$ cfs (10% probability Q will exceed, March-September)

Peak flows in ND are April-June, with spring runoff. By the time construction starts in August, flows are typically very low (< 1 ft in riffle sections). Sizing for the 25% probability duration seems a conservative choice- 4.8 cfs or 2154 gpm.

Hydraulics/Pipe Design:

From past experience evaluating pipe material alternatives for seasonal above ground irrigation pipeline that is durable enough to be repeatedly dragged over rough ground, PE 4710 pipe setup with with bolted mechanical connectors on joints will work best. Pipe can then be moved in 40 ft sections (or shorter if so desired by the contractor). Will need to place a 3-way air/vac relief in one of the line sections to place at the summit, to avoid generating vacuum conditions in the line. They will need to have bolted connections with a variety of bend angles available also.

$$Q := 2154 \text{ gpm}$$

Try 12" SDR 32.5 PE 4710, 40 ft sections = 268 lbs

$$d := 11.919 \text{ in}$$

$$h_{elev} := 53 \text{ ft}$$

$$L := 1400 \text{ ft}$$

$$C := 150$$

$$V := \frac{Q \cdot 1 \text{ gpm}}{\pi \cdot \left(\frac{d \cdot 1 \text{ in}}{2} \right)^2} = 6.194 \frac{\text{ft}}{\text{s}}$$

Friction loss through pipeline:

$$h_f := \frac{10.44 \cdot L \cdot Q^{1.85}}{C^{1.85} \cdot d^{4.8655}} \cdot 1 \text{ ft} = 11.725 \text{ ft}$$

Minor losses (pump discharge side):

$$6" \text{ swing check valve} \quad k_1 := 2.0$$

$$6" \text{ to } 12" \text{ manifold (2 tees, } 12/6=2 \text{ sudden expansion)} \quad k_2 := 2 \cdot 0.8 + 2 \cdot .56 = 2.72$$

$$\text{tee to 3-way CAV} \quad k_3 := 0.5$$

$$K := k_1 + k_2 + k_3 = 5.22$$

$$h_{f_minor} := K \cdot \frac{V^2}{2 \cdot g} = 3.112 \text{ ft}$$

$$TDH := h_{elev} + h_f + h_{f_minor} = 67.837 \text{ ft} \quad \sim 30 \text{ psi} \quad \text{So, SDR 32.5 would be fine (72\% of 64 psi = 46 psi allowable for longterm use). No closing valves on the system, so no potential for significant pressure surges.}$$

Check 10" SDR 32.5, 40 ft sections = 191 lbs

$$d := 10.05 \text{ in}$$

$$V := \frac{Q \cdot 1 \text{ gpm}}{\pi \cdot \left(\frac{d \cdot 1 \text{ in}}{2} \right)^2} = 8.712 \frac{\text{ft}}{\text{s}}$$

Friction loss through pipeline:

$$h_f := \frac{10.44 \cdot L \cdot Q^{1.85}}{C^{1.85} \cdot d^{4.8655}} \cdot 1 \text{ ft} = 26.886 \text{ ft}$$

$$TDH := h_{elev} + h_f + h_{f_minor} = 82.998 \text{ ft}$$

$\sim 36 \text{ psi}$ So, SDR 32.5 would still work. Each 40 ft piece would be 77 lb lighter, but additional velocity will require that more portable thrust blocks will be placed.

HDPE pipe may reach high temperatures, particularly where it is in direct sunlight, transferring heat to water in the pipe. Table A.2 in The Handbook of Polyethylene Pipe (Plastics Pipe Institute, 2008) indicates a sustained temperature of 80 degrees (which for a max run length of 1400 ft seems conservatively high) has a multiplier of 0.94 for PR. So allowable $0.72 \times 0.94 \times 64 = 43 \text{ psi}$.

Our design for the Bismarck PMC irrigation system intake utilizes 10, 20 ft long sections of 12" dia SDR 13.5 HDPE that is seasonally installed to a trailer mounted floating pump in the river. Pipe remains full of water throughout the season, and bolted connections on the pipe ends are rated by the manufacturer for the same tensile loads as the pipe. It is not particularly easy to line up the pipe at the joints, which will be a challenge for the contractor- but this seems like the best solution.

Pumps:

Dewatering pump(s) will be placed upstream of the segment of the river to be dewatered for construction, on a point bar adjacent to an existing pool. Less than 5 ft of suction lift required. Typical low flows during the August-November construction period are 1-2 cfs, so makes more sense to have two pumps with one set to start up after the water reaches a certain elevation in the pool used to set the suction intakes. Both pumps will need to have float activated automatic start/stop controls, vacuum pump automatic priming systems. Double wasled fuel tank to service pumps (typical storage capacity on the pump trailer only provides ~26 hours of pumping) should be located out of the bankfull channel, but always have adequate pump capacity for 5 days on hand. Contractor will build an HDPE manifold for the two 6" discharge hose lines to connect to, to transition to the 10" HDPE.

Example product that would work is Thompson 6HT-DIST-4LE2T shown below:

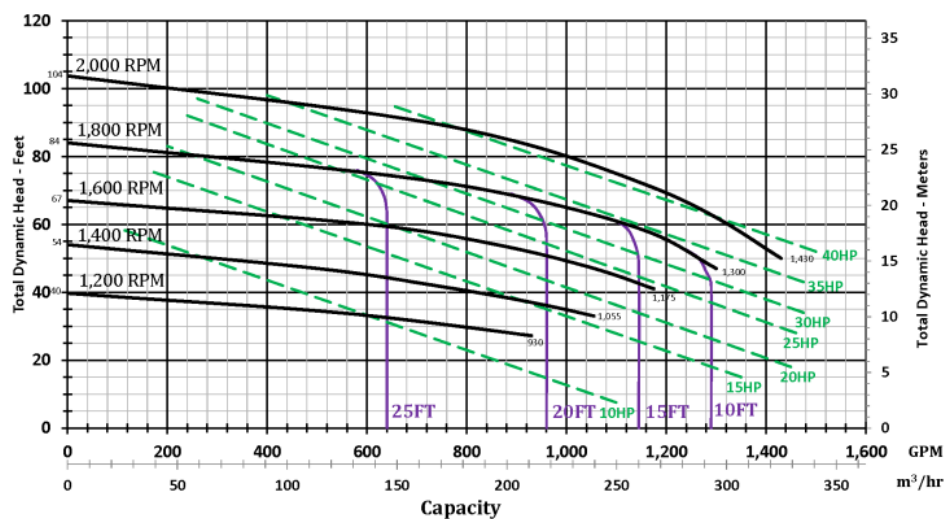
With its heavy-duty cast-iron construction, ability to self-prime when filled with water, and re-prime automatically, the Thompson 6HT leads the industry in construction, industrial and wastewater applications. The Thompson 6HT is designed for moderate flow rates up to 1,430 gpm and can handle heads up to 104 feet.



Photo shown may not be exact model.

Consult factory for other versions & options including site trailers, sound attenuation, etc.

Pump End Materials	
Pump Casing	Heavy-duty class 30 cast-iron.
Impeller	Dynamically balanced, non-clogging, semi-open, 65-45-12 ductile iron with rear-equalizing vanes to reduce axial loading and prolong seal and bearing life; diameter 9.74".
Mechanical Seal	Dry-running, grease or oil lubricated with Tungsten Carbide rotating and Silicon Carbide stationary seal faces. Single inside mounted, non- pusher type with self-adjusting elastomeric bellows. Other components are 304 stainless steel and Viton.
Head	Rugged, back pull out design, heavy-duty class 30 cast iron with tapered bore design.
Bearings	Heavy-duty grease lubricated to carry both axial and radial loads.
Bearing Frame	Heavy-duty class 30 cast iron.
Shaft	High quality stress-proof steel.
Wear Plate	Replaceable, class 30 cast iron with abrasion resistant rubber facing to extend service life.



Rock Ramp Calculations:

1. Base Stone Analysis 1:
 - a. Equation; $D_{50} = (q_{\text{design}})^{0.56} \times S^{0.43} \times 5.23$ (eq. TS14C-14, Abt and Johnson)
 - b. Assumptions;
 - i. $Q_{\text{design}} = 872$ cfs (50-year event rock ramp flow rate for Alternative 2, from RAS)
 - ii. $W_{\text{design}} = 23$ feet, bottom width of ramp
 - iii. $q_{\text{design}} = Q_{\text{design}} / W_{\text{design}} = 872 \text{ cfs} / 23 \text{ feet} = 37.9 \text{ cfs/ft}$
 - iv. Slope = 0.05 ft/ft
 - c. Calculation; $D_{50} = (37.9)^{0.56} \times 0.05^{0.43} \times 5.23 = 11 \text{ inches} = 0.9 \text{ feet}$
2. Base Stone Analysis 2:
 - a. Equation; $D_{50} = 12(1.923qS^{1.5})^{0.529}$ (eq. TS14C-16, ARS Rock Chute)
 - b. Assumptions; same as above
 - c. Calculation; $D_{50} = 12(1.923 \times 37.9 \times 0.05^{1.5})^{0.529} = 10.8 \text{ inches} = 0.9 \text{ feet}$
3. Base Stone Analysis 3:
 - a. Equation; $D_{30} = 1.95S^{0.555}(Cq)^{2/3} / g^{1/3}$ (eq. TS14C-12, USACE Steep Slope Design)
 - b. Assumptions; same as above
 - i. $C=1.25$
 - c. Calculation; $D_{30} = 1.95 \times (0.05)^{0.555} (1.25 \times 37.9)^{2/3} / 32.2^{1/3} = 1.5 \text{ feet}$
4. Boulder Size Analysis:
 - a. Equation; $D_{\text{min}} = 18(\text{depth})S_f / (SG-1)$ (eq. TS14C-13, USACE Boulder Habitat Design)
 - b. Assumptions;
 - i. Depth = 3.5 feet, maximum depth at low flow notch
 - ii. $S_f = 0.05$, friction slope equals bed slope
 - iii. $SG = 2.6$, Specific gravity of 2 ton boulders
 - c. Calculation; $D_{\text{min}} = 18(3.5\text{ft})0.05 / (2.6-1) = 2.0 \text{ feet}$
5. Plunge Pool Analysis: (USDA 210-VI-DN-6, 1986)
 - a. Assumptions;
 - i. $Q_{\text{design}} = 872$ cfs
 - ii. $S=0.05$ ft/ft
 - iii. $D_{50} = 2.1$ feet
 - iv. $Z_p = 3$ feet, max drop due to boulder height
 - v. $D = 10.2$ feet, In velocity calculations (Eqns 1-6), D =normal depth (3.2') isn't appropriate as that would assume flow is through 3.2' diameter pipe where flow is actually from channel bottom width of 23'. Therefore, solve D based on same area with 23' width, which is 10.2 feet. Trapezoidal flow area = $23' \times 3.2' + 3 \times 3.2 = 83.2 \text{ ft}^2$. Converted to pipe, $r = (83.2/\pi)^{0.5} = 5.1'$, $d=10.2$ feet.
 - b. Equations and Analysis:
 - i. $Q/(gD^5)^{1/2} = 872/(32.2 \times 10.2^5)^{1/2} = 0.5$
 - ii. $V_0 = 4Q/\pi D^2 = 4 \times 872 / \pi 10.2^2 = 10.7 \text{ fps}$

- iii. Eqn. 1: $V_h = V_0 \cos(\sin^{-1} S) = 10.7 \cos(\sin^{-1} 0.05) = 10.7$ fps
- iv. Eqn. 2: $V_v = ((V_0 S)^2 + 2g[z_p + D/2 \cos(\sin^{-1} S)])^{1/2} = ((10.7 \times 0.05)^2 + 2 \times 32.2[3 + 10.2/2 \cos(\sin^{-1} 0.05)])^{1/2} = 22.8$ fps
- v. Eqn. 3: $\tan \alpha = V_v/V_h = 22.8/10.7 = 2.1$
- vi. Eqn. 4: $V_p = (V_h^2 + V_v^2)^{1/2} = (10.7^2 + 22.8^2)^{1/2} = 25$ fps
- vii. Eqn. 5: $X_p = V_h/g (V_v - V_0 S) = 10.7/32.2 (22.8 - 10.7 \times 0.05) = 7.4$ feet
- viii. Eqn. 6: $F_d = V_p/(g d_{50}(\beta_s - \beta)) / \beta = 25/(32.2 \times 2.1(2.64 - 1)) / 1 = 2.4$
- ix. $Z_p/D =$ Use normal depth for D (3.2') $3/3.2 = 0.94 < 1$, therefore use equation 7a.
- x. Eqn. 7a: $\underline{Z_m} = 7.5D [1 - e^{-0.6(F_d - 2)}] = 7.5 \times 3.2 [1 - e^{-0.6(2.4 - 2)}] = \underline{4.8}$
- xi. Eqn. 8: $1 + 25 d_{50}/D = 1 + 25 \times 2.1/3.2 = 17.4$,
 - 1. Using normal depth, $Q/(gD^5)^{1/2} = 872/(32.2 \times 3.2^5)^{1/2} = 8.4$
 - 2. Both cases satisfy, so go to step 9.
- xii. Eqn. 9: $\underline{X_m} = [X_p + Z_m/\tan \alpha] 1.15e^{-0.15[Q/(gD^5)^{1/2}]} = [7.4 + 4.8/2.1] 1.15e^{-0.15(8.4)} = 3.2$, round up to next nearest foot, therefore 4 feet.
- xiii. Eqn. 10: $L_e = Z_m [3/2 + 1/3 Q/(gD^5)^{1/2}] = 4.8 [1.5 + 0.33 \times 0.5] = 8$ feet
- xiv. $L_{r2} = 0.2 L_e = 0.2 \times 8 = 1.6$ feet
- xv. $Z_l = L_e - L_{r2}/0.8 Z_m = 8 - 1.7/0.8 \times 4.8 = 1.6$ feet
- xvi. $\underline{L_r} = 0.8 Z_m \times Z_l + L_{r2} = 0.8 \times 4.8 \times 1.6 + 1.6 = \underline{7.7 \text{ feet}}$

Plunge Pool Results:

- 1. Distance to max scour pool from $X_m = 4$ feet
- 2. Scour depth from $0.8 Z_m = 4$ feet
- 3. Pool Length from $L_r = 8$ feet

PROJECT



TONGUE RIVER STABILIZATION PROJECT
TONGUE RIVER WATERSHED PLAN
PEMBINA WATER RESOURCE DISTRICT
in
PEMBINA COUNTY, NORTH DAKOTA

PREPARED BY
U.S. DEPARTMENT OF AGRICULTURE
NATURAL RESOURCES CONSERVATION SERVICE

INDEX OF DRAWINGS	
SHT #	TITLE
1	COVER SHEET
2	QUANTITIES, DIVERSION DETAILS
3	PLAN OVERVIEW
4-11	PLAN & PROFILE CUT SHEETS
12	TYPICAL CROSS SECTION DETAIL
13-16	CROSS SECTIONS
17	TYPE 1 BANK PROTECTION
18	TYPE 2 BANK PROTECTION
19-20	ROCK ARCH RAPIDS DETAILS
21	ROCK CROSS VANE DETAILS
22	COBBLE PATCH & DEBRIS COLLECTOR
23-24	REVEGETATION & STAGING DETAILS
25	ROCK SILL DETAILS

NRCS JOB APPROVAL AUTHORITIES		
CODE	CONSERVATION PRACTICE	JC
582	OPEN CHANNEL	IV
410	GRADE STABILIZATION STRUCTURE	V
391	RIPARIAN FOREST BUFFER	III
390	RIPARIAN HERBACEOUS COVER	IV
512	PASTURE AND HAYLAND PLANTING	V
342	CRITICAL AREA SEEDING	IV
484	MULCHING	IV



PROJECT LOCATION MAP
(NOT TO SCALE)
SECTION 28 & 29, T161N R56W

Date

Designed

Drawn

Checked

Approved

PRELIMINARY

NOT FOR CONSTRUCTION

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
COVER SHEET
PEMBINA COUNTY WATER RESOURCE DISTRICT, NORTH DAKOTA



File Name

Tongue_Details.dwg

Date

July, 2021

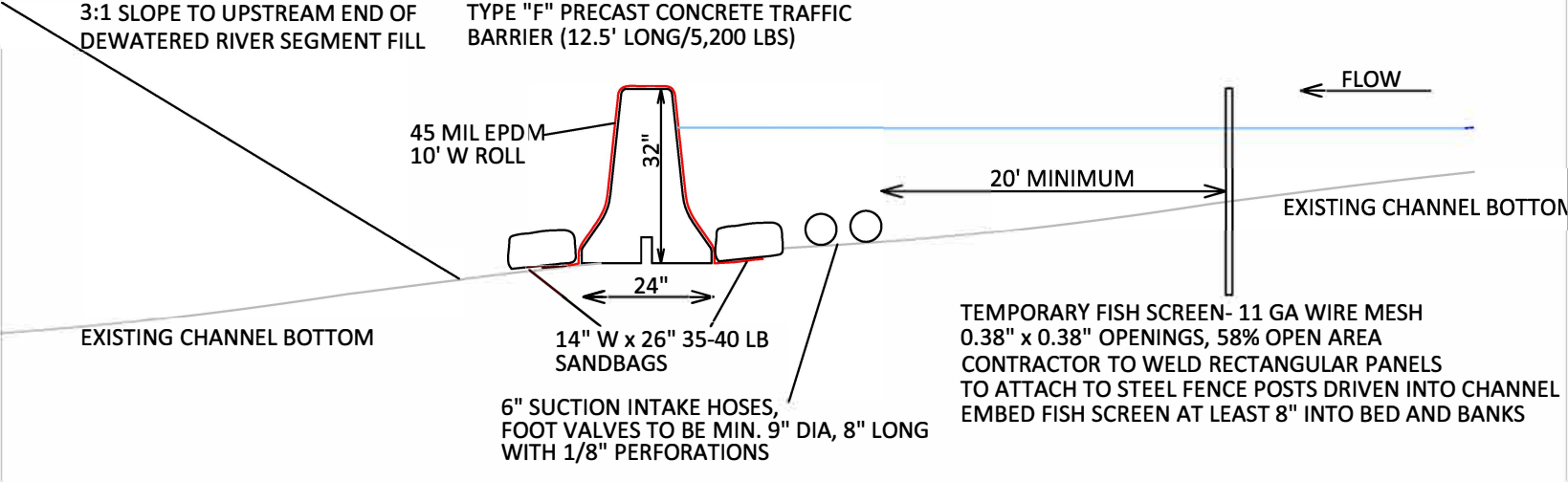
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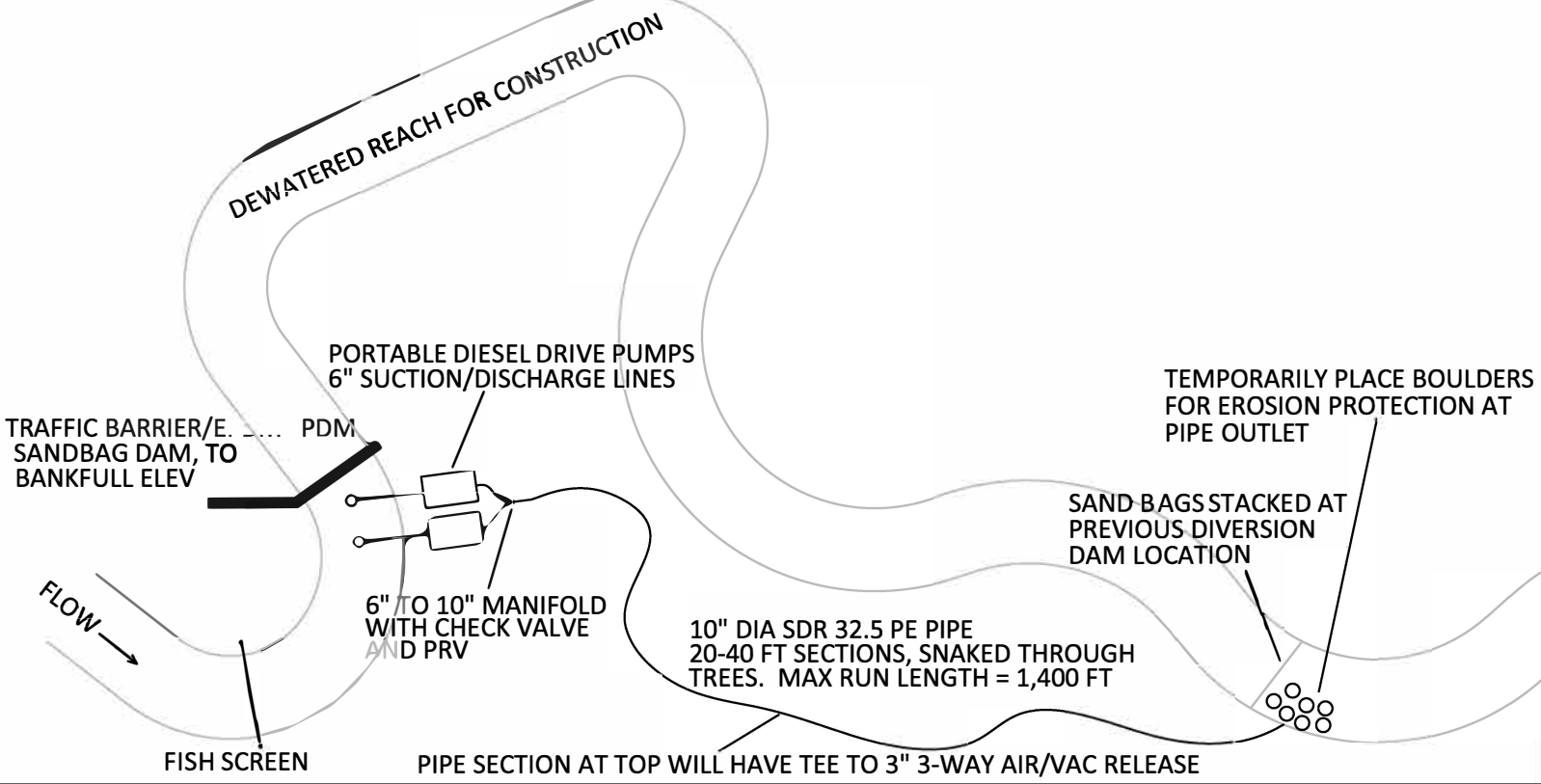
PROJECT QUANTITIES

ITEM	QUANTITY
CLEAR LEVEES & TEMPORARY ACCESS ROUTES.	2 AC
PZ27 A572 GRADE 50 SHEETPILE	4,856 SF
TOPSOIL STRIPPING	5,857 CY
LEVEE EXCAVATION	3,138 CY
FLOODPLAIN EXCAVATION	69,978 CY
TEMPORARY DIVERSION SETUPS	8
10" DIA SDR 32.5 PE PIPE, 20-40 FT SECTIONS	1,400 FT
10" DIA BOLTED HDPE COUPLERS, FABRICATED BENDS	as needed
6" HOSE, 6-10" MANIFOLD, 10" CHECK, 3" CAV, PRV	as needed
6" PORTABLE PUMP, MIN Q=2.4 CFS @ 55 FT TDH	2
TYPE "F" PRECAST TRAFFIC BARRIERS, 2.7'Hx12.5'L	as needed
SANDBAGS, 14" x 26" (35-40 LBS)	as needed
45 MIL EPDM, 10' WIDE ROLL	as needed
FISH SCREEN W/STEEL POSTS FOR SUPPORT	as needed
CHANNEL RESTORATION EARTHFILL PLACEMENT	67,927 CY
CHANNEL RESTORATION GRAVEL/COBBLE PLACEMENT	14,185 CY
IMPORTED CUSTOM SCREENED GRAVEL MIX	14,415 TONS
IMPORTED COBBLE (RAILROAD MIX)	6,678 TONS
IMPORTED BOULDERS	1,116 TONS
IMPORTED ND DOT GRADE 2 RIPRAP	4,002 TONS
TYPE 1 BANK PROTECTION	6,960 FT
TYPE 2 BANK PROTECTION	9,310 FT
700 GM/SQM COIR FABRIC - 13.1' x 164' ROLLS	144
EROSION CONTROL BLANKET - 8' x 112' ROLLS	249
SILT FENCE	19,800 FT
14"+ DIA LOGS W/ROOTWADS, AVG LENGTH 18 FT	48
10"+ DIA LOGS, AVG LENGTH 15 FT	44
LIVE WILLOW/DOGWOOD CLUMPS	30
LIVE WILLOW/DOGWOOD CUTTINGS	55,680
PRAIRIE CORDGRASS PLUGS	500
BARE ROOT TREES	5,770
TREE PROTECTORS	2,885
CONSERVATION GRADE SHRUBS	2,885
GRASS SEEDING (DRILL)	54.8 AC
HYDROSEEDING/MULCHING	44.1 AC

TEMPORARY DIVERSION DETAILS- PROFILE VIEW
(NOT TO SCALE)



TEMPORARY DIVERSION DETAILS- PLAN VIEW
(NOT TO SCALE)



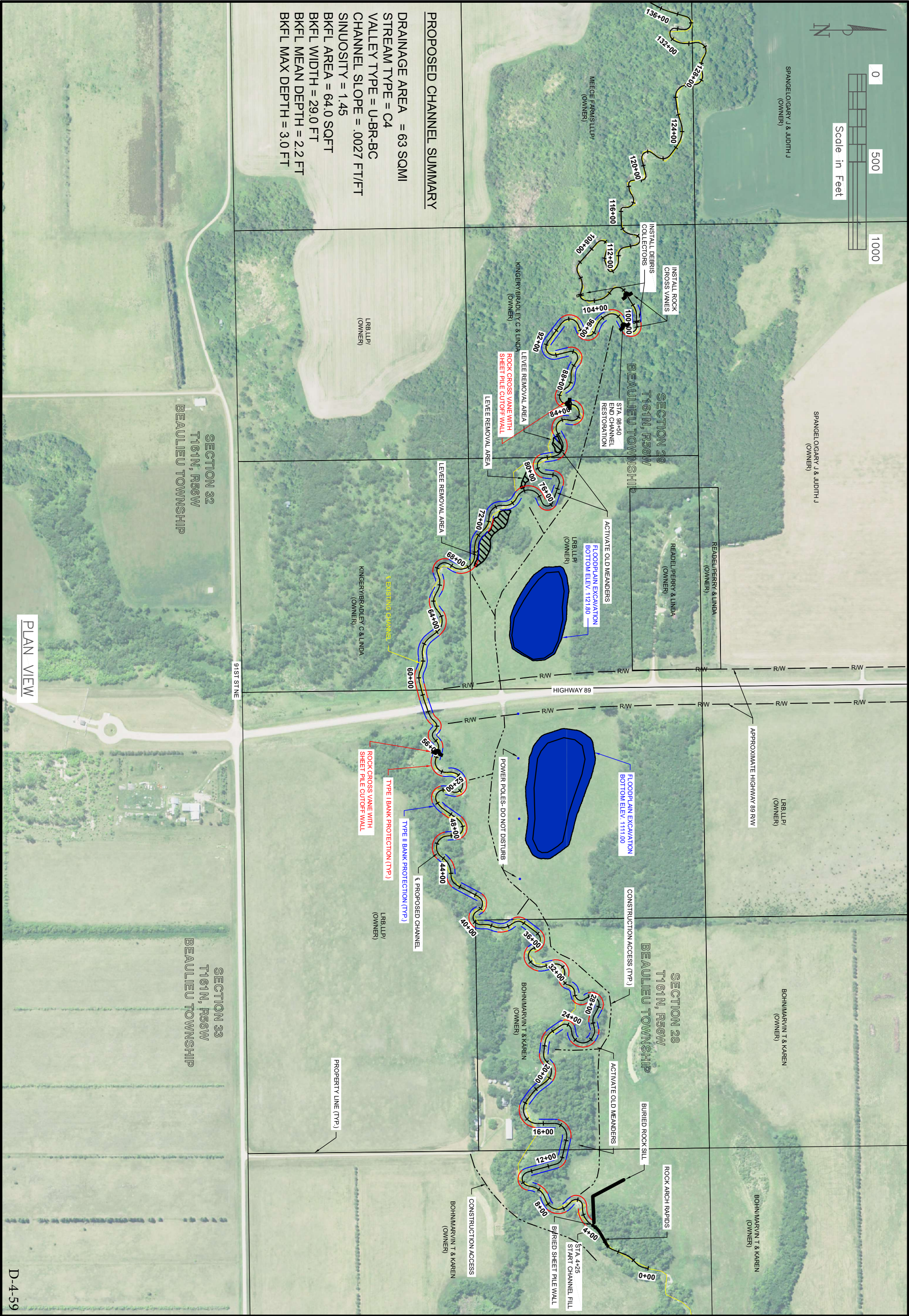
PUMP NOTES: Each of the portable pumps will have a minimum capacity of 1080 gpm at a TDH of 85 ft, at 15 ft of suction lift. Each will have float activated automatic start/stop controls and vacuum pump automatic priming sytems. Double walled auxillary fuel tank will ensure 5 days of fuel for both pumps is on site at all times. Start elevation for high stage pump will be set approximately 6 inches below the top of the diversion dam. Shutoff elevation for low stage pump will be set at 6" water depth from existing channel bottom.

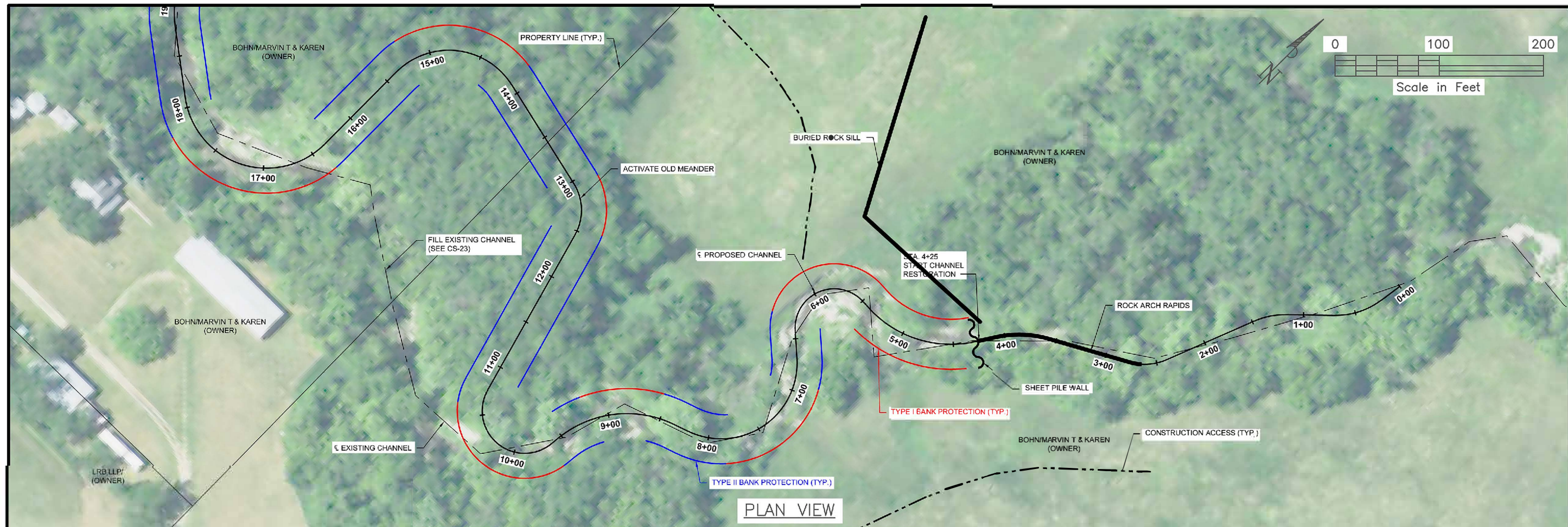
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TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
QUANTITIES, DIVERSION DETAILS
PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

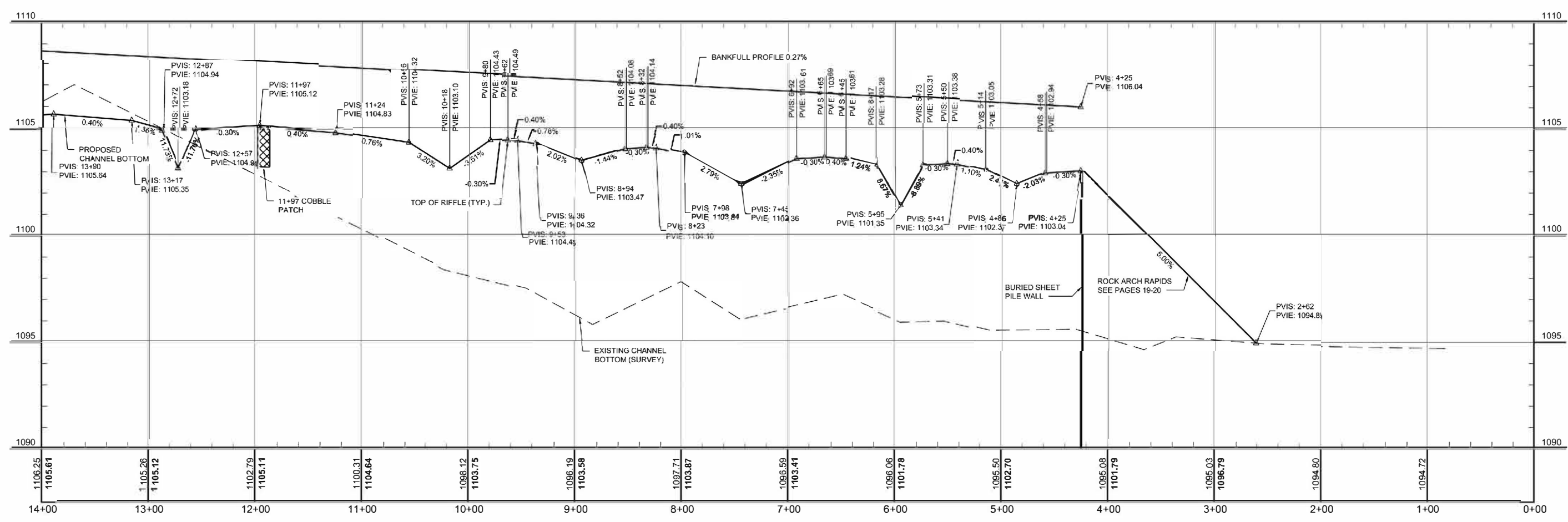


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July, 2021





PLAN VIEW



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TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

PLAN AND PROFILE 0+00 TO 14+00

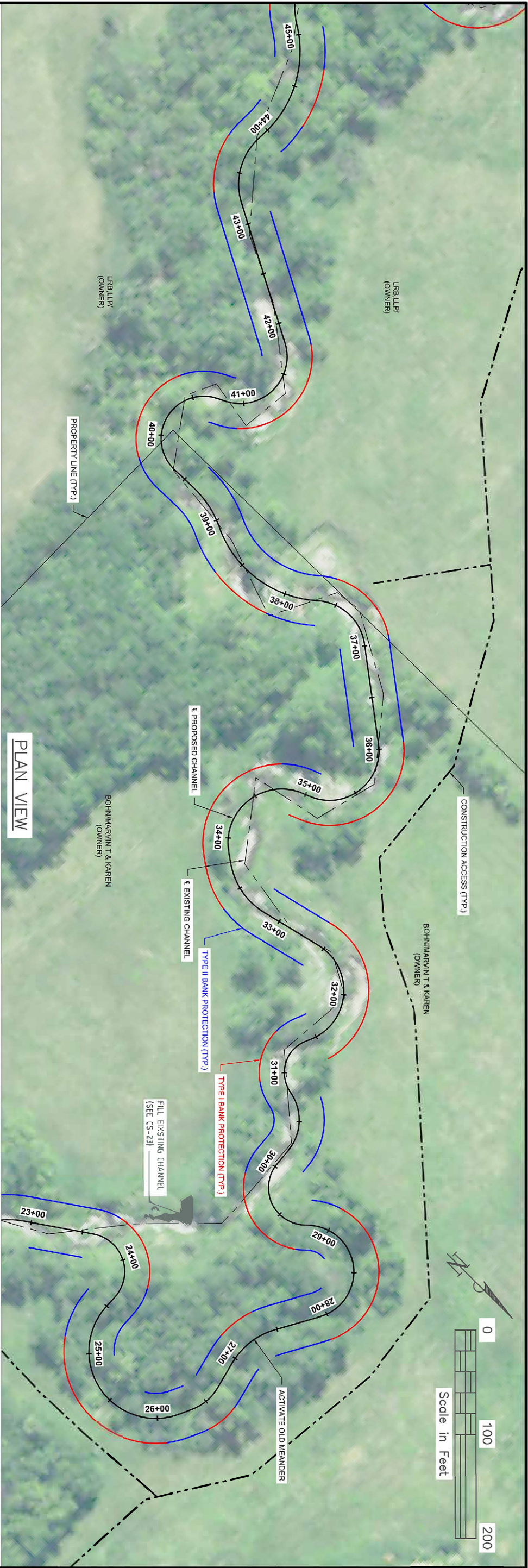
PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



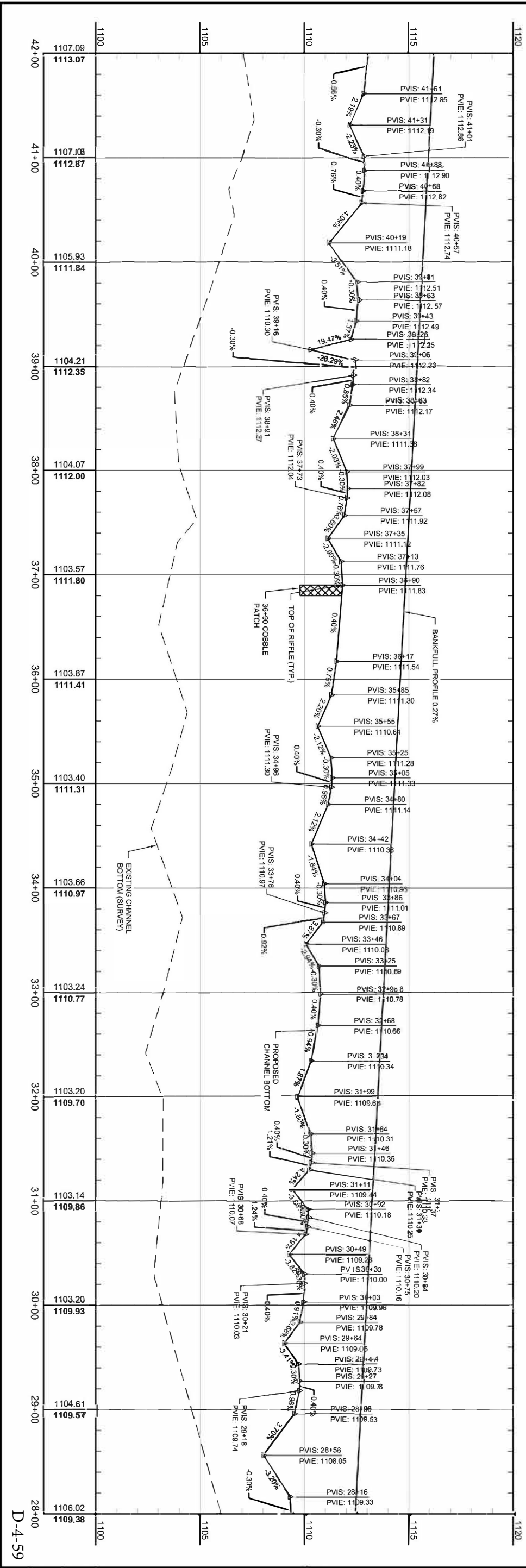
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Date
July, 2021

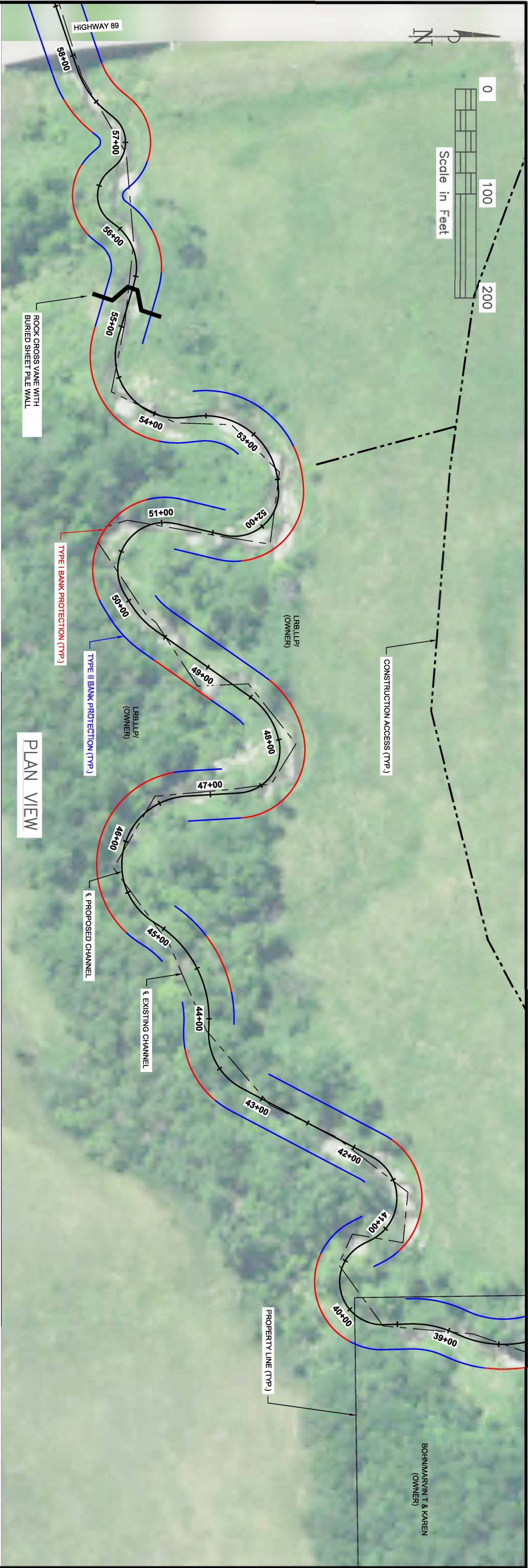
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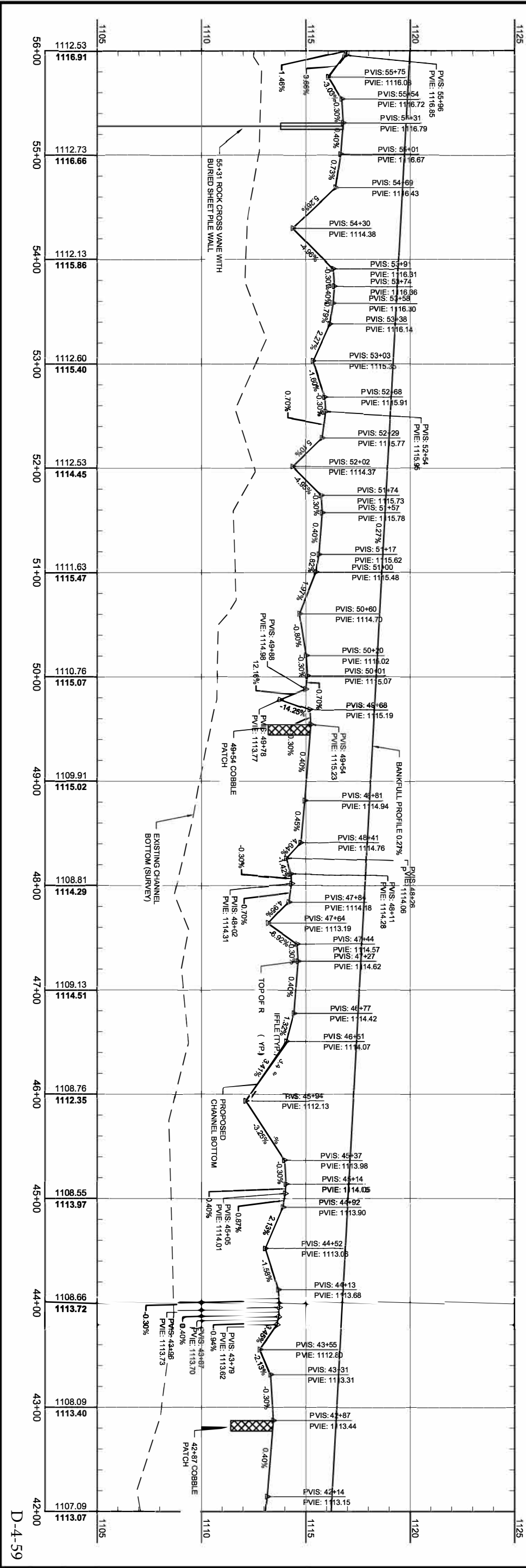
PLAN VIEW



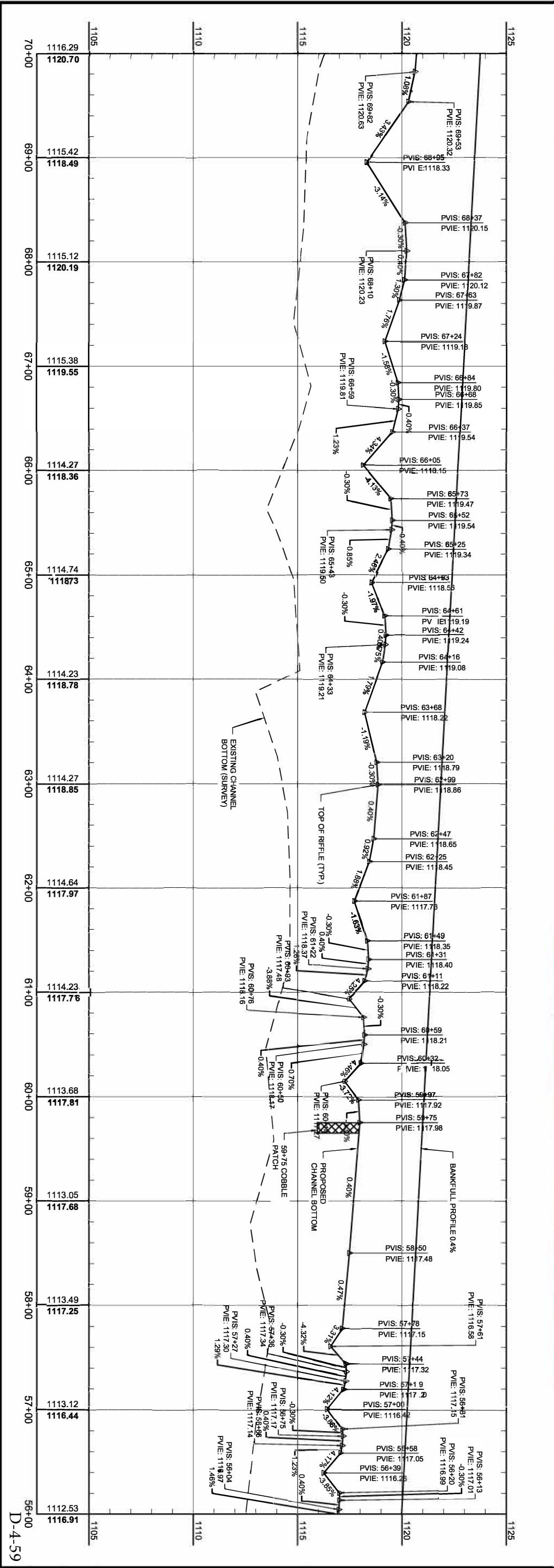
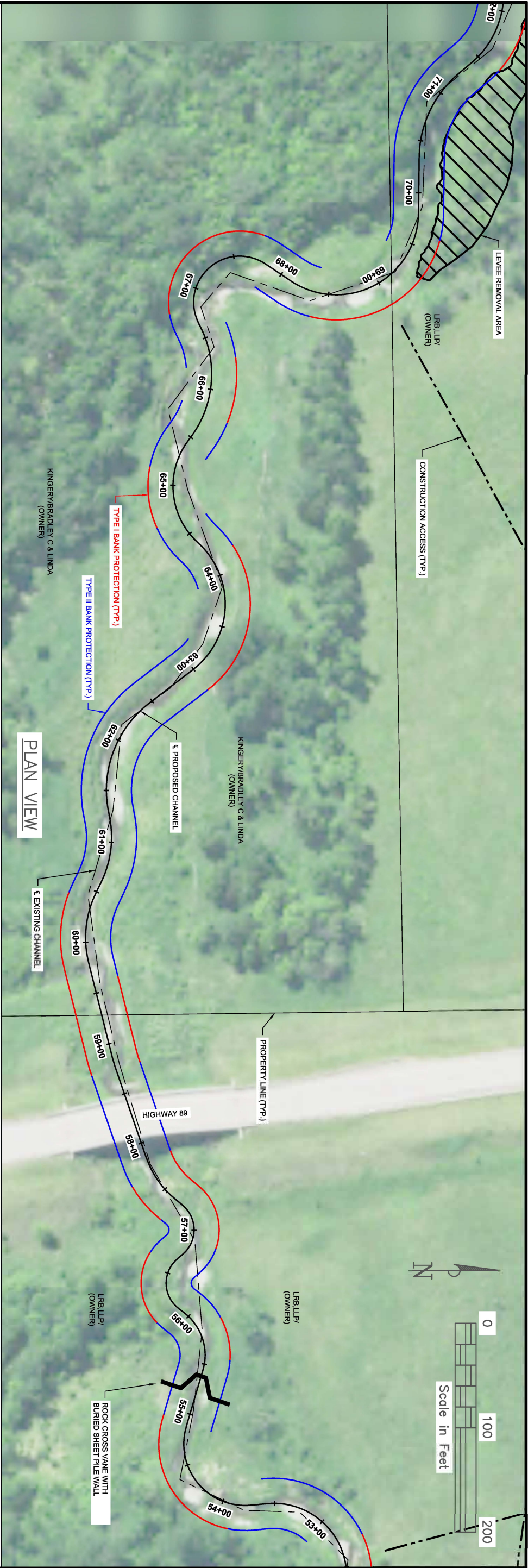
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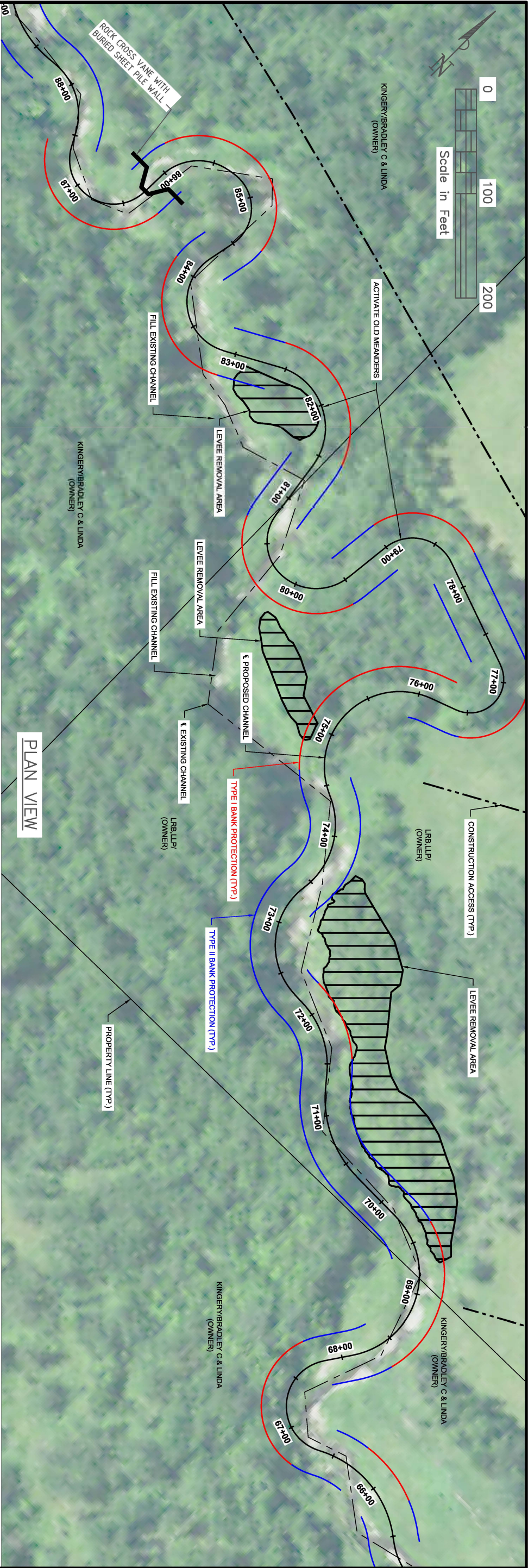


PLAN VIEW

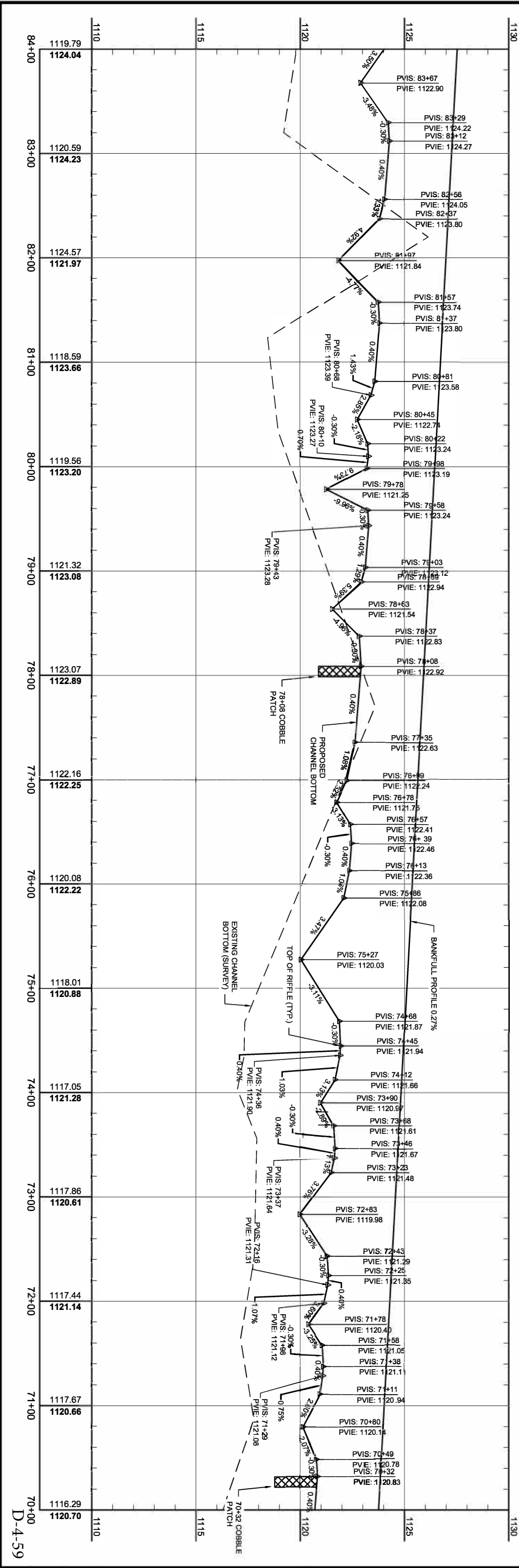


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PLAN VIEW



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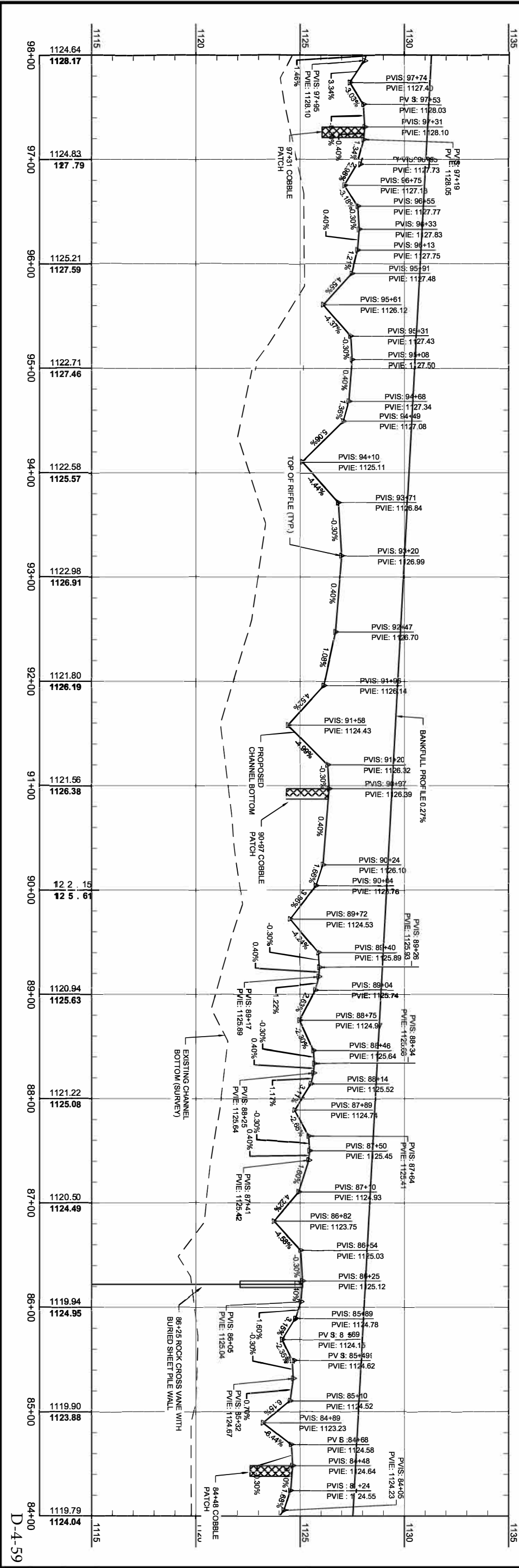
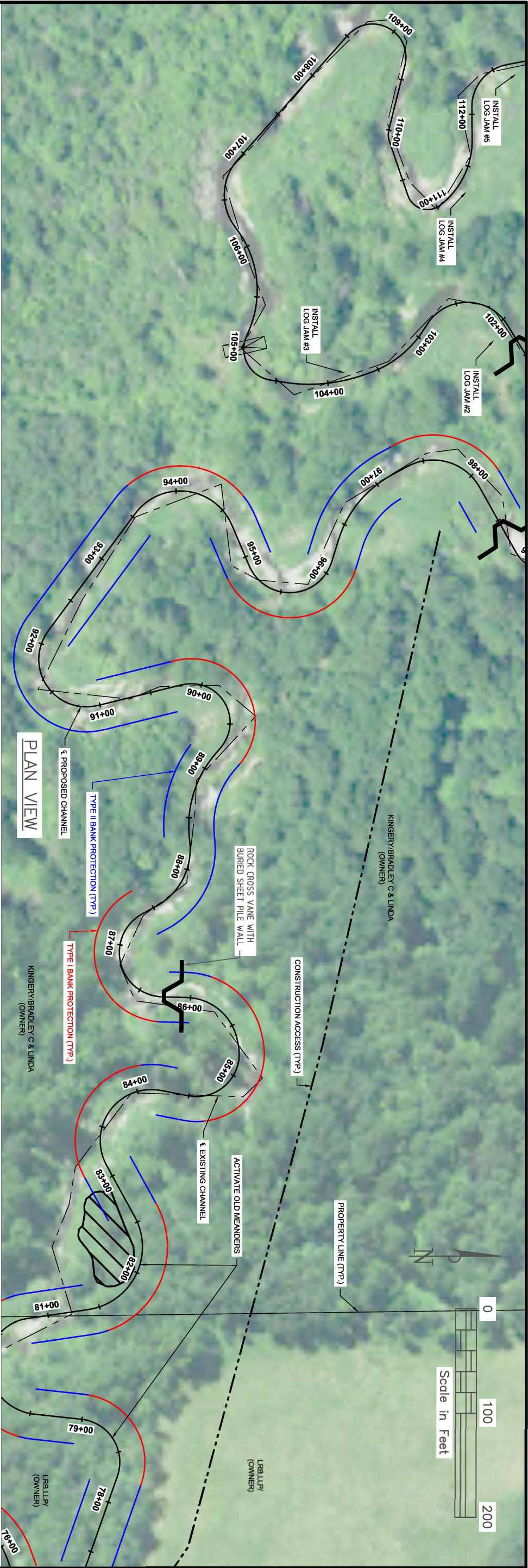


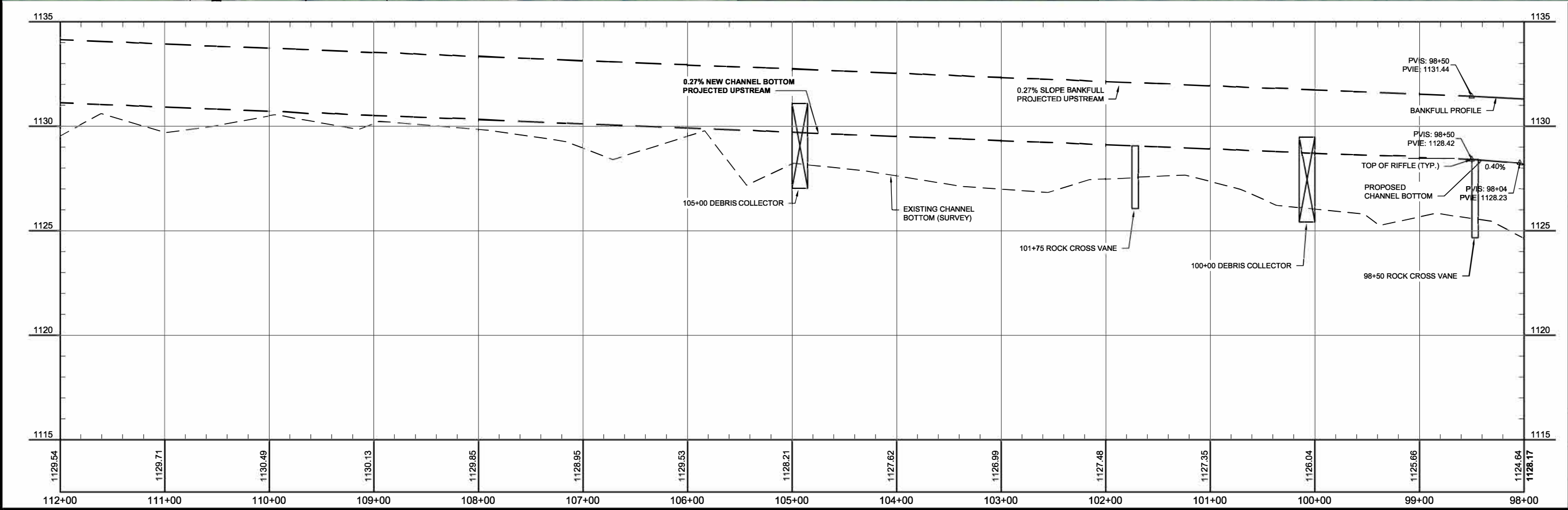
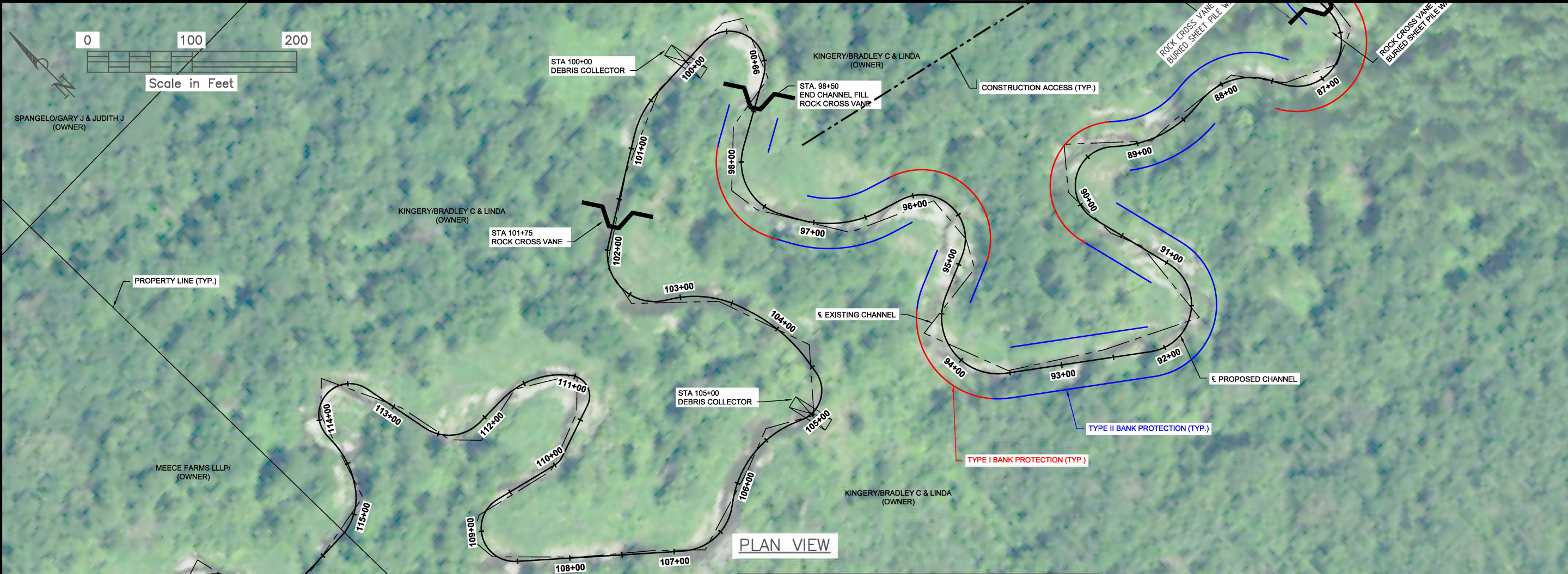
TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
PLAN AND PROFILE 70+00 TO 84+00
PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

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9 of 25





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TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
PLAN AND PROFILE 98+00 TO UPSTREAM OF PROJECT

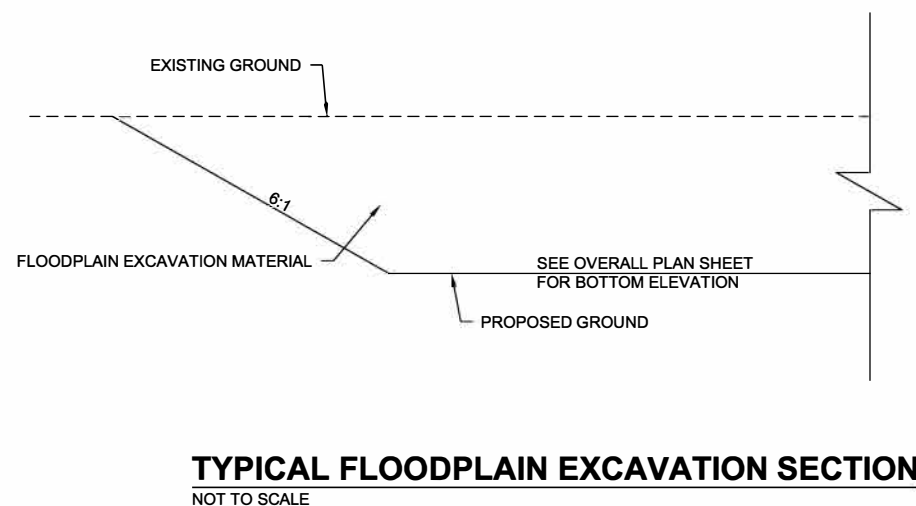
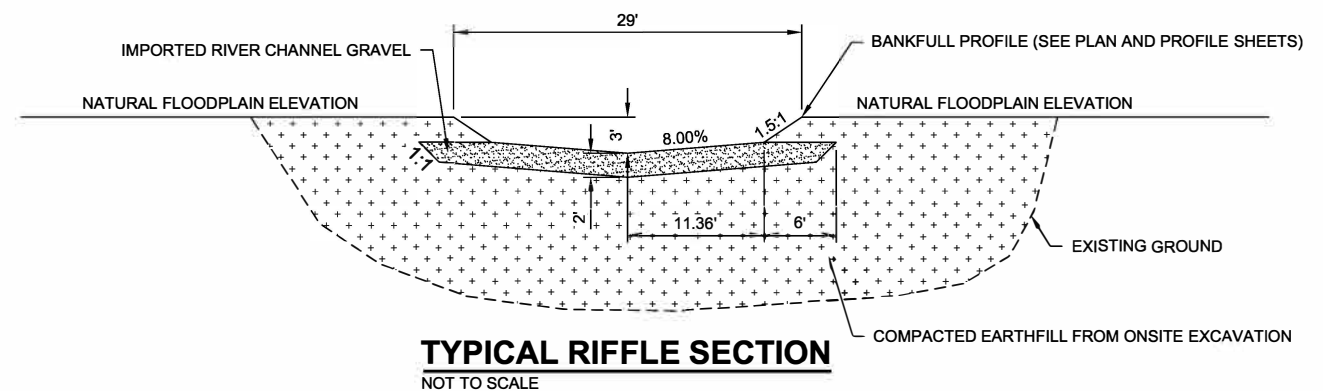
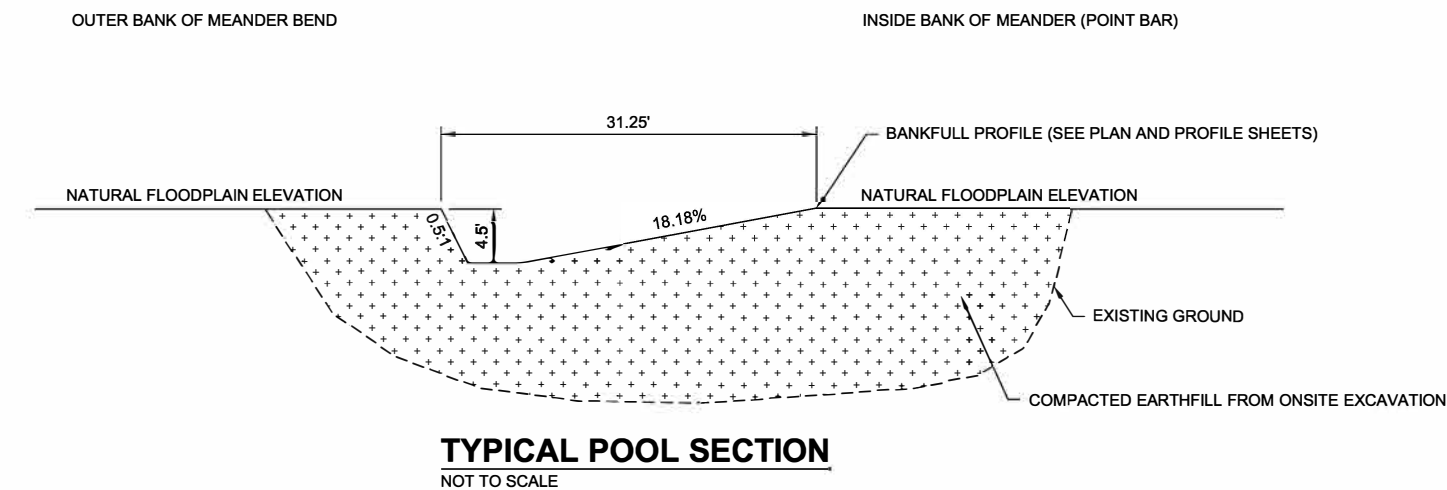
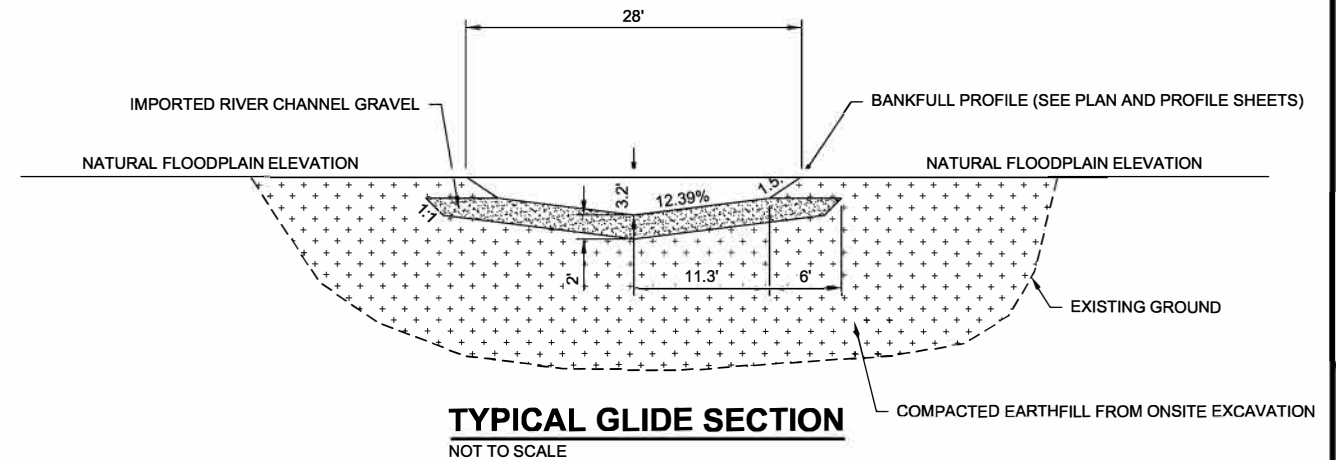
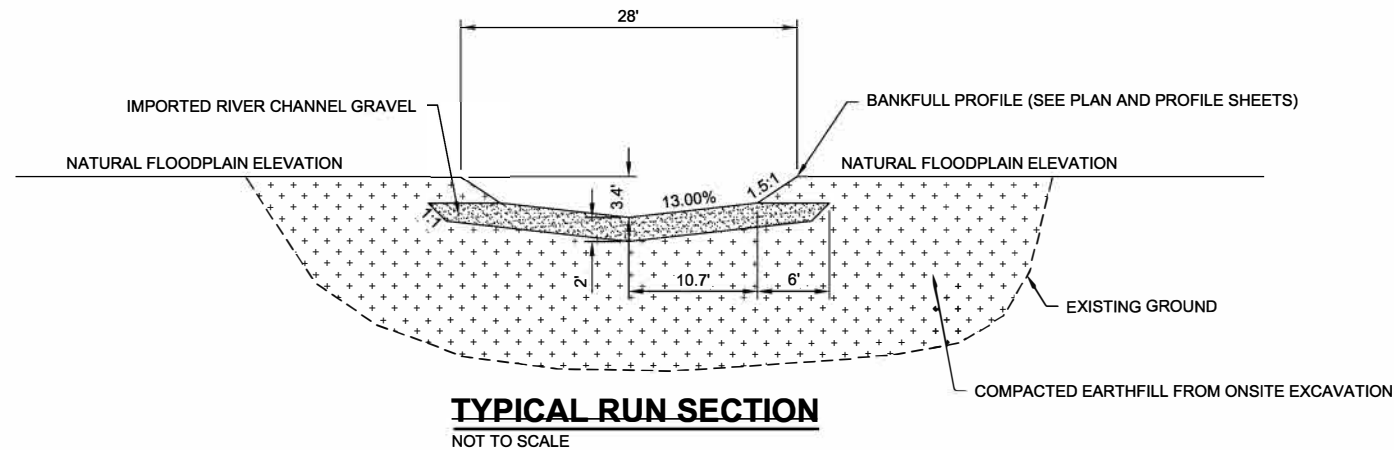
PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

NRCS
Natural Resources Conservation Service

File Name
Alt 2 2021-06-03.dwg

Date
July, 2021

11 of 25



IMPORTED CHANNEL GRAVEL GRADATION

SIEVE SIZE	% PASSING
3"	100%
2"	70-85%
1-1/2"	60-65%
1"	48-63%
3/4"	43-48%
3/8"	33-38%
#4	20-25%
#8	13-18%
#10	4-9%
#30	<3%

Notes

1) Contractor will be required to utilize hydraulic excavator(s) equipped with a 3D grade control system, capable of being submerged (Trimble GCS900/AS456 or similar). NRCS will provide a DEM for channel elevations at the top of the earthfill and another for finished ground elevations, at a 1ft by 1ft cell size grid, to guide channel construction.

2) Compacted earthfill will be from onsite floodplain excavations; excavations are sized to ensure there will be no need for material to be hauled across Hwy 89. Test holes across the floodplain indicate uniform conditions of 3-6" topsoil, with degraded shale material below. Shale has soil mechanics characteristics similar to a CH material.

2) Shale material will be placed and compacted in 6" maximum depth loose lifts into the channel bottom, to 95% proctor, at the elevations shown on sheets 4-11. Moisture may need to be added via overnight watering of the excavation site, prior to the next days fill placement. Note that while riffle cross sections are constant, some variation in the depth of pools, runs, and glides is built into the design.

3) Gravel for channel fill will need to be specially screened for the project at a local gravel pit. Material will be stockpiled and handled with care to ensure fines are not added to the mix during onsite operations. Gravel will be placed in 8" maximum depth loose lifts, and compacted by at least 3 passes of a fully loaded 10 ton front end loaders over it. Material will be rounded in at least 2 dimensions and meet requirements as outlined in MS-521.

4) Installation of bank protection elements and cobble patches will occur as channel construction proceeds. NRCS inspector will approve each segment of construction prior to water being turned into it.

Date _____
Designed _____
Drawn _____
Checked _____
Approved _____

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

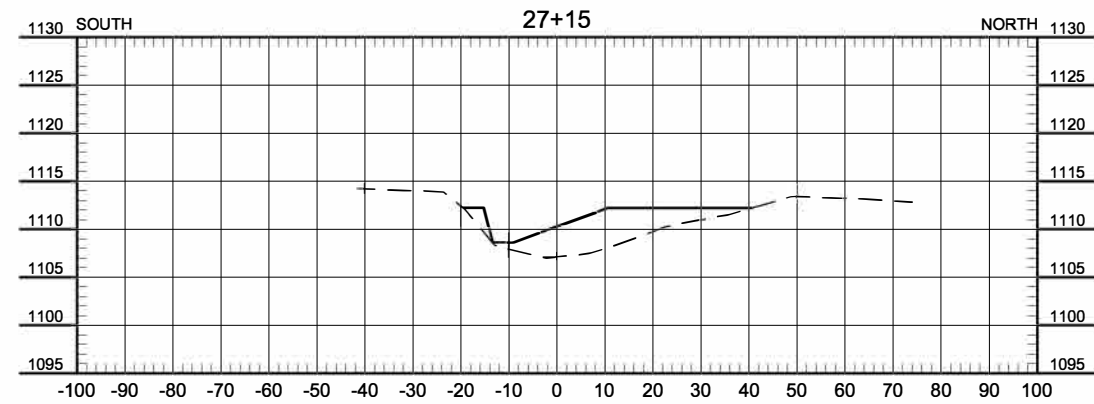
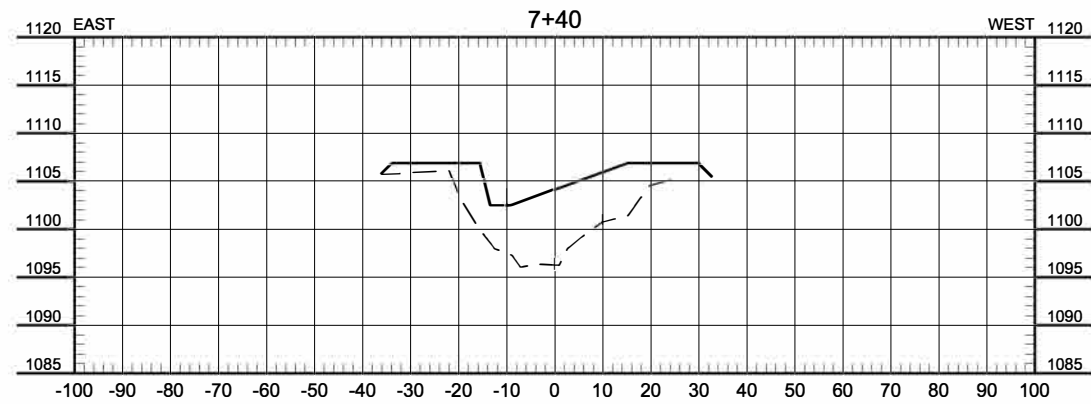
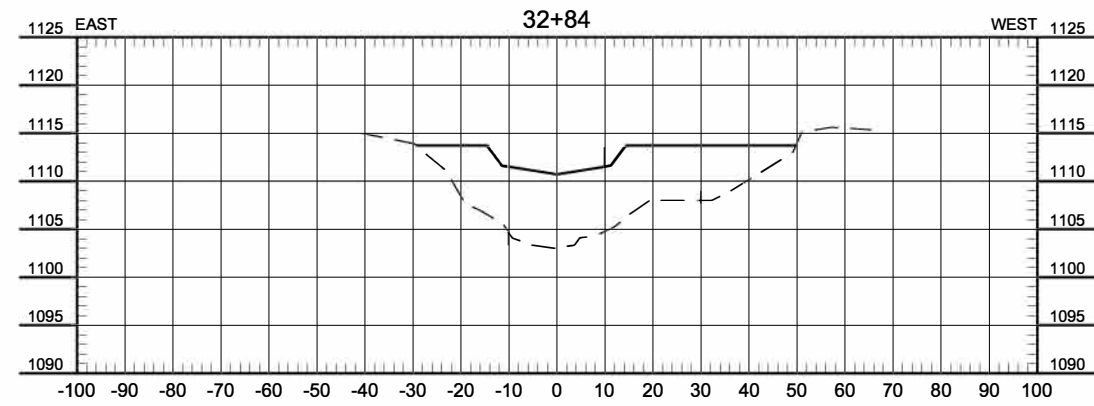
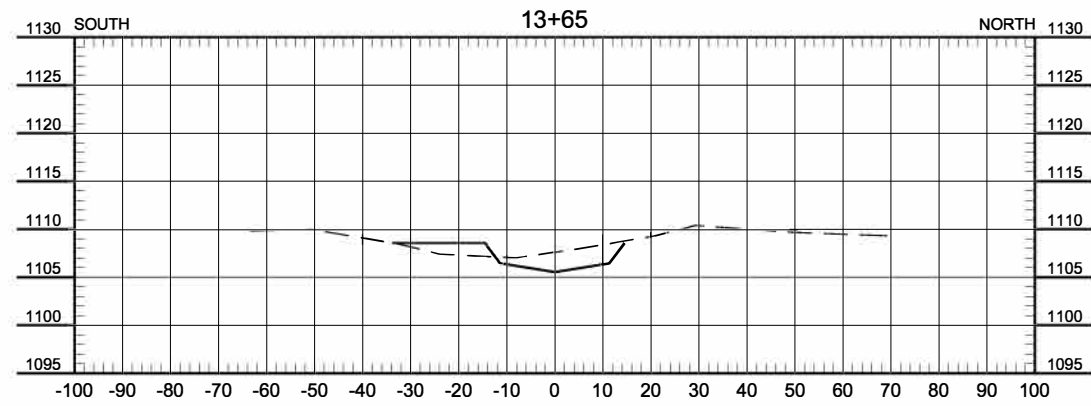
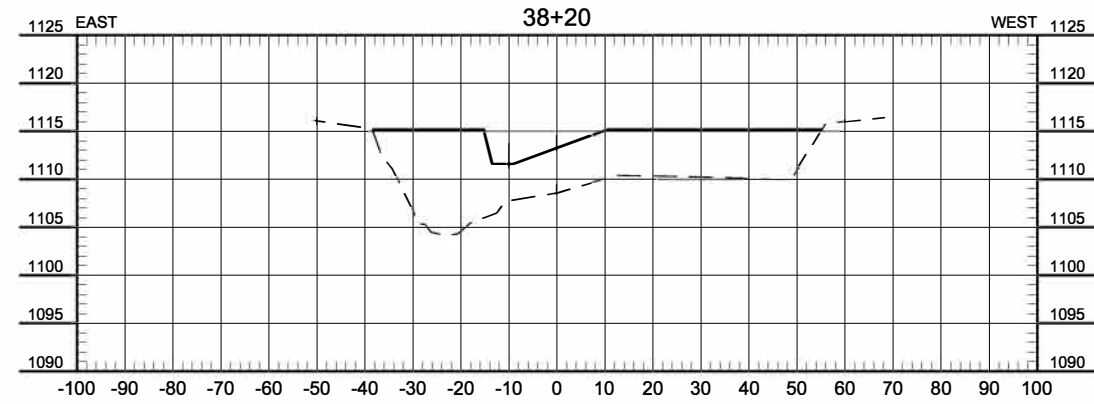
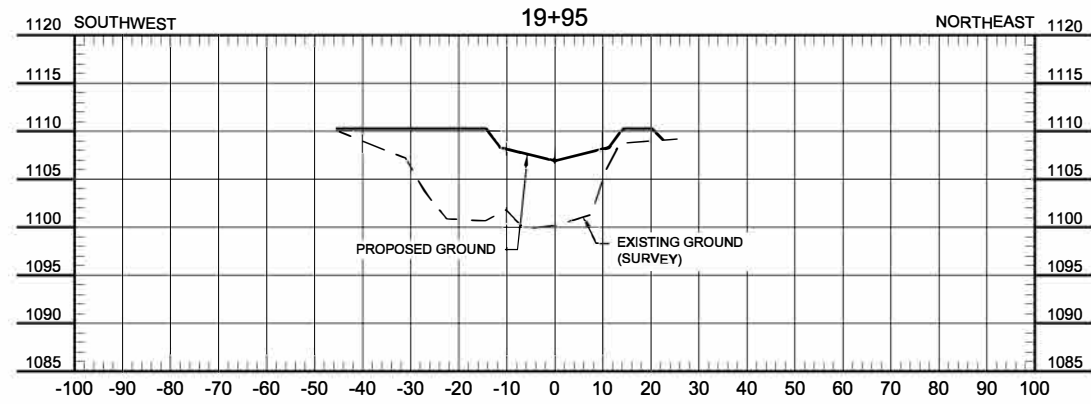
TYPICAL CROSS SECTION DETAIL

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



File Name
Typical Sections Alt2.dwg

Date
July, 2021



Date _____

Designed _____

Drawn _____

Checked _____

Approved _____

PRELIMINARY
NOT FOR CONSTRUCTION

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

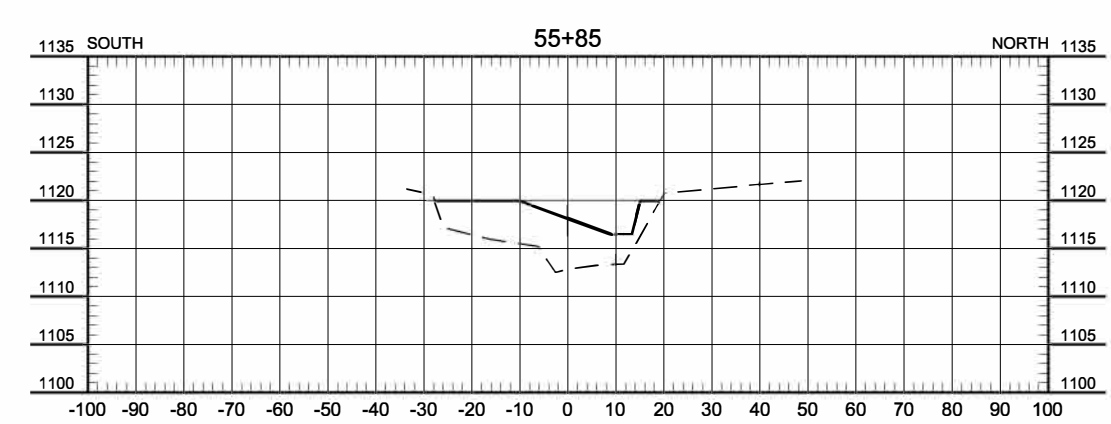
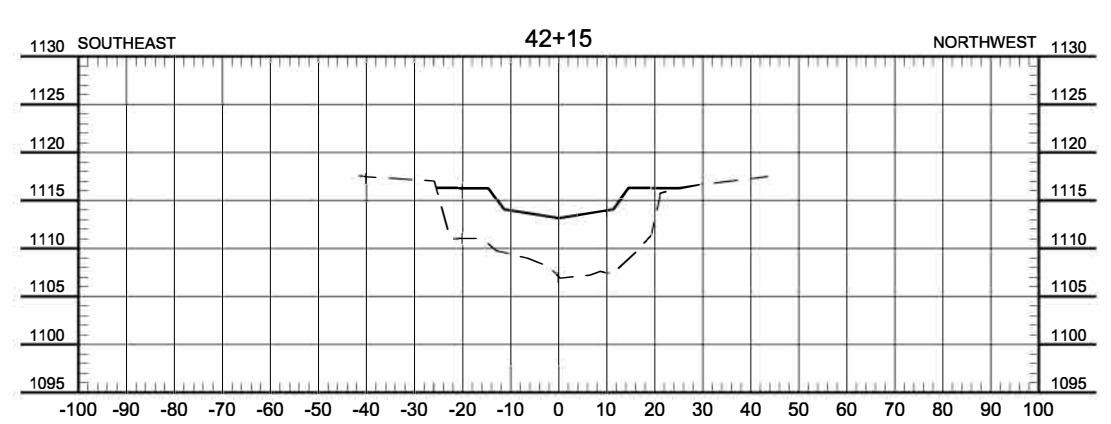
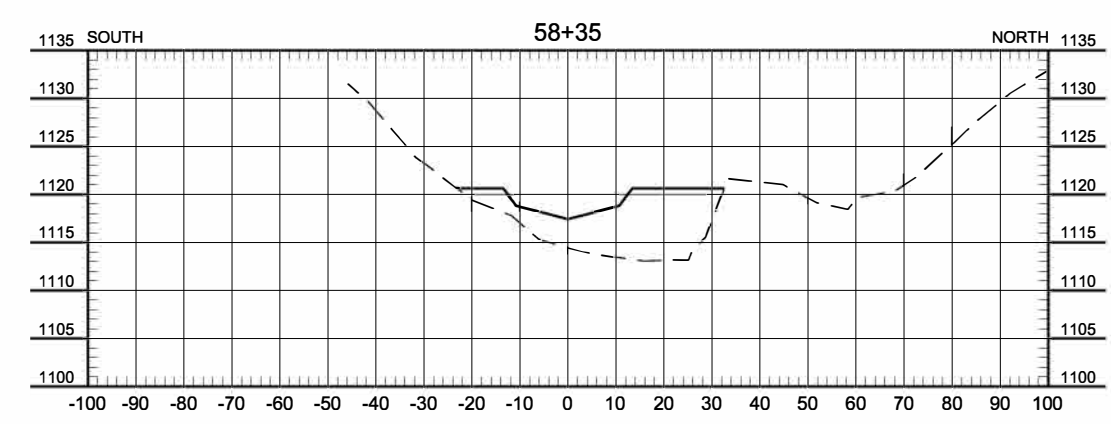
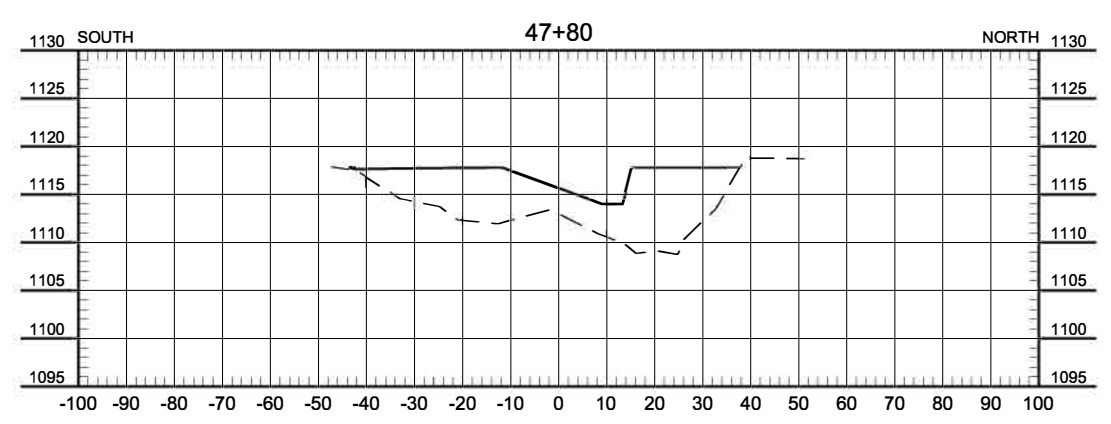
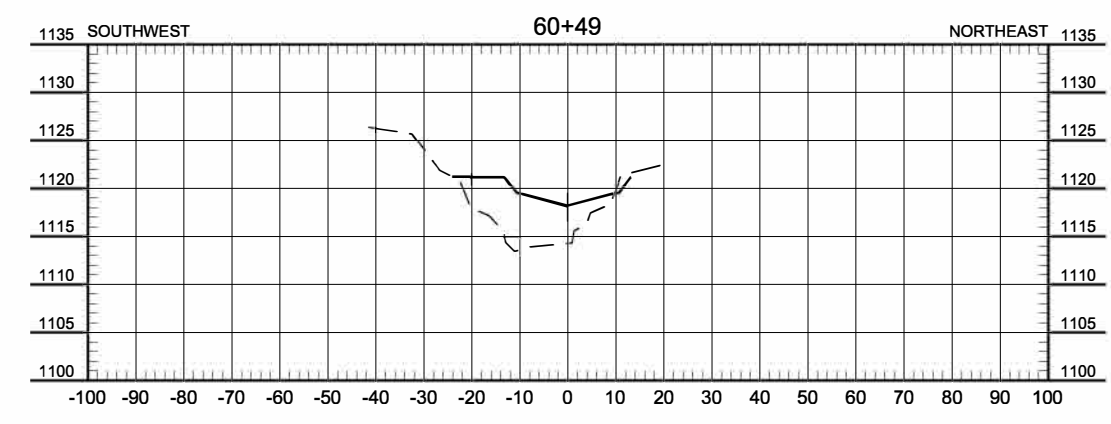
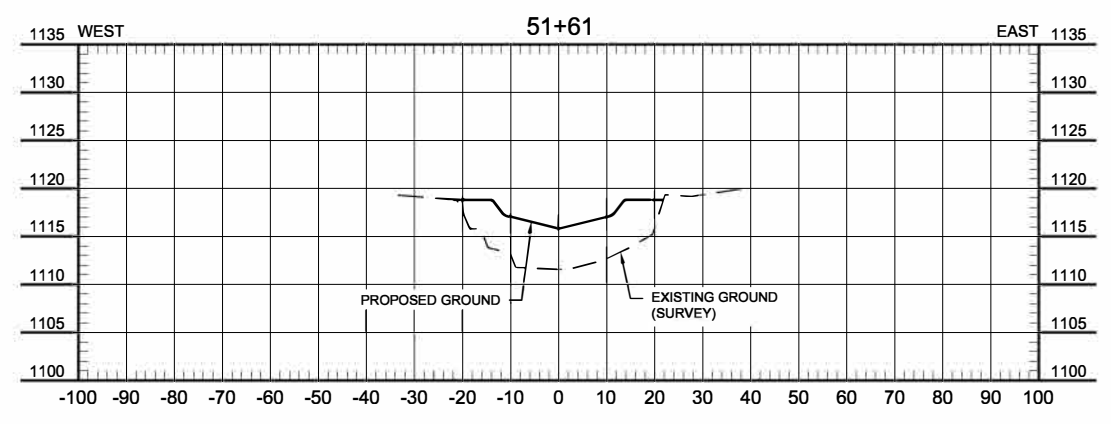
CROSS SECTIONS

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



File Name
Tongue River Cross
Sections.dwg

Date
July, 2021



Date _____

Designed _____

Drawn _____

Checked _____

Approved _____

PRELIMINARY

NOT FOR CONSTRUCTION

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

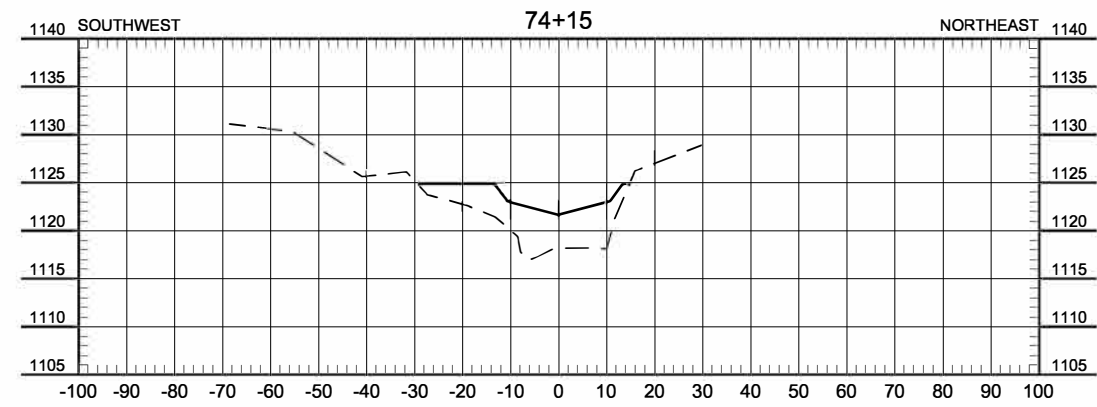
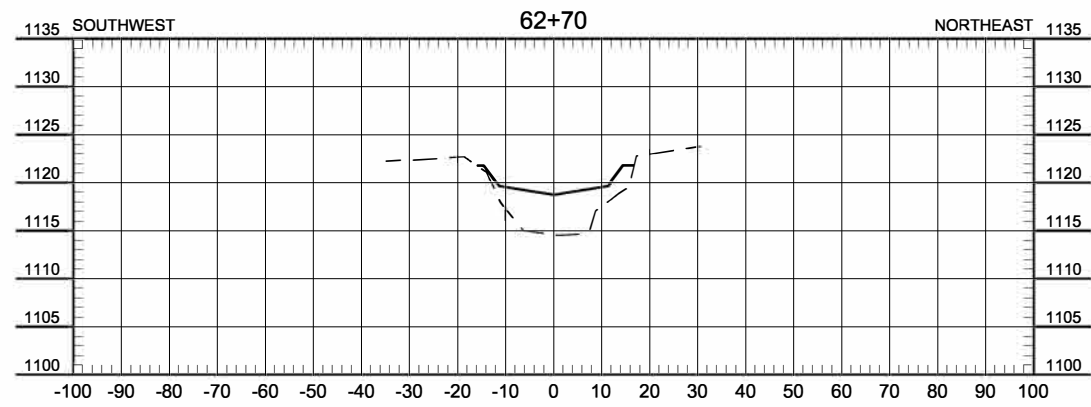
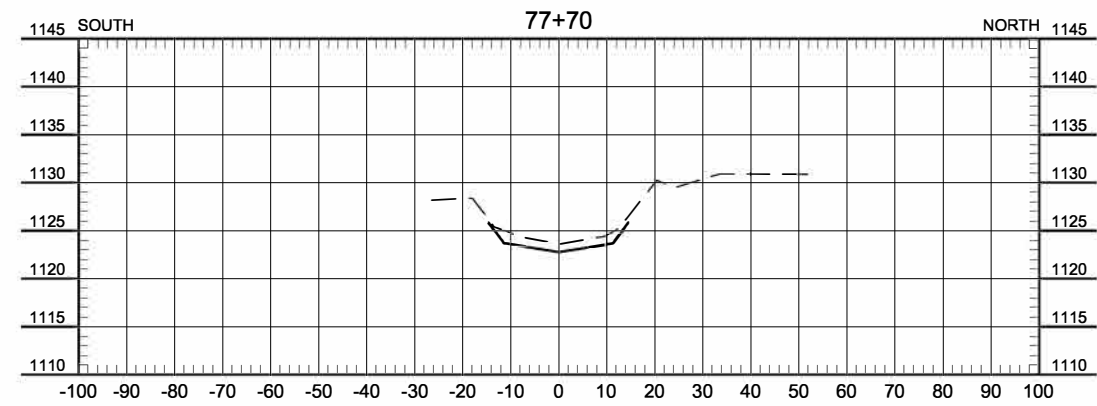
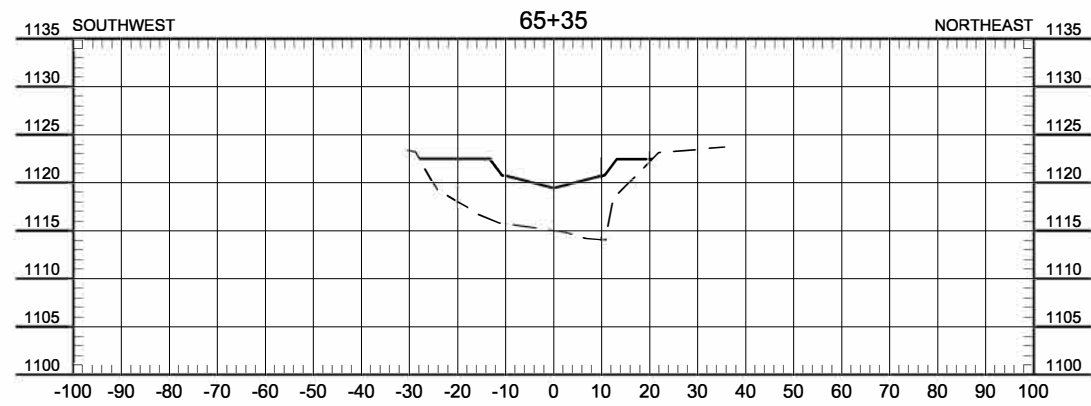
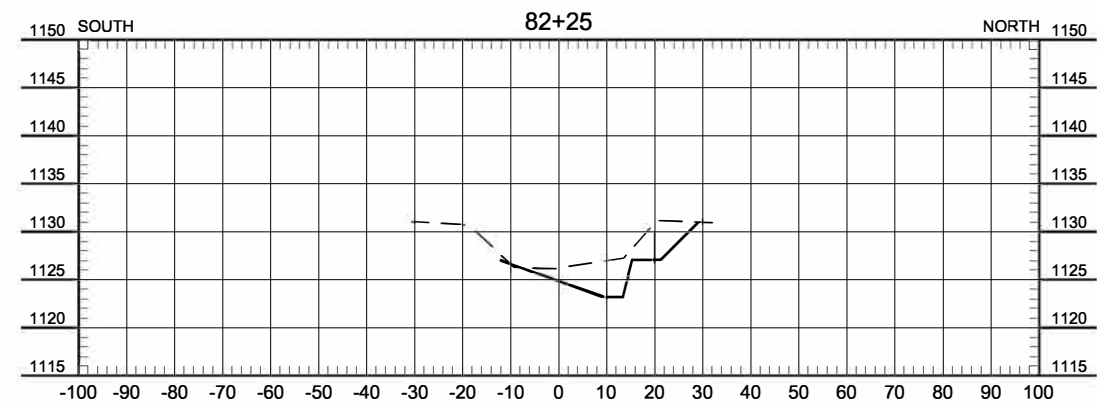
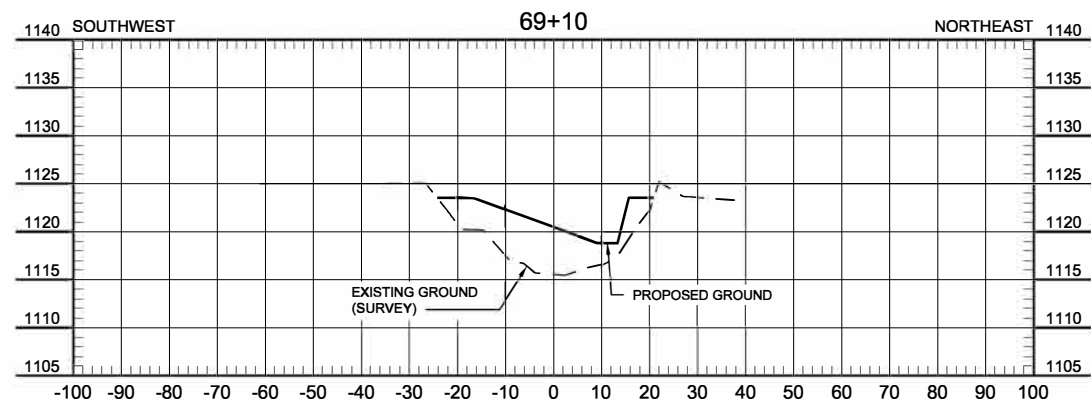
CROSS SECTIONS

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



File Name
Tongue River Cross
Sections.dwg

Date
July, 2021



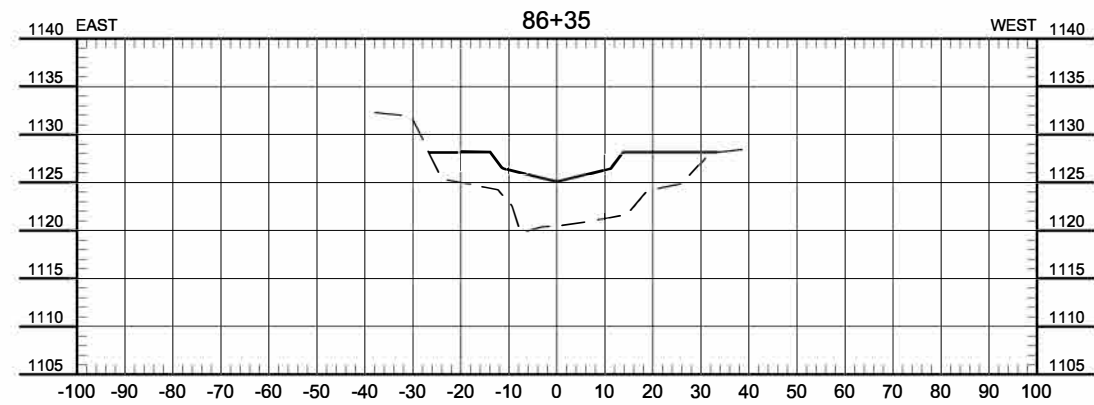
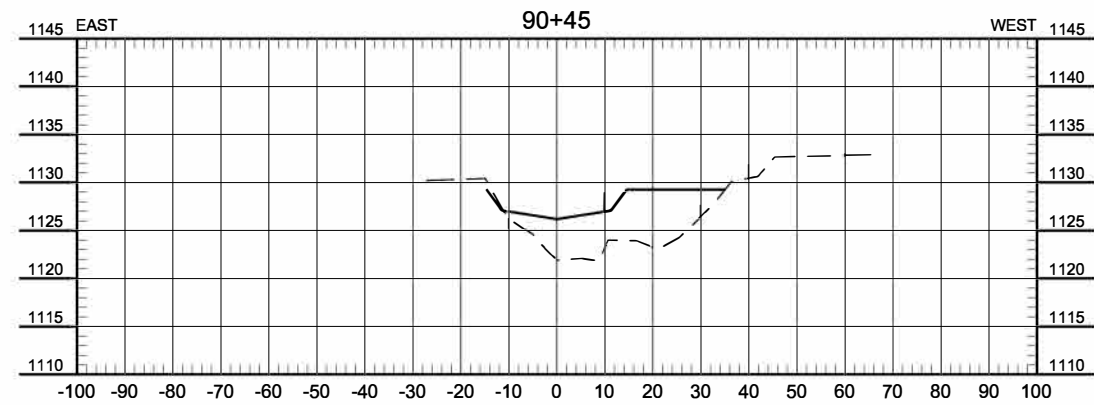
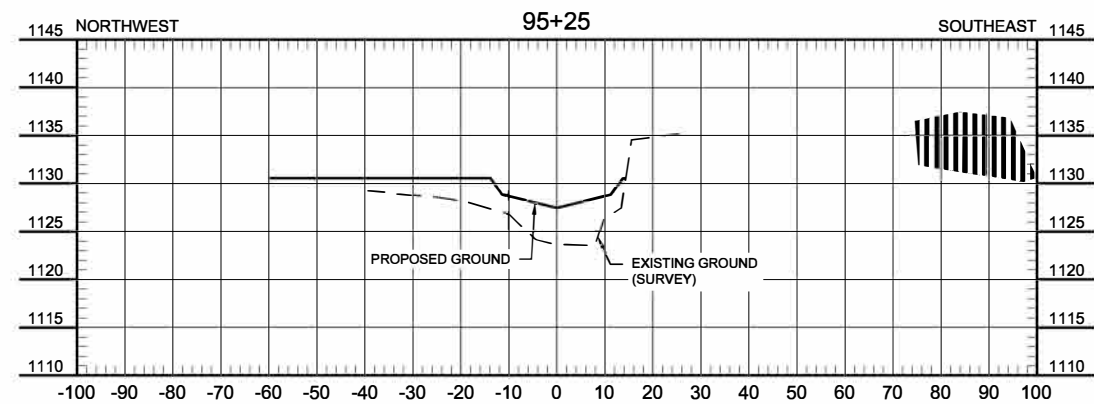
Date _____
 Designed _____
 Drawn _____
 Checked _____
 Approved _____

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
 CROSS SECTIONS
 PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



File Name
 Tongue River Cross
 Sections.dwg

Date
 July, 2021



Date _____

Designed _____

Drawn _____

Checked _____

Approved _____

PRELIMINARY
NOT FOR CONSTRUCTION

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

CROSS SECTIONS

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

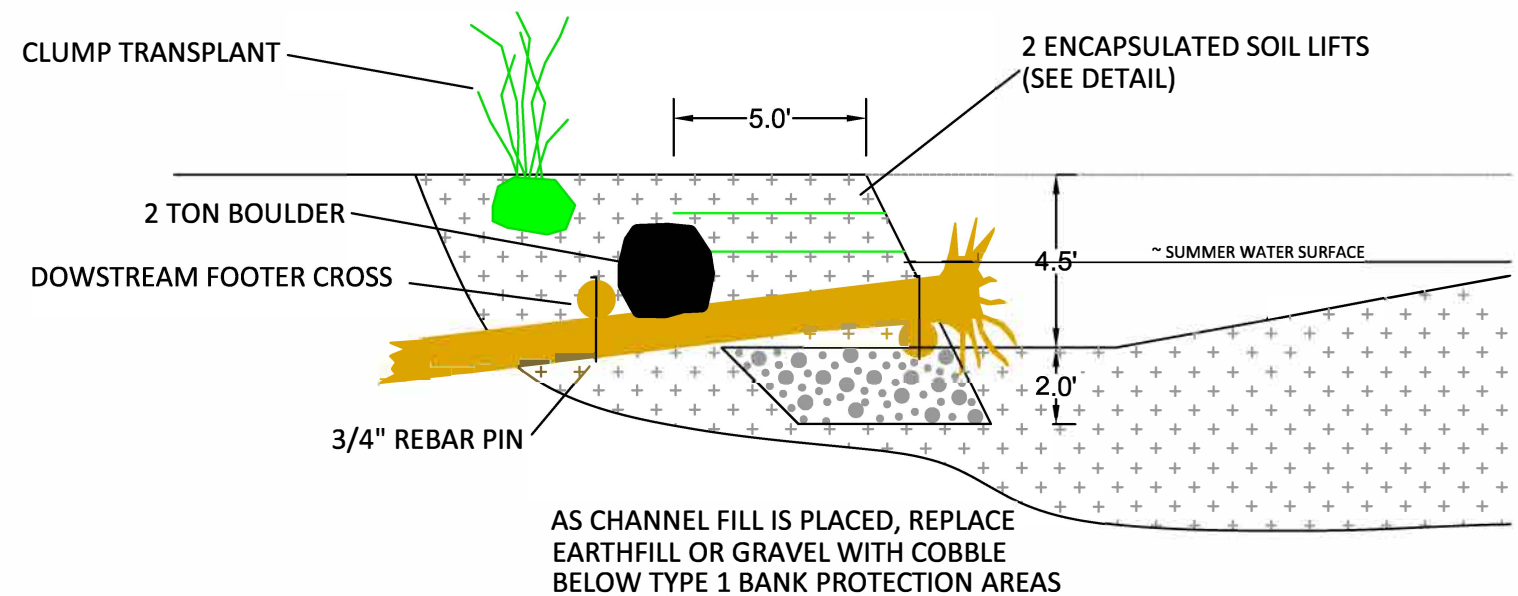


File Name
Tongue River Cross
Sections.dwg

Date
July, 2021

SECTION VIEW- TYPE 1 BANK PROTECTION

NOT TO SCALE

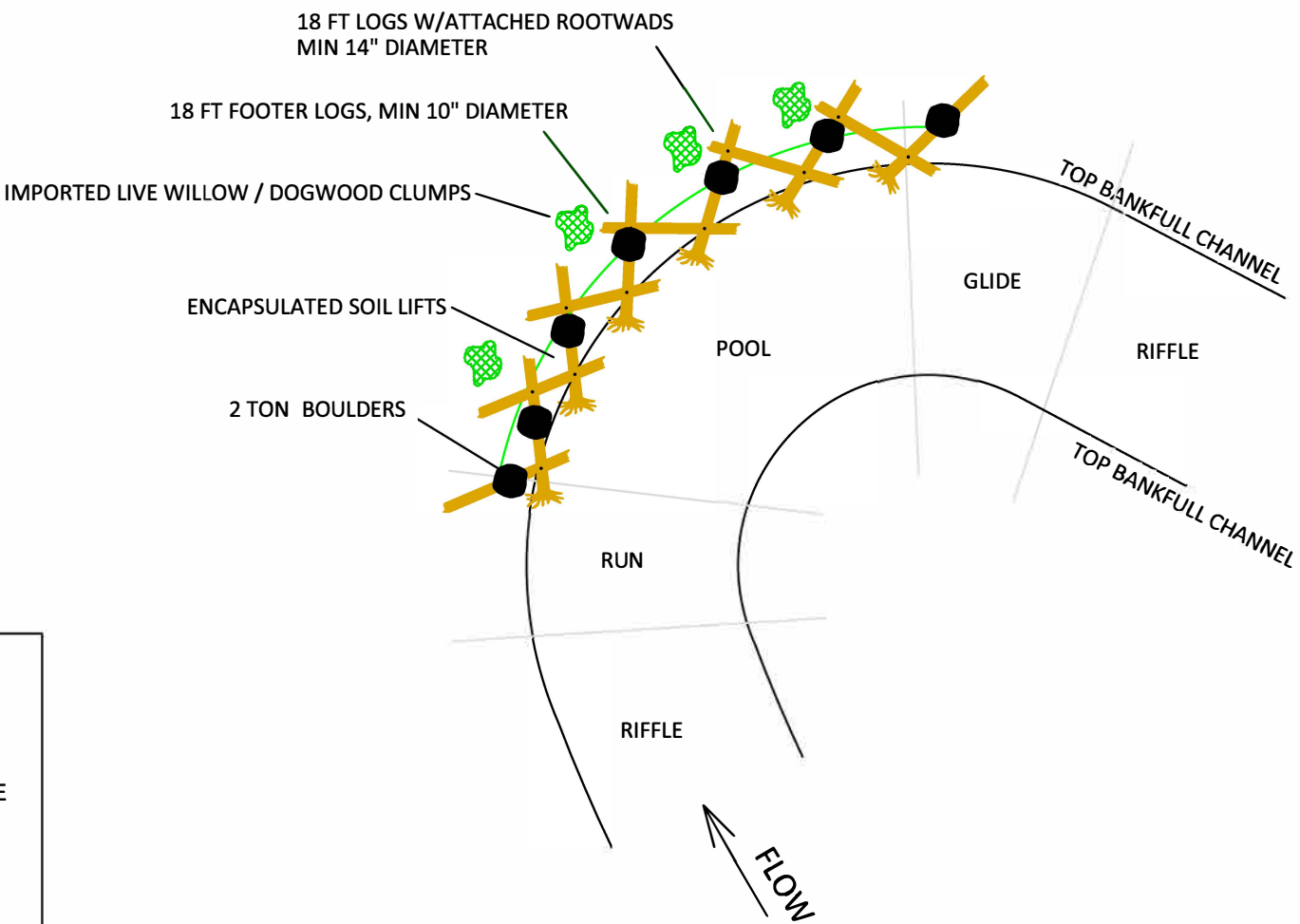


INSTALLATION NOTES

1. PLACE FOOTER LOGS AND LOGS WITH ATTACHED ROOTWAD AROUND MEANDER BEND BANK AS SHOWN. ENSURE ROOTWADS ARE ORIENTED PERPENDICULAR TO FLOW AS SHOWN. LOGS WILL BE A COMBINATION OF ONSITE AVAILABLE MATERIALS FROM LEVEE CLEARING, EXISTING DOWNED TREES, AND IMPORTED MATERIAL TO BE PROCURED BY THE CONTRACTOR. TREES FROM RECENTLY CLEARED FILED TREE ROWS AND/OR CLEARING AND SNAGGING PROJECTS IN THE AREA WILL TYPICALLY BE ACCEPTABLE. LOGS WILL BE SOUND AND FREE OF ROT.
2. PRE-DRILL HOLES FOR REBAR PINS, THEN INSTALL REBAR BY VIBRATING IN WITH A PNEUMATIC POST DRIVER. BEND THE TOP OF THE PROTRUDING REBAR FLUSH WITH THE TOP LOG. PLACE ALL LOGS ALONG THE MEANDER, THEN PLACE EARTHFILL OVER THEM. USE EXCAVATOR BUCKET TO COMPACT TO AN ELEVATION JUST ABOVE TOP OF LOGS TO BE BURIED, ADDING WATER TO FILL MATERIAL AS REQUIRED.
3. PLACE BOULDERS AS SHOWN, ENSURING SOLID CONTACT OVER EACH LOG WITH ATTACHED ROOTWAD. EACH BOULDER WILL HAVE A MINIMUM WEIGHT OF 2 TONS. THEN PROCEED TO PLACE AND COMPACT EARTHFILL UP TO THE FLOODPLAIN ELEVATION BEHIND THE ENCAPSULATED SOIL LIFT WALL AND TO THE BOTTOM LAYER OF THE WALL.
4. THE INNER FABRIC LAYER ON ENCAPSULATED SOIL LIFTS WILL BE FULLY BIODEGRADABLE DOUBLE JUTE EROSION CONTROL BLANKET, MINIMUM 9.5 OZ/SQYD, RATED FOR 2.35 PSF UNVEGETATED. A SINGLE 8 FT WIDE ROLL OF BIONET C125BN (OR APPROVED SUBSTITUTE) CENTERED ON THE FACE OF EACH LIFT WILL PROVIDE ADEQUATE LENGTH INTO THE BANK. THAT SAME MATERIAL, IN TWO LAYERS, WILL BE PLACED ABOVE THE COBBLE SECTION OF FILL BELOW THE BANK. THE OUTER LAYER FABRIC LAYER ON THE LIFTS WILL BE FULLY BIODEGRADABLE COIR FABRIC, MINIMUM 18 OZ/SQYD, 50% OPEN AREA, RATED FOR 10 FT/SEC. A SINGLE 13.1 FT WIDE ROLL OF GEOCOIR 700 (OR AN APPROVED SUBSTITUTE) CENTERED ON THE FACE OF EACH LIFT, WITH 6" OVERLAPS, IS ANTICIPATED. OVERLAPS AT ROLL ENDS FOR ALL FABRICS WILL BE 6 INCHES MINIMUM.
5. CONSTRUCT ENCAPSULATED SOILS LIFTS AS SHOWN, UTILIZING A WALK BEHIND VIBRATORY PLATE COMPACTOR OR HYDRAULIC PLATE COMPACTOR ATTACHMENT FOR AN EXCAVATOR, FOR COMPACTION WITHIN THE LIFTS. LIVE WILLOW AND DOGWOOD TO BE USED WITHIN THE LIFTS WILL BE HARVESTED NO MORE THAN 2 WEEKS PRIOR FROM LOCAL SOURCES, STRIPPED OF LEAVES, AND SOAKED A MINIMUM OF 48 HOURS PRIOR TO PLACEMENT. SEE CS-6 FOR ADDITIONAL DETAILS ON LIVE MATERIALS AND GRASS SEED TO BE PLACED BELOW FABRIC AT TOP AND ON FACE. 2" OF LOOSE SOIL SHOULD BE RAKED OVER FABRIC, LIVE MATERIALS PLACED, AND THEN AN ADDITIONAL 2" OF SOIL RAKED OVER CUTTINGS. ALL OF THAT WILL BE WATERED WELL PRIOR TO CONSTRUCTION OF THE NEXT LIFT.
6. LIVE CLUMP TRANSPLANTS WILL BE TRIMMED WITHIN 2 FT OF THE GROUND, HAVE THEIR ROOT SYSTEMS FULLY EXCAVATED AND BE TRANSPORTED TO THE PROJECT SITE, AND BE TRANSPLANTED INTO THE TOP OF THE BANK WITHIN 48 HOURS. DEPENDING ON WEATHER CONDITIONS, THIS MAY OCCUR IN THE FALL FOLLOWING CONSTRUCTION OR THE FOLLOWING SPRING, SEE CS-6.

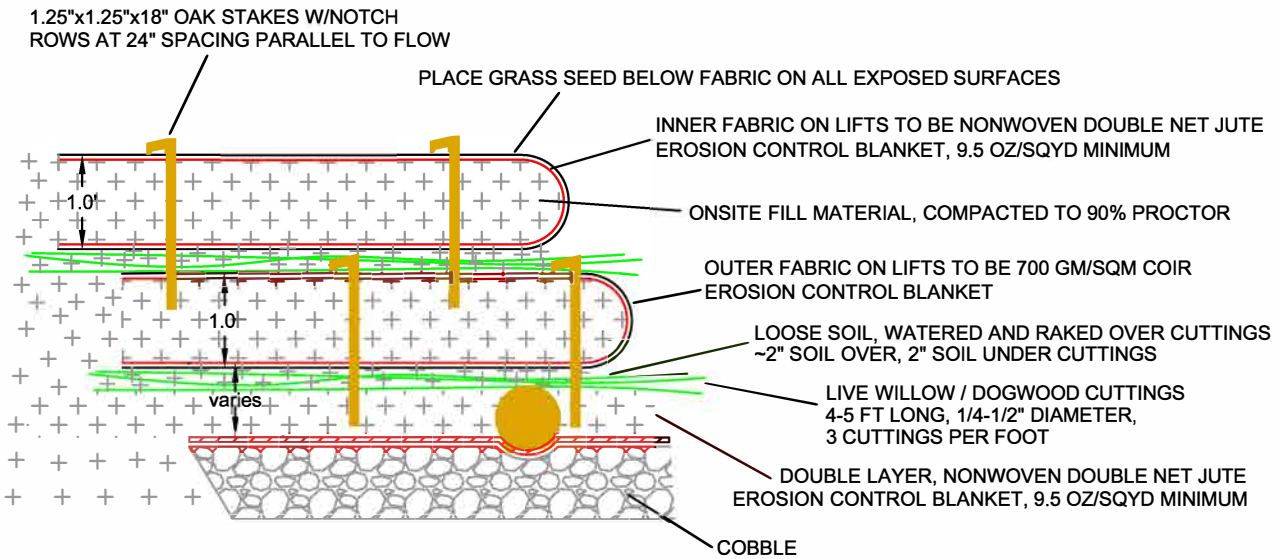
PLAN VIEW- TYPE 1 BANK PROTECTION

NOT TO SCALE



ENCAPSULATED SOIL LIFT DETAIL

NOT TO SCALE



Date _____
Designed _____
Drawn _____
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Approved _____

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
TYPE 1 BANK PROTECTION DETAILS
PEMBINA COUNTY NORTH DAKOTA

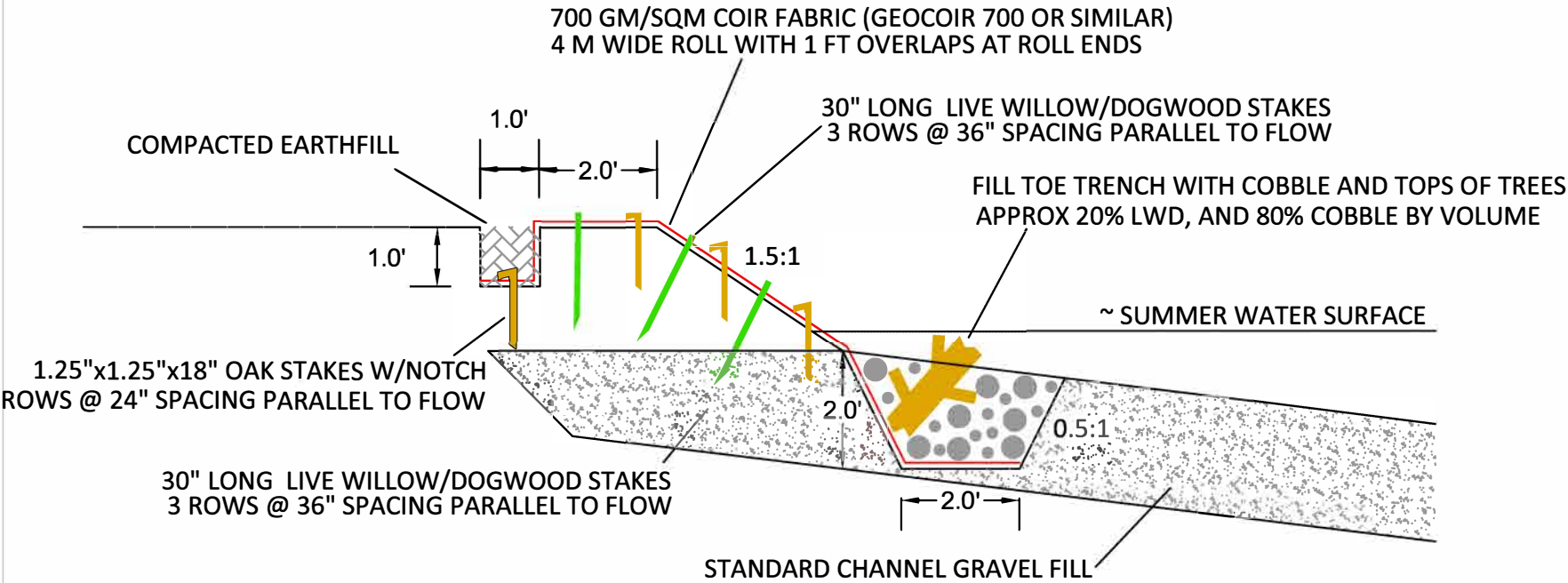


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Date
July, 2021

TYPE 2 BANK PROTECTION DETAILS

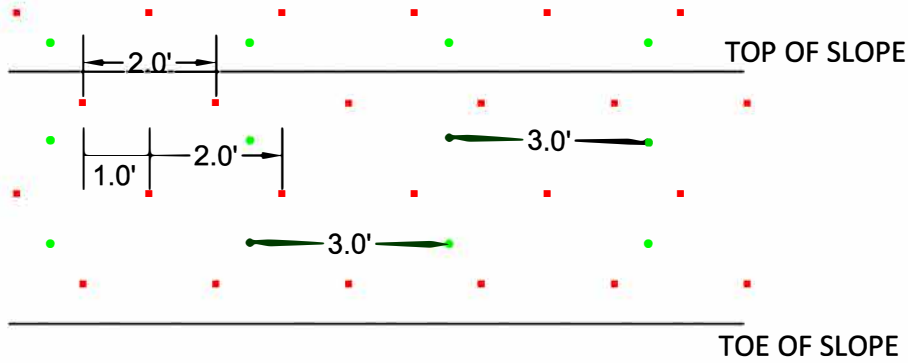
TYPE 2 BANK PROTECTION- CROSS SECTION

NOT TO SCALE



STAKING DETAIL

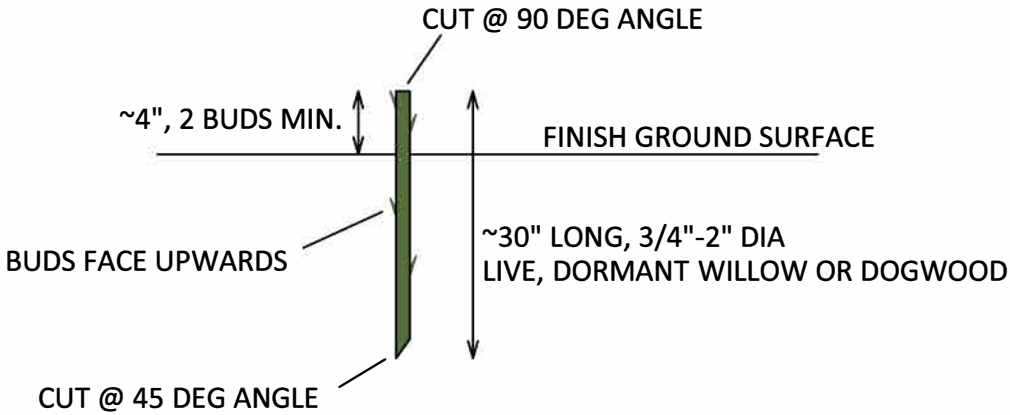
NOT TO SCALE



- =1.25"x1.25"x18" OAK STAKES W/NOTCH
- =30" LONG LIVE WILLOW/DOGWOOD STAKES

LIVE STAKE INSTALLATION

NOT TO SCALE



INSTALLATION NOTES

1. CONSTRUCT STREAM CHANNEL TO GRADE, AND COMPACT EARTHFILL AS NEEDED TO CREATE ADJACENT BANKS.
2. UTILIZE SMALL BUCKET MACHINERY TO EXCAVATE TOP AND TOE TRENCHES. EXCAVATED GRAVEL TO BE UTILIZED FOR ADJACENT TYPE 1 BRANCH/GRAVEL MIX.
3. SPREAD AND RAKE IN GRASS SEED ON THE SLOPE AS OUTLINES IN CS-6.
4. COIR FABRIC WILL BE FULLY BIODEGRADABLE COIR FABRIC, MINIMUM 18 OZ/SQYD, 50% OPEN AREA, RATED FOR WATER VELOCITIES TO 10 FT/SEC (EXAMPLE BELTON GEOCOIR 700). SPREAD COIR FABRIC ACROSS SLOPE, INTO KEY TRENCHES. SEAMS AT ENDS OF ROLLS SHALL BE AT LEAST 1 FT OVERLAP, WITH A STAKE ON THE OVERLAP. ENSURE UPSTREAM PIECE OVERLAPS DOWNSTREAM AT SEAMS. ONE 13.1 FT WIDE ROLL WILL PROVIDE ADEQUATE COVERAGE ON THE SLOPE.
5. HAND INSTALL 1.25"x1.25"x18" OAK STAKES AS SHOWN, IN 1 ROW IN THE TOP TRENCH, 3 ROWS ON THE SLOPE, ALL AT 24" SPACING, STAGGERED AS SHOWN IN DETAIL.
6. COMPACT FILL BACK INTO UPPER KEY TRENCH. COMPACTION MAY BE VIA TAMPING OF A MIN 2.5 MTON HYDRAULIC EXCAVATOR OR A WALK BEHIND VIBRATORY PLATE COMPACTOR.
7. PLACE COBBLE AND TREE TOPS OR LARGE BRANCHES IN TOE TRENCH. MINIMUM DIAMETER OF WOODY DEBRIS IS 6 INCHES. TAMP WITH EXCAVATOR BUCKET.
8. DURING JANUARY/FEBRUARY FOLLOWING CONSTRUCTION, HARVEST DORMANT WILLOW AND DOGWOOD POSTS AT THE LOCAL SITES OUTLINED IN CS-6. STORE IN A COOL, DRY LOCATION.
9. IN SPRING, SOAK LIVE STAKES FOR MINIMUM 48 HOURS PRIOR TO INSTALLATION.
10. UTILIZE HYDRAULIC INJECTOR, PUNCH BAR, OR AUGER TO CREATE PILOT HOLES AT 3 FT SPACING PARALLEL TO THE RIVER IN EACH OF THE 3 ROWS. PLACE LIVE STAKES INTO HOLES, TAMP, AND WASH IN WITH WATER TO ENSURE GOOD SOIL CONTACT. SEE CS-6 FOR ADDITIONAL DETAILS.

Date _____
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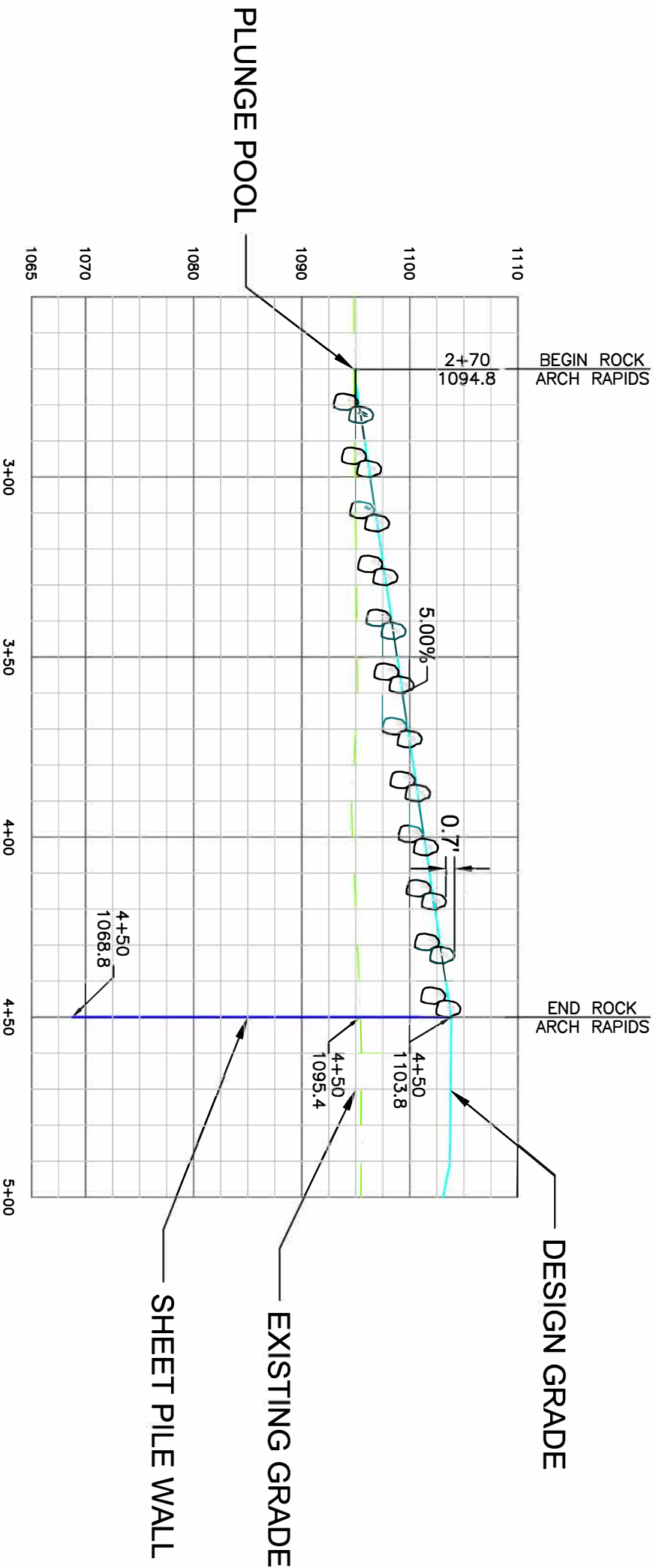
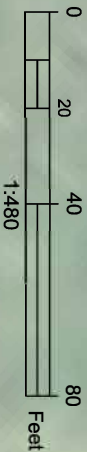
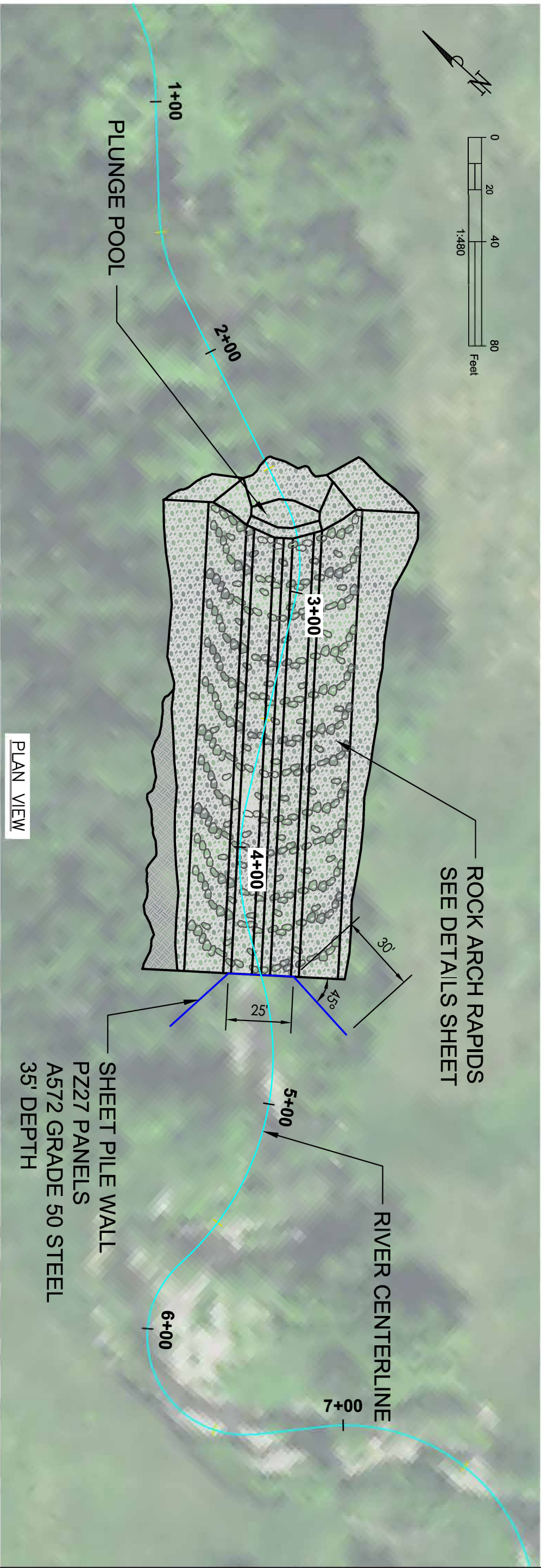
TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
TYPE 2 BANK PROTECTION DETAILS

PEMBINA COUNTY NORTH DAKOTA



File Name
Tongue_Details.dwg

Date
July, 2021



ROCK ARCH RAPIDS PROFILE VIEW

SCALE: 3V:1H



TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

ROCK ARCH RAPIDS - PLAN AND PROFILE

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

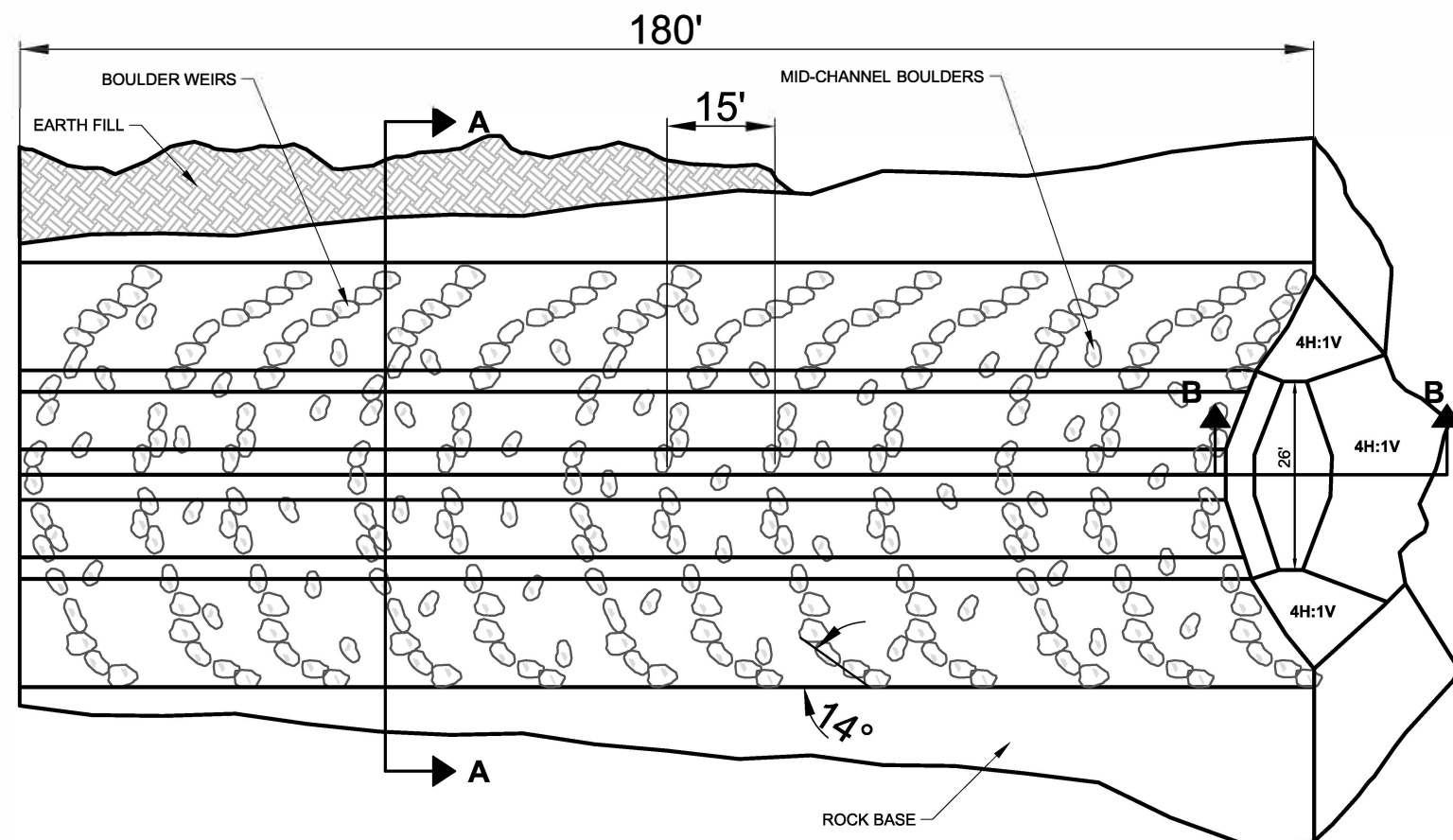
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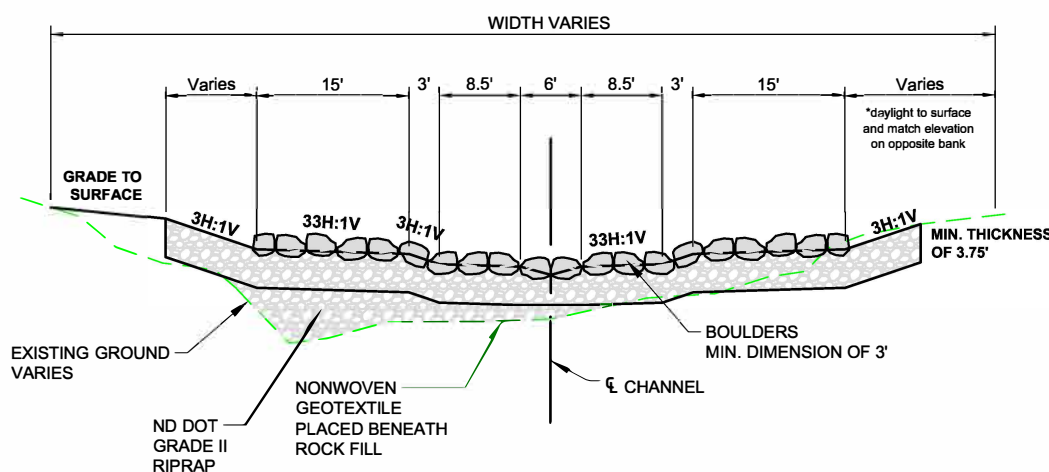
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NOT FOR CONSTRUCTION



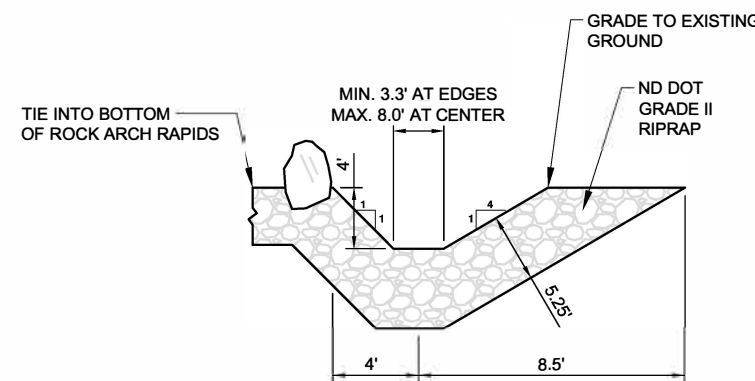
PLAN VIEW
NOT TO SCALE



EXAMPLE STRUCTURE
LOOKING UPSTREAM



TYPICAL SECTION A-A
NOT TO SCALE



TYPICAL SECTION B-B
NOT TO SCALE

INSTALLATION NOTES

1. SHEET PILE PANELS SHALL BE 35-FEET IN LENGTH AND INSTALLED ACROSS THE CHANNEL AT THE LOCATION SHOWN IN THESE DRAWINGS. PANELS SHALL DRIVEN SUCH THAT THE TOP OF THE SHEET PILE WALL MATCHES THE UPSTREAM CROSS SECTION OF THE ROCK ARCH RAPIDS.
2. NONWOVEN 8 OZ GEOTEXTILE SHALL BE PLACED BENEATH ROCK FILL WITHIN CHANNEL. FABRIC SHALL BE PLACED PARALLEL TO THE CHANNEL, BEGINNING ON THE CHANNEL BOTTOM AND PROGRESSING UP THE SLOPE. OVERLAPS SHALL BE A MINIMUM OF 18". FABRIC SHALL BE ANCHORED AS NECESSARY WITH ANCHORING PINS TO PREVENT MOVEMENT PRIOR TO ROCK INSTALLATION.
3. NORTH DAKOTA DOT GRADE II RIPRAP SHALL BE USED TO FILL THE CHANNEL TO THE GRADES AND CROSS SECTION DIMENSIONS SHOWN IN THESE DRAWINGS. RIPRAP SHALL BE PLACED WITH CARE TO PREVENT DAMAGE TO THE GEOTEXTILE FILTER FABRIC.
4. CHOKE STONE SHALL BE PLACED IN THE TOP 2-FEET OF ROCK FILL BETWEEN THE SHEET PILE WALL AND THE MOST UPSTREAM BOULDER WEIR. CHOKE STONE SIZE SHALL BE GRADED TO CONSIST OF 3/8" - 6" STONE SIZE.
5. COMPACTED EARTH FILL SHALL BE PLACED BEYOND THE EXTENTS OF THE ROCK ARCH RAPIDS AS NECESSARY TO MATCH EXISTING GROUND. MATERIAL SHALL BE FREE OF LARGE ROCKS, ORGANICS, AND OTHER DELETERIOUS MATERIAL. LOCATIONS OF REQUIRED EARTH FILL ARE SHOWN IN THESE DRAWINGS.
6. BOULDERS SHALL HAVE APPROXIMATE DIMENSIONS OF 3'x3'x5' AND SHALL BE FREE OF SHALE, CRACKS, AND OTHER DEFECTS.
7. BOULDER WEIRS SHALL BE INSTALLED AT THE DIMENSIONS SHOWN IN THESE DRAWINGS. AND SHALL BE PROTRUDE FROM THE ROCKFILL 6" - 18".
8. PLUNGE POOL SHALL BE CONSTRUCTED DOWNSTREAM OF THE LAST BOULDER WEIR TO THE DIMENSIONS SHOWN. THE MATERIAL SHALL CONSIST OF ND DOT GRADE II RIPRAP.

Date _____
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TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

ROCK ARCH RAPIDS - DETAILS

PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA



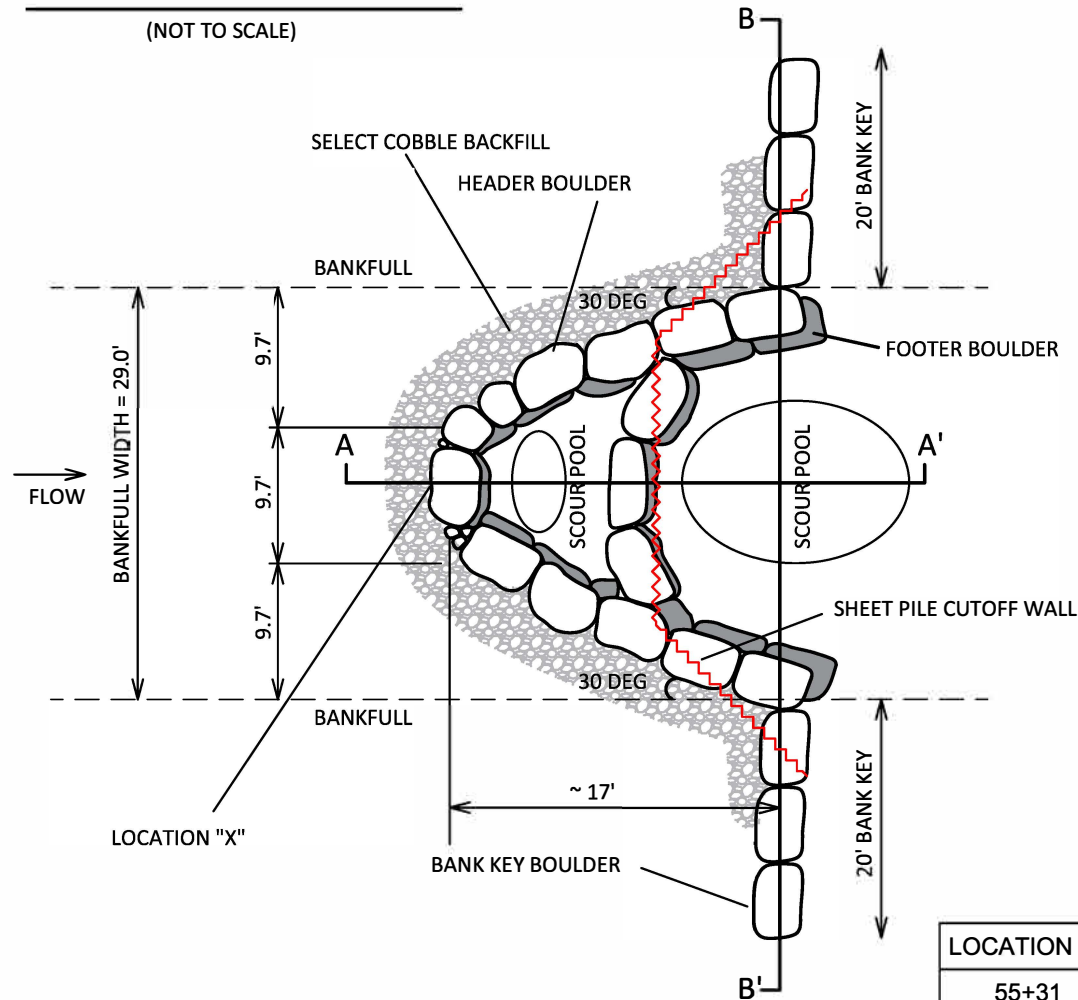
File Name
Arch Rapids.dwg

Date
July, 2021

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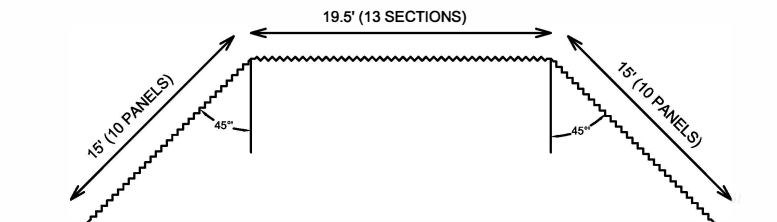
PLAN VIEW - ROCK CROSS VANE

(NOT TO SCALE)



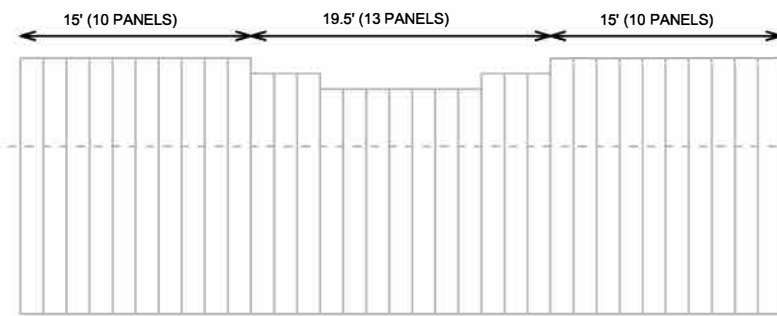
PLAN VIEW - SHEET PILE

(NOT TO SCALE)



SECTION VIEW - SHEET PILE

(NOT TO SCALE)



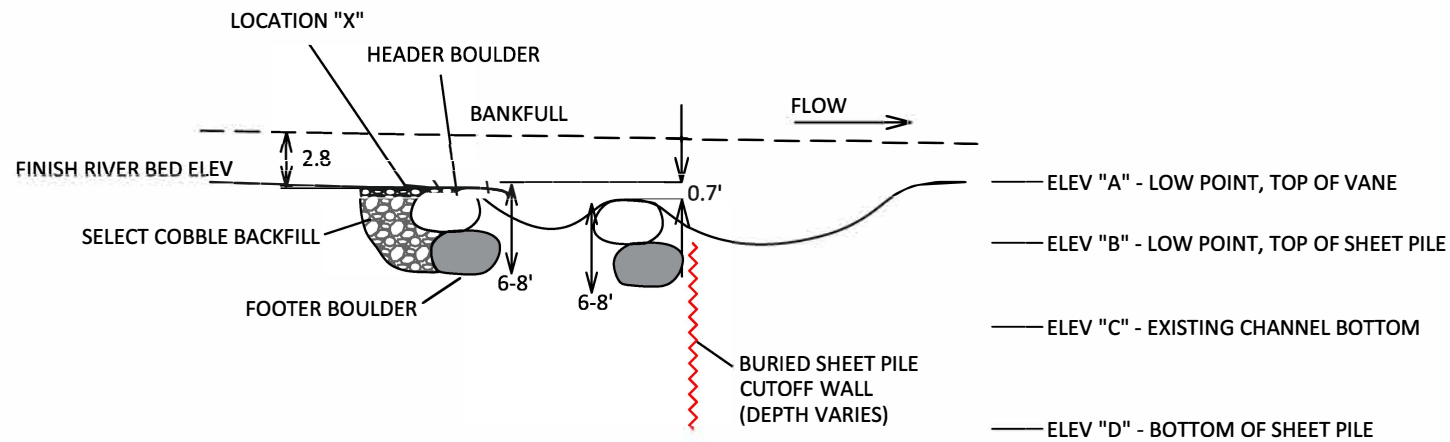
ELEVATION "B" + 2 FT
ELEVATION "B" + 1 FT
ELEVATION "B"

ELEVATION "C" - EXISTING CHANNEL BOTTOM

ELEVATION "D"

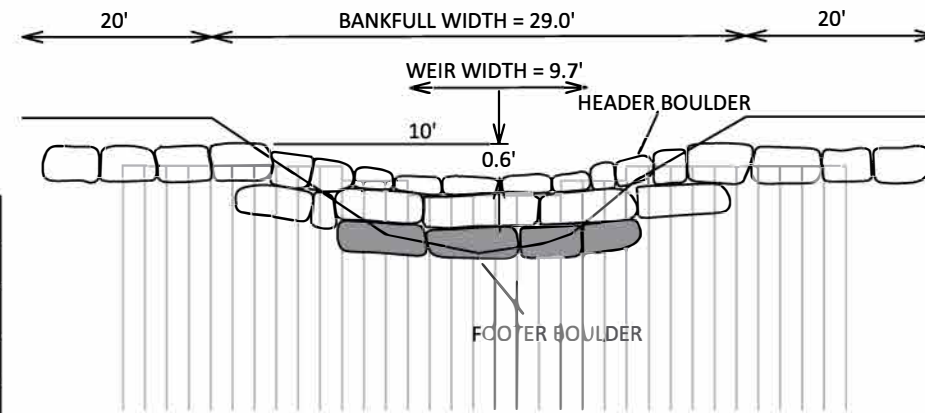
SEC A-A' - ROCK CROSS VANE

(NOT TO SCALE)



SEC B-B' - TYPICAL CROSS VANE

(NOT TO SCALE)



CROSS VANE DETAILS

LOCATION "X"	ELEV "A"	ELEV "B"	ELEV "C"	ELEV "D"
55+31	1116.8	1114.8	1112.8	1100.8
86+25	1125.1	1123.1	1119.8	1104.1
98+50	1128.4	1126.4	1125.6	NO SHEETPILE
101+75	1129.1	1127.1	1127.5	NO SHEETPILE

INSTALLATION NOTES

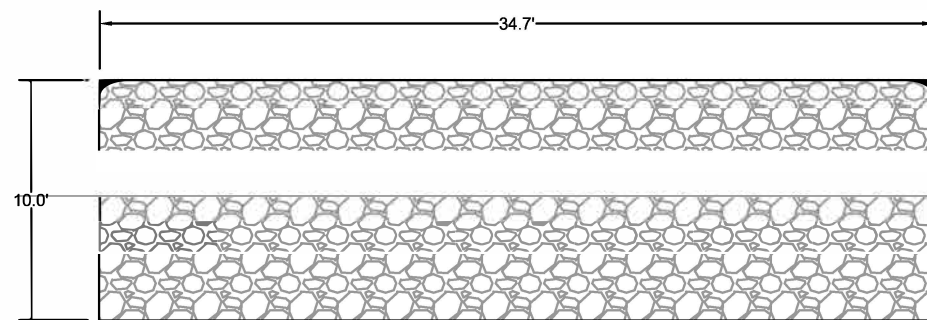
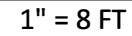
- BOULDERS TO BE UTILIZED FOR ROCK CROSS VANE CONSTRUCTION WILL BE ANGULAR SHOT ROCK, OF A GENERALLY RECTANGULAR SHAPE. THE SOURCE WILL BE APPROVED BY NRCS PRIOR TO USE. TYPICAL BOULDER DIMENSIONS WILL BE APPROXIMATELY 4'x3'x2' FOR CONSTRUCTION OF THE VANE. USE A MIXTURE OF SMALLER SIZES (NDDOT CLASS II RIPRAP) TO FILL VOIDS BETWEEN THE BOULDERS FOR THE INSTREAM PORTION OF THE VANE. THE LEAST DIMENSION OF ANY ONE BOULDER SHOULD NOT BE LESS THAN 50% THE DIMENSION OF THE GREATEST DIMENSION OF THE BOULDER. MINIMUM BOULDER WEIGHT = 1.5 TONS.
- COBBLE TO BE UTILIZED FOR BACKFILL WILL BE IDENTICAL IN GRADATION TO THAT USED FOR COBBLE PATCHES.
- VANES WILL BE CONSTRUCTED WITH AN NRCS INSPECTOR PRESENT. BOULDERS WILL BE PLACED BY AN EXCAVATOR WITH CAREFUL ATTENTION TO ENSURE THEY ARE STABLE AND GAPS ARE REDUCED TO A MINIMUM. EACH BOULDER SHOULD BEAR AGAINST IT'S DOWNSTREAM NEIGHBOR SO THAT THE FORCE OF STREAMFLOW IS TRANSFERRED THROUGH THE WEIR TO THE BANK. AS EACH BOULDER IS PLACED, GIVE A TRIAL PUSH IN THE DOWNSTREAM DIRECTION TO ENSURE IT WILL NOT TIP UNDER LOAD.
- ROCK CROSS VANES AT 55+31 AND 86+25 WILL HAVE EMBEDDED SHEET PILE CUTOFF WALLS DRIVEN INTO THE CHANNEL BED, AS SHOWN. SHEET PILE WILL CONSIST OF INTERLOCKING PZ27 PANELS, OF AS72 GRADE 50 STEEL. VANE AT 55+31 WILL REQUIRE 792 SF AND VANE AT 86+25 1,089 SF OF SHEET PILE.
- ANTICIPATED QUANTITIES FOR EACH INDIVIDUAL CROSS VANE ARE: 30 BOULDERS (45 TONS)
30 TONS ND DOT CLASS II RIPRAP
45 TONS COBBLE

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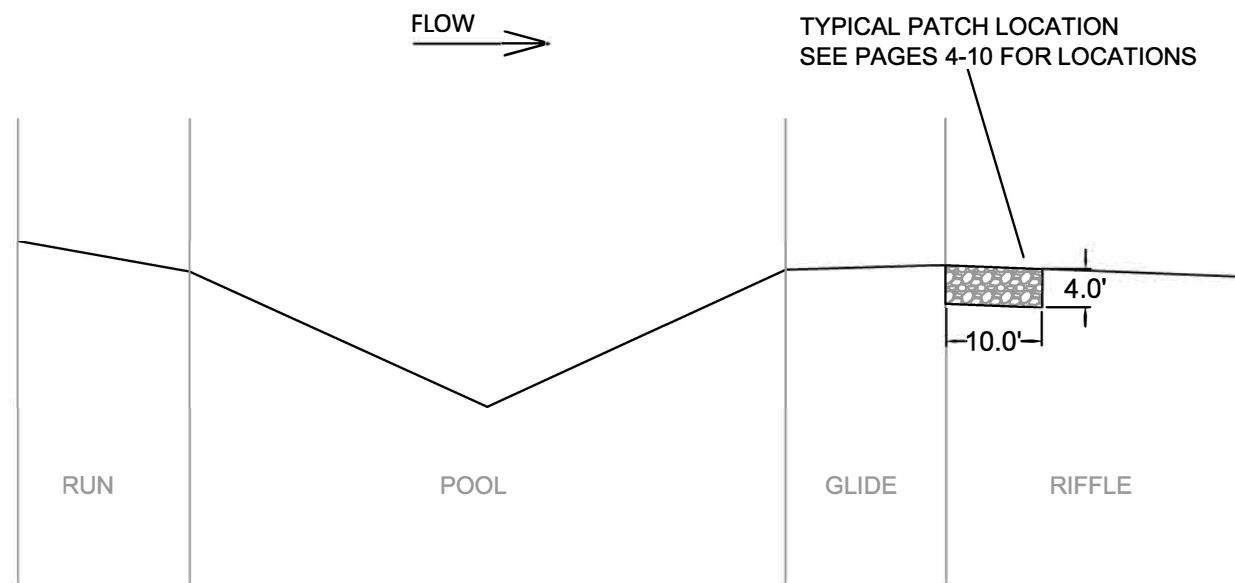
TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
ROCK CROSS VANE DETAILS
PEMBINA WRD, PEMBINA COUNTY NORTH DAKOTA

NRCS
Natural Resources Conservation Service

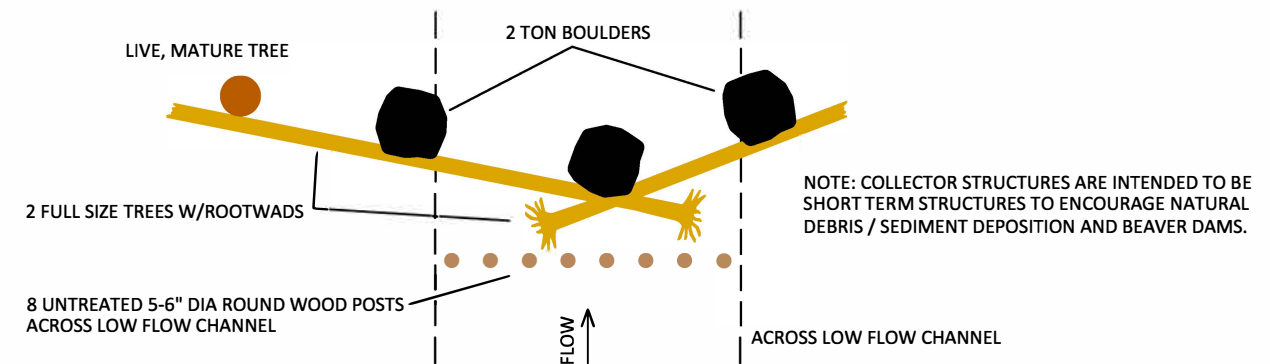
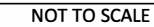
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Date
July, 2021

$$1'' = 8 \text{ FT}$$


NOT TO SCALE

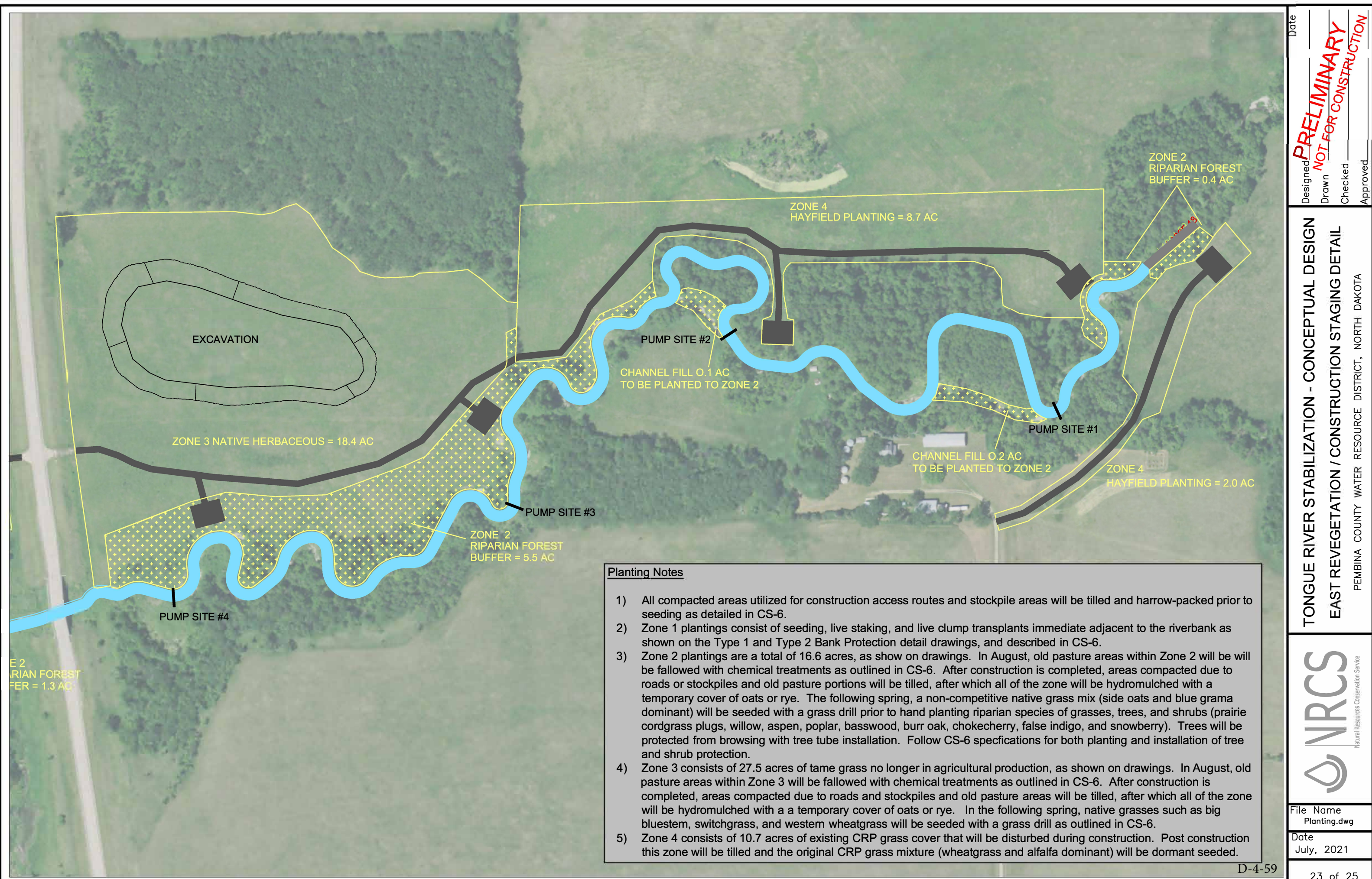


NOT TO SCALE, LOOKING UPSTREAM @ STRUCTURE



1. DEBRIS COLLECTOR WILL BE INCLUDE TWO TREES, WITH ROOTWADS AND AS MANY BRANCHES THAT ARE PRACTICAL TO LEAVE ON. ONE SIDE WILL BE PLACED ON THE UPSTREAM SIDE OF A LIVE TREE, THE OTHER WILL BE ANCHORED TO A LARGE BOULDER TO BE IMPORTED TO THE SITE. A SECOND BOULDER WILL BE PLACED DOWNSTREAM OF WHERE THE TWO TREES CROSS. ANGLE TREES TO ENSURE EVEN AT LOW FLOWS A MINIMUM 50% POROSITY IS ALLOWED. UNTREATED 4" DIAMETER ROUND POSTS WILL BE PUSHED INTO THE RIVERBED AS SHOWN. WOODY DEBRIS FROM UPSTREAM AND/OR THE LOCAL BEAVER POPULATION WILL THEN AGGRADE UPSTREAM OF THE STRUCTURE NATURALLY. DEBRIS COLLECTORS TO BE CONSTRUCTED AT 101+75 AND 105+00.
2. COBBLE PATCHES WILL BE CONSTRUCTED TO MAINTAIN GRADE AT THE UPSTREAM END OF RIFFLES LISTED BELOW, AND SHOWN ON THE PROFILE SHEETS: 11+97, 19+28, 27+90, 36+90, 42+87, 49+54, 59+75, 70+32, 78+08, 84+48, 90+97, 97+31. THE VOLUME OF MATERIAL REQUIRED FOR EACH IS 45 CY.
3. COBBLE MATERIAL WILL BE ROUNDED ON AT LEAST 2 SIDES, CONFORM TO MS-521 REQUIREMENTS, AND MEET THE FOLLOWING GRADATION:

SIEVE SIZE	% PASSING
10"	100%
8"	80-90%
4"	35-50%
2"	10-35%
1"	0-15%



Planting Notes

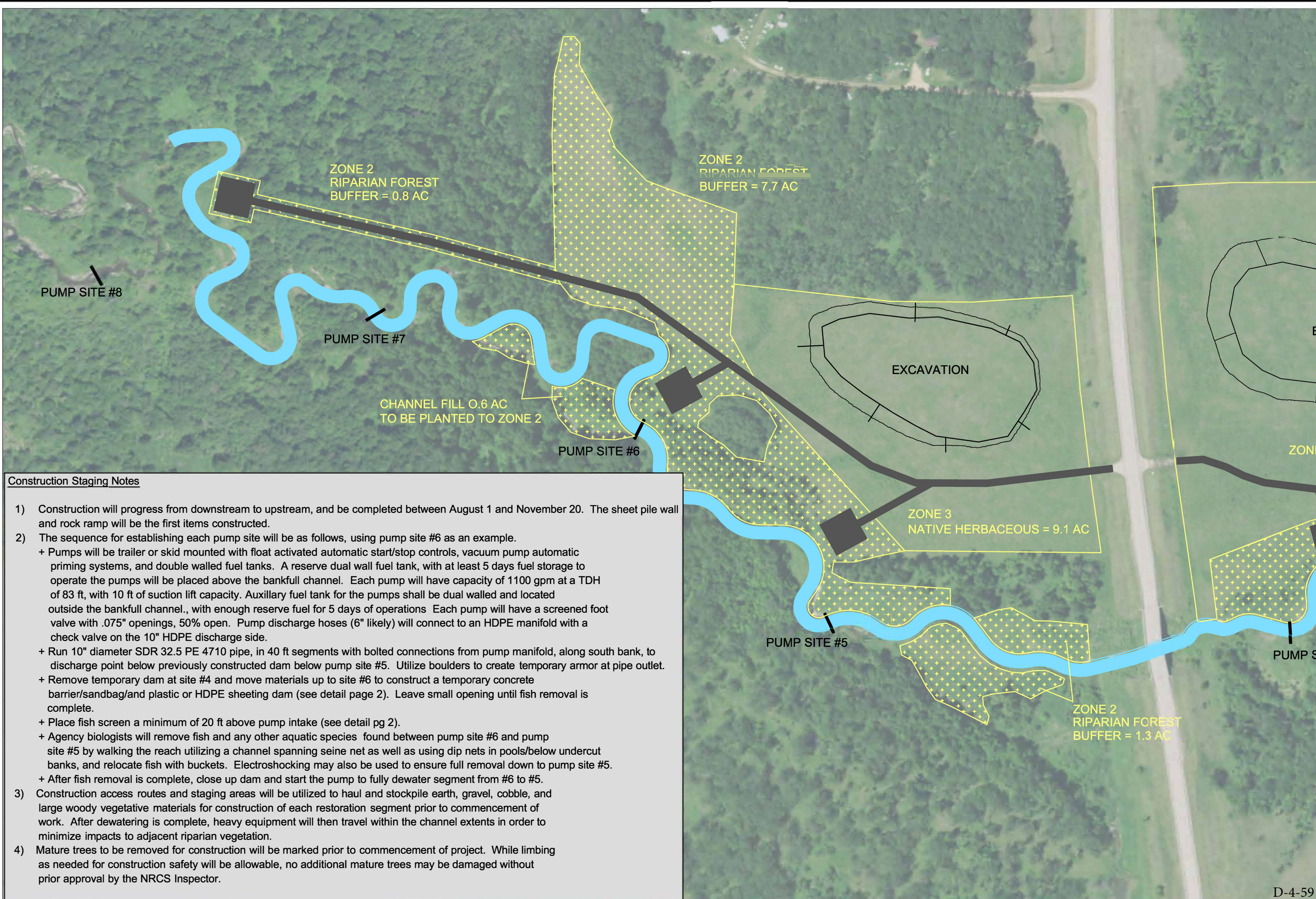
- 1) All compacted areas utilized for construction access routes and stockpile areas will be tilled and harrow-packed prior to seeding as detailed in CS-6.
- 2) Zone 1 plantings consist of seeding, live staking, and live clump transplants immediate adjacent to the riverbank as shown on the Type 1 and Type 2 Bank Protection detail drawings, and described in CS-6.
- 3) Zone 2 plantings are a total of 16.6 acres, as show on drawings. In August, old pasture areas within Zone 2 will be will be fallowed with chemical treatments as outlined in CS-6. After construction is completed, areas compacted due to roads or stockpiles and old pasture portions will be tilled, after which all of the zone will be hydromulched with a temporary cover of oats or rye. The following spring, a non-competitive native grass mix (side oats and blue grama dominant) will be seeded with a grass drill prior to hand planting riparian species of grasses, trees, and shrubs (prairie cordgrass plugs, willow, aspen, poplar, basswood, burr oak, chokecherry, false indigo, and snowberry). Trees will be protected from browsing with tree tube installation. Follow CS-6 specifications for both planting and installation of tree and shrub protection.
- 4) Zone 3 consists of 27.5 acres of tame grass no longer in agricultural production, as shown on drawings. In August, old pasture areas within Zone 3 will be fallowed with chemical treatments as outlined in CS-6. After construction is completed, areas compacted due to roads and stockpiles and old pasture areas will be tilled, after which all of the zone will be hydromulched with a a temporary cover of oats or rye. In the following spring, native grasses such as big bluestem, switchgrass, and western wheatgrass will be seeded with a grass drill as outlined in CS-6.
- 5) Zone 4 consists of 10.7 acres of existing CRP grass cover that will be disturbed during construction. Post construction this zone will be tilled and the original CRP grass mixture (wheatgrass and alfalfa dominant) will be dormant seeded.

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TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN
EAST REVEGETATION / CONSTRUCTION STAGING DETAIL
PEMBINA COUNTY WATER RESOURCE DISTRICT, NORTH DAKOTA



File Name
Planting.dwg
Date
July, 2021



Construction Staging Notes

- 1) Construction will progress from downstream to upstream, and be completed between August 1 and November 20. The sheet pile wall and rock ramp will be the first items constructed.
- 2) The sequence for establishing each pump site will be as follows, using pump site #6 as an example.
 - + Pumps will be trailer or skid mounted with float activated automatic start/stop controls, vacuum pump automatic priming systems, and double walled fuel tanks. A reserve dual wall fuel tank, with at least 5 days fuel storage to operate the pumps will be placed above the bankfull channel. Each pump will have capacity of 1100 gpm at a TDH of 83 ft, with 10 ft of suction lift capacity. Auxillary fuel tank for the pumps shall be dual walled and located outside the bankfull channel., with enough reserve fuel for 5 days of operations. Each pump will have a screened foot valve with .075" openings, 50% open. Pump discharge hoses (6" likely) will connect to an HDPE manifold with a check valve on the 10" HDPE discharge side.
 - + Run 10" diameter SDR 32.5 PE 4710 pipe, in 40 ft segments with bolted connections from pump manifold, along south bank, to discharge point below previously constructed dam below pump site #5. Utilize boulders to create temporary armor at pipe outlet.
 - + Remove temporary dam at site #4 and move materials up to site #6 to construct a temporary concrete barrier/sandbag/and plastic or HDPE sheeting dam (see detail page 2). Leave small opening until fish removal is complete.
 - + Place fish screen a minimum of 20 ft above pump intake (see detail pg 2).
 - + Agency biologists will remove fish and any other aquatic species found between pump site #6 and pump site #5 by walking the reach utilizing a channel spanning seine net as well as using dip nets in pools/below undercut banks, and relocate fish with buckets. Electroshocking may also be used to ensure full removal down to pump site #5.
 - + After fish removal is complete, close up dam and start the pump to fully dewater segment from #6 to #5.
- 3) Construction access routes and staging areas will be utilized to haul and stockpile earth, gravel, cobble, and large woody vegetative materials for construction of each restoration segment prior to commencement of work. After dewatering is complete, heavy equipment will then travel within the channel extents in order to minimize impacts to adjacent riparian vegetation.
- 4) Mature trees to be removed for construction will be marked prior to commencement of project. While limbing as needed for construction safety will be allowable, no additional mature trees may be damaged without prior approval by the NRCS Inspector.

Date _____

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PRELIMINARY

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July, 2021

24 of 25

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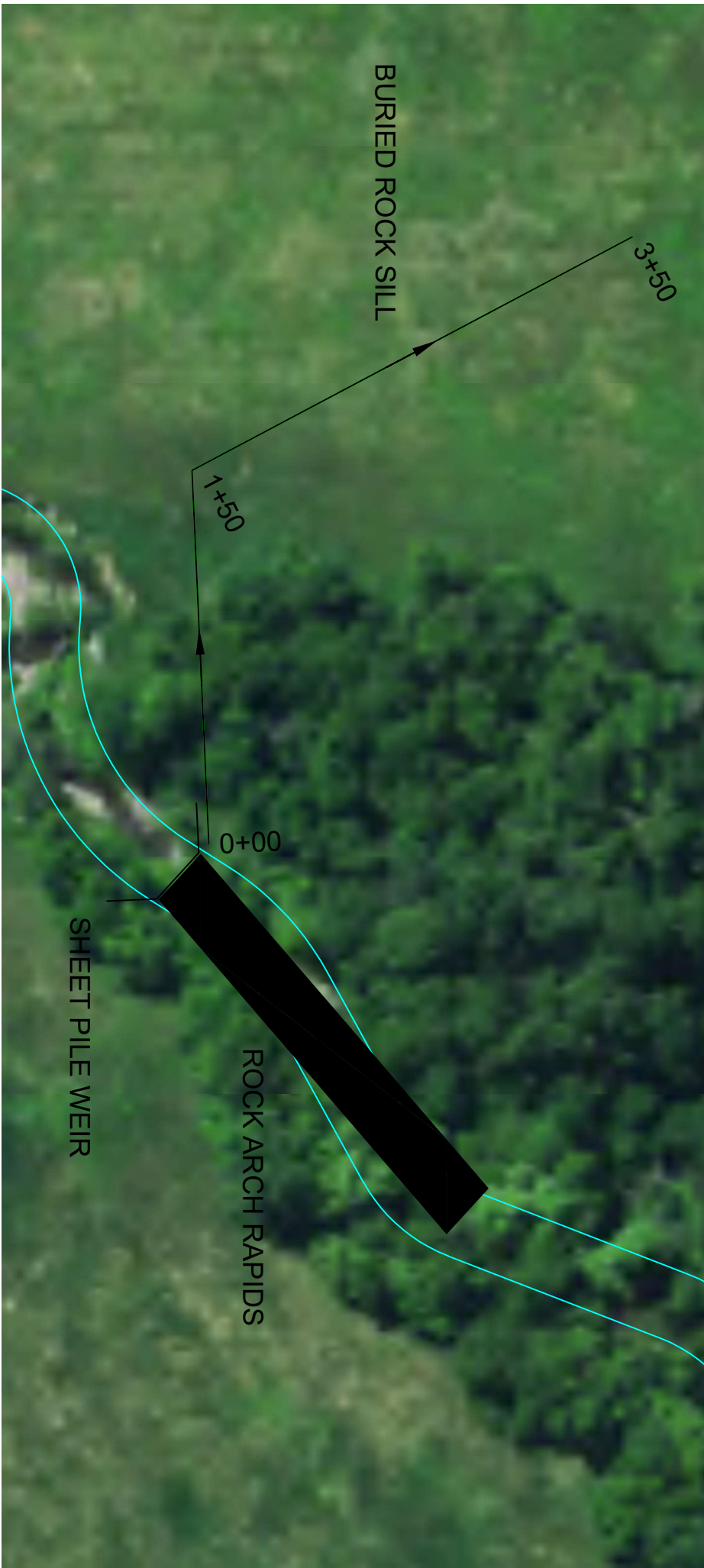
Natural Resources Conservation Service

TONGUE RIVER STABILIZATION - CONCEPTUAL DESIGN

WEST REVEGETATION / CONSTRUCTION STAGING DETAIL

PEMBINA COUNTY WATER RESOURCE DISTRICT, NORTH DAKOTA

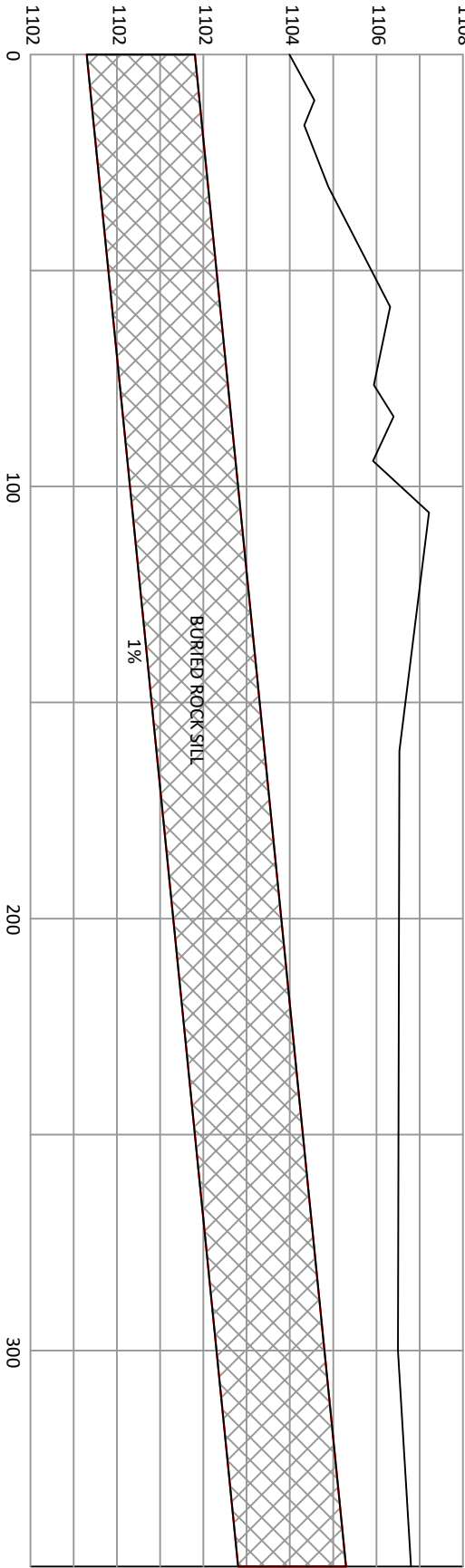
PLAN VIEW- BURIED ROCK SILL



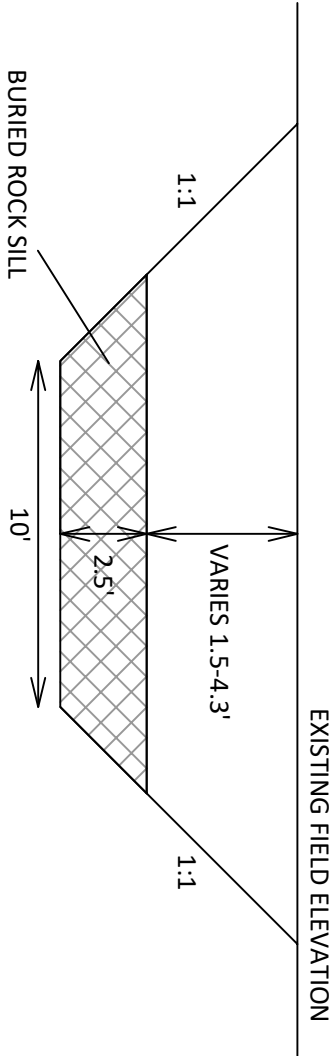
INSTALLATION NOTES

- 1. BURIED ROCK SILL TO CONSIST OF ND DOT CLASS II RIPRAP.
- 2. EXCAVATE TO GRADES SHOWN, PLACE ROCK, AND RECOMPACT EXCAVATED EARTHFILL TO 90% PROCTOR OVER THE TOP.

PROFILE VIEW- BURIED ROCK SILL



SECTION VIEW- BURIED ROCK SILL



PRELIMINARY
NOT FOR CONSTRUCTION