Poudre Ponds Bank Stabilization

Basis of Design Report

Submitted to:

City of Greeley
Water and Sewer Department
and
Culture, Parks & Recreation Dept.
Natural Areas & Trails Division
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1. Introduction

The City of Greeley (City) Water and Sewer Department and Culture, Parks, and Recreation Department, Natural Areas & Trails Division have partnered with Otak, Inc. (Otak) to stabilize an actively failing bank along the Cache la Poudre River (Poudre River) in the vicinity of Poudre Ponds. The instabilities in this reach of the Poudre are threatening not only the Poudre River Trail (Trail), but also the slurry wall creating a barrier between the Poudre River and Pond A.

1.1. Project Description

Approximately 1,300 feet downstream of the N 35th Avenue Bridge over the Poudre River, the bank of the river is eroding and threatening the stability and safety of the Trail and the Pond A slurry wall. The actively eroded section of bank is approximately 65 feet in length, with an additional 45 feet of bank upstream that is showing signs of instability. Bank erosion of varying severity is evident throughout the 600-foot-long meander bend. Several emergency repair efforts have been made along this reach, including riprap, debris, and concrete block placement in an attempt to stabilize the active erosion.

1.2. Project Goals and Objectives

This Poudre Ponds Bank Stabilization Project (Project) strives to balance the immediate needs to protect the existing slurry wall and Trail in the near-term with an ecologically-beneficial solution, while also recognizing that there is a bigger vision for the trail alignment in this reach for the long-term time frame.

The specific goals of the Project are:

- Protection of the slurry wall separating the river from Pond A.
- Protection of the Trail and user safety.

The objectives of the Project include:

- Designing bank stabilization measures to stabilize the approximately 600 feet of bank, incorporating ecological benefits/more resilient options, as feasible given the site constraints.
- Meeting a condition of ‘no rise’ for the floodplain development permit.

2. Background Information and Site Conditions

The Poudre River was historically a sinuous, single-thread channel, with frequent side channels and floodplain features. However, numerous modifications to the channel and floodplain corridor have occurred throughout the last 75 years, including gravel mining, channelization, and berms/levees, which have resulted in the straightening of the river channel, limiting lateral migration. A previous memo (submitted to the City on November 4, 2021) discussed the impacts that channelization has had on the Poudre River. Notably, stream length within the project vicinity has been reduced from approximately 3.4 miles in 1948 to 1.9 miles in 1999; a 44% reduction in stream length (Figure 1). This reduction in stream length has resulted in a steeper channel, which has higher velocities, higher shear stresses, and a higher sediment transport capacity. The reduction in stream length has resulted in a river that is presently out of equilibrium. When sediment transport capacity is increased in a river, incision and lateral migration are often the geomorphic responses to the change. Lateral migration increases stream length, which decreases the slope of the river, reducing shear stresses and sediment transport capacity. These tendencies towards lateral migration are likely causing the erosion issues that are occurring at the project site as the river attempts to return to an equilibrium state. When rivers are trending towards lateral migration, the majority of the migration occurs on the outside of meander bends, as shear stresses undercut the outside banks, which subsequently fail into the river and are transported downstream.

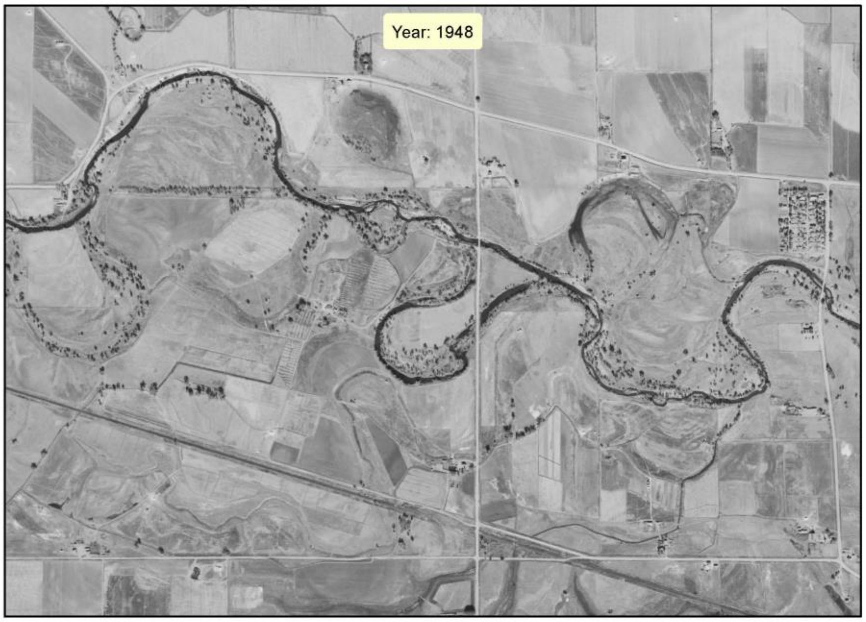

Figure 1: Comparison of Impacts to Poudre River Corridor Between 1948 and 1999

2.1. Site Visit

Site visits occurred on July 28th, September 30th, and November 5th, 2020 to assess the existing conditions of both the banks and the geomorphic character of the river, and to discuss potential alternatives with stakeholders.

The current condition of the bank in the project area is summarized in Figure 2 with the extents of each bank treatment highlighted. Areas where bank protection is absent, the banks are actively scalloping (Photo 1) and are at risk of failure during higher flow events. Historical aerial imagery shows that the top of the bank has eroded away by approximately 15 feet from 2014 to 2016 (Figure 3). Existing bank protection treatments include ecology blocks, riprap, poured asphalt, and concrete rubble (Photo 2, Photo 3, Photo 4, Photo 5). Some of these bank treatments are poorly installed, unstable, and have a high risk of undermining (especially the poured concrete slabs). Movement or undermining of the slabs would put the bank at risk and is a threat to the safety of boaters, fishermen, and anyone exploring the riverbank from the trail. Transition zones in the existing bank treatments also have the potential to fail, given the lack of tie-in from one treatment to another.

Figure 2: Existing Bank Treatments

Photo 1: Actively Scalloping Bank

Photo 2: Ecology Blocks

Photo 3: Rip Rap

Photo 4: Poured Asphalt

Photo 5: Concrete Rubble

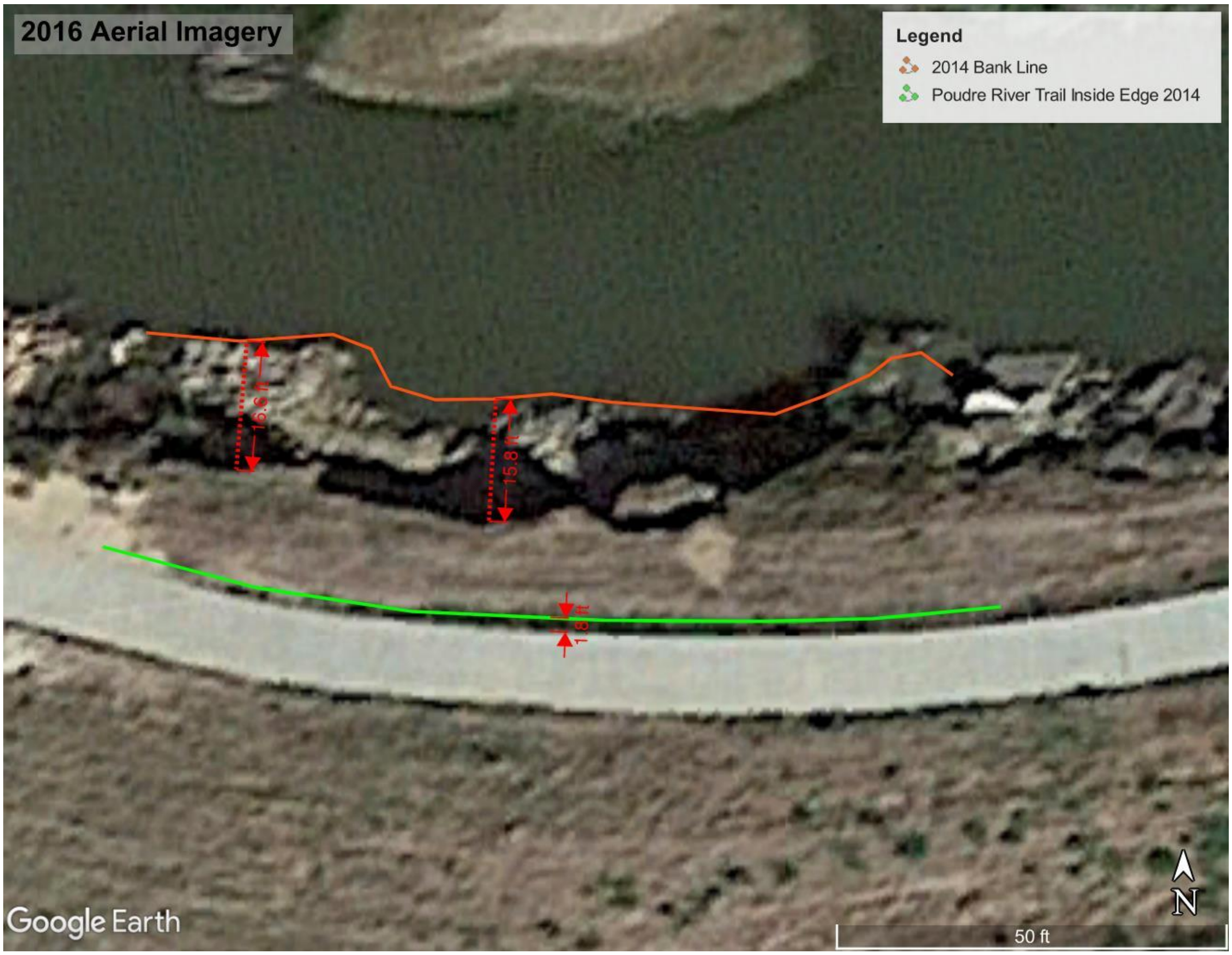

Figure 3: 2014 Bank Lines Superimposed upon Aerial Imagery from 2016
(Showing Approximately 15 feet of Top of Bank Movement)

Existing infrastructure within the project area, including surface water storage ponds, the Poudre River Trail to the south, and a berm to the north, confine the channel resulting in a lack of connected and functioning floodplain access. However, vegetated lateral, mid-channel, and point bars were identified upstream and downstream of the project area.

2.2. Geomorphology

In the vicinity of the project area, the channel has an approximate slope of 0.3% (measured using LiDAR) with a pool-riffle morphology, and an armored layer of pebbles and cobbles sitting atop coarse sands (Figure 4). Channel slope near the project area was also measured using the 2021 survey which resulted in an adverse slope. It is possible that construction of the temporary repair disturbed the channel bed resulting in a localized negative slope in this area. Additional survey after the spring runoff will reflect a more accurate slope of this reach. Embeddedness of fine material was observed atop the armored layer within the project area, likely due to active erosion of the banks, as well as the disturbance associated with the construction of emergency bank treatment. One pebble count and one bulk sample were collected to characterize the bed material (Photo 6, Table 1). An additional bulk sample was collected from the bank to characterize the bank material. Bulk samples were analyzed at Advanced Terra Testing. The majority of the bed and bank material consists of gravels, with some sands present.

Photo 6: Upstream Mid Channel Bar and Riffle (Location of Pebble Count and Bulk Sample)

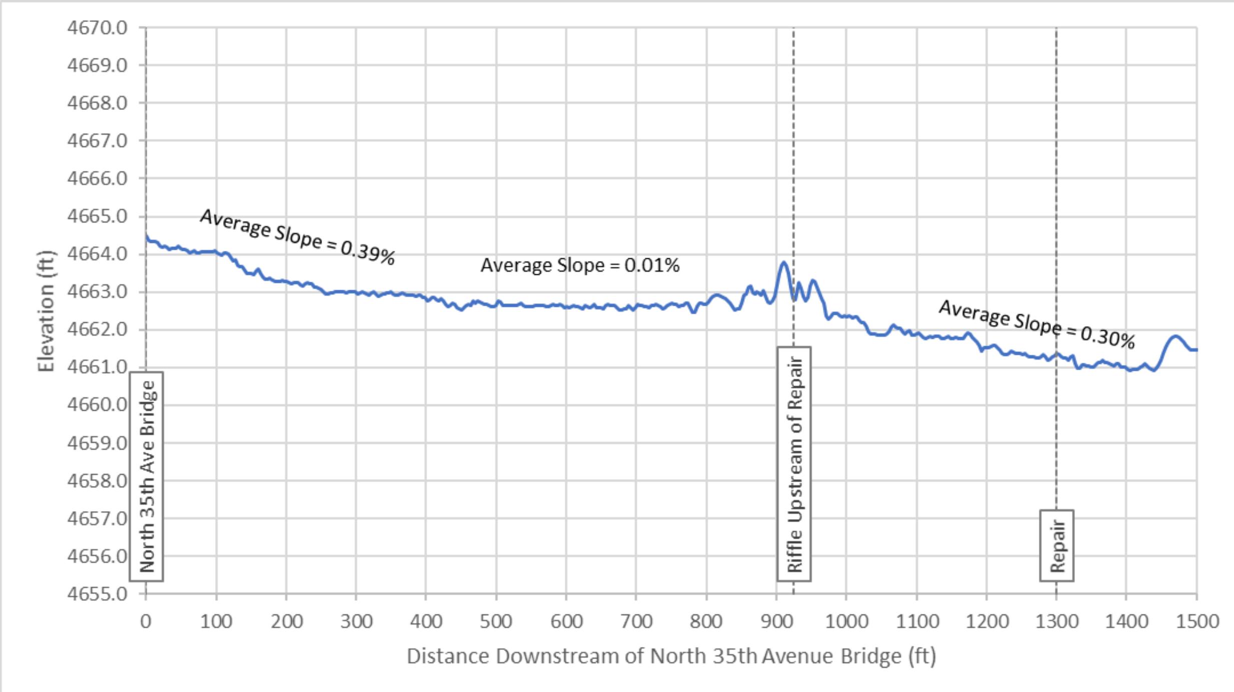

Figure 4: Profile Derived from 2013 LiDAR Data.

(The average slope of the entire reach is 0.2%. Localized slopes as noted.)

Table 1: Pebble Count and Bulk Sample Results

	Pebble Count - Bed	Bulk Sample - Bed	Bulk Sample - Bank	Average Bed
D16 (mm)	7.0	0.8	1.0	3.9
D50 (mm)	20.0	4.1	14.1	12.1
D84 (mm)	36.5	21.8	43.9	29.2
D90 (mm)	40.9	28.6	54.0	34.7

Within the project area, the river left channel banks were generally 2.5 feet tall, moderately sloped, and stable, in part due to their location on the inside of the meander bend. Limited bank failure was observed on river left upstream of the project area (Photo 7). The banks on river right were 6 to 12 feet tall, nearly vertical, and stability of the banks are dependent on the bank protection present (where banks were unstable and actively eroding where no treatments were located). Bank material predominantly consists of cohesive sands and gravels, with layers of cobbles also present (Photo 8). The bank material is very difficult to break apart when dry; however, when wetted, the banks have the potential to fail, due to the increased mass of the wetted soil, as well as reductions in frictional shear stresses. There is a significant amount of loose bank material along the toe of the bank as a result of the 15 feet of bank migration in this area. Large wood was minimally present within the project area, however, two beaver dams were observed under the N 35th Avenue Bridge (Photo 9, Photo 10). Additionally, a wood jam was identified on a side channel upstream of the project area (Photo 11). AlpineEco observed beaver entering and exiting the concrete rubble along the bank. Beaver may be utilizing the concrete rubble as a den. The river is fairly stable both upstream and downstream of the project area, with the exception of one other location just upstream of the project reach where bank erosion was observed on the outside of the meander bend (Photo 7). In terms of stream evolution, the project reach is in the stage of degradation and widening (Cluer and Thorne, 2014).

Photo 7: Evidence of Bank Failure on River Left

Photo 8: Unprotected Bank Material within Project Reach

Photo 9: Beaver Dam Located on River Left

Photo 10: Beaver Dam Located on River Left

Photo 11: Wood Jam Within Side Channel

2.3. Topography

Existing Conditions topographic survey data were collected by Boulder Land Consultants, Inc. (BLC) from September through November, 2021. Throughout the project area, the survey captured the channel thalweg, banks, thorough coverage of the overbanks, and the existing Poudre River Trail. A total of approximately 900 feet of the thalweg was surveyed (approximately 220 feet upstream and 310 feet downstream of the project extents). A surface was developed from the survey data to create a continuous existing conditions terrain model of the project area (Figure 5). The survey data are in the NAD 1983 (2011) State Plane Colorado North FIPS 0501 (US Feet) coordinate system. The vertical datum is NAVD88.

A spillway, located just upstream of the project extents, was constructed in 2021. Elevations along the crest of the spillway were surveyed by Northern Engineering in October, 2022 (Figure 5).

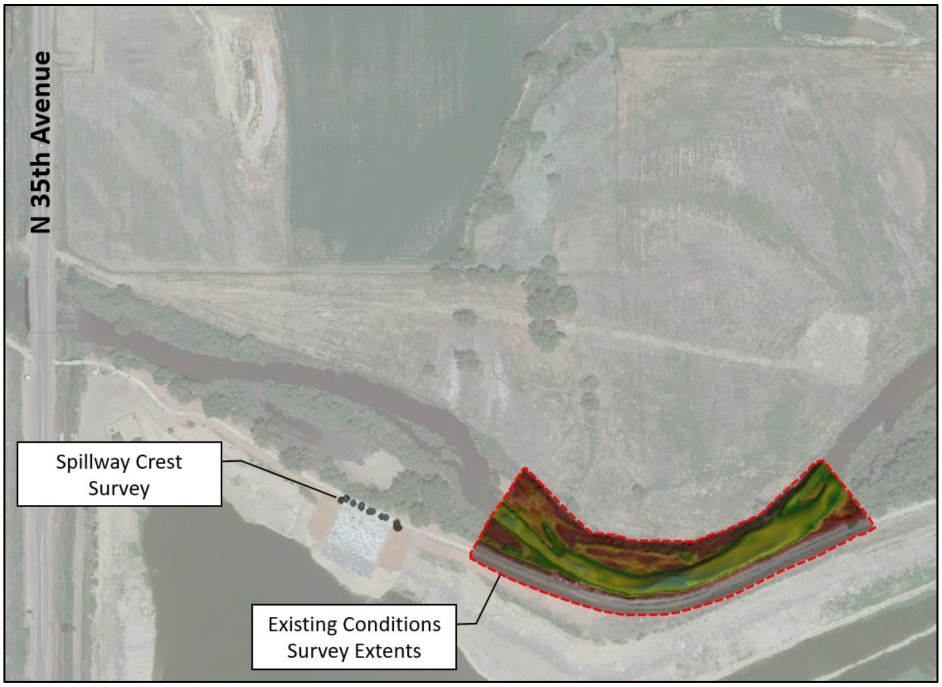

Figure 5: Existing Conditions Survey Extents and Surveyed Spillway Points

2.4. Hydrology

The sources of flows used in the design and floodplain permitting are summarized in Table 2.

Table 2: Design Flow Summary

Recurrence Interval	Flow (cfs)	Source
2-yr	1,752	Estimated from flows included in USACE (2014)
5-yr	2,885	Estimated from flows included in USACE (2014)
10-yr	3,700	Risk MAP
25-yr	5,700	Risk MAP
50-yr	7,800	Risk MAP
100-yr	10,400	Risk MAP
500-yr	11,516	Risk MAP

2.5. Existing Conditions Hydraulics

One-dimensional (1D) steady flow hydraulic modeling of the project area was carried out to provide a sound basis for the hydraulic design of the project and to support the FDP application. The modeling was conducted using the U.S. Army Corps of Engineers (USACE) HEC-RAS v5.0.5 software (USACE, 2018), following the HEC-RAS version that was utilized to develop the Risk MAP hydraulic model. The Risk MAP hydraulic model is made up of a network of multiple reaches that exchange flow in a variety of areas through lateral structures. The Risk MAP hydraulic model, as well as all subsequent hydraulic modeling was completed in the NAD 1983 (2011) State Plane Colorado North FIPS 0501 (US Feet) coordinate system. The vertical datum is NAVD88.

For purposes of evaluating the feasibility of potential concepts (and to support the floodplain development permitting process) the Risk MAP hydraulic model was modified to provide more detail and to represent the hydraulics more accurately through the project reach. Four cross-sections were added to the model

(Figure 6). These four cross-sections, and the middle Risk MAP cross-section were updated with topographic survey to develop an updated hydraulic model to represent Existing Conditions.

Figure 6: HEC-RAS Cross-Sections

The results of the Existing Conditions model show that the average channel velocity through the meander bend is about 4.9 ft/s during the 10-year event. In contrast, the average channel velocity in the downstream meander bend, which has a natural floodplain bench and is stable, is around 2.4 ft/s. Research indicates that velocities along the outside of a meander bend are greater than the channel average. Average channel velocities were used to estimate velocities along the banks using methods outlined in USACE Engineering Manual No. 1110-2-1601[1]. The resulting bank velocities are presented in Figure 7 for the existing project meander (CE Project Meander), the existing downstream meander (CE Downstream Meander) for a range of flows (10-yr to 500-yr).

[1] V_{des} from Equation C2.1 in NCHRP Report 568, Riprap Design Criteria, Recommended Specifications and Quality Control by Lagasse, Clopper, Zevenbergen, and Ruff (Transportation Research Board, 2006).

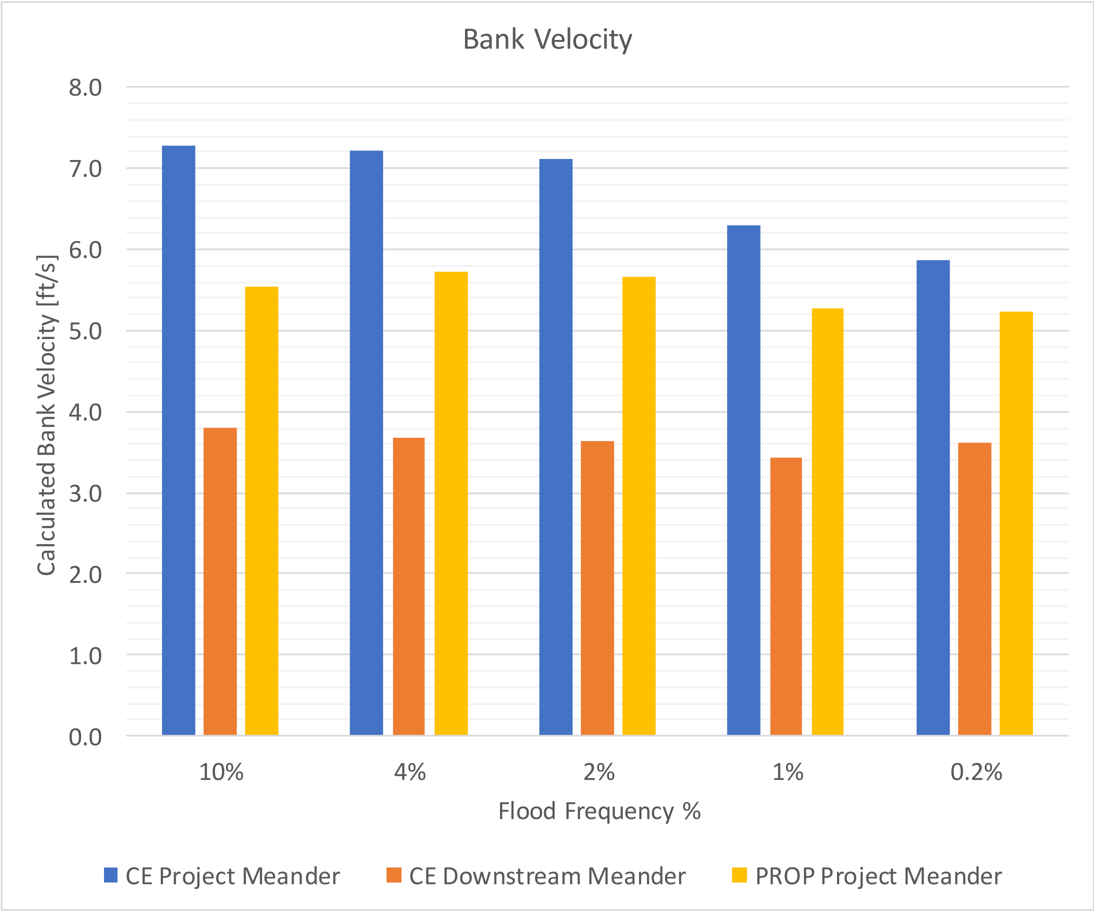

Figure 7: Bank Velocity Estimates

3. Alternatives Assessment

Given bioengineering stabilization methods are more suitable to lower velocities (ideally well below 5 ft/s), the site constraints, and the results presented above indicating the existing project meander may have bank velocities greater than 7 ft/s, harder stabilization treatments will be necessary at this location.

3.1. Bank Stabilization Alternatives

The three options to stabilize the bank within the project area were identified based on site conditions, preliminary hydraulic modeling, and stakeholder feedback. The options treated the full extent of the bend [approximately 500 linear feet (LF)] to avoid the development of weak points along the bend.

3.1.1. Option 1

The first option for stabilization utilizes a combination of imbricated boulder wall and soil wrapped lifts. The 2-year flow is approximately 7.5 feet deep through the project reach. Based on the 1-D model velocities, an up to 6-foot-high imbricated wall would be combined with, 1-foot-high, soil lifts (see concept in Figure 8). Above this elevation the slope would be graded at 3H:1V or flatter to catch grade 2 feet away from the path. The boulder wall could also include large woody material incorporated into the rock protection to provide additional erosion protection and habitat variability. Additionally, with further evaluation, the boulder rows may be able to be set back to allow for plantings within the wall, as well as potentially reducing the extent of the treatment based on refinements to the hydraulic model and site conditions. The success of the plantings will be dependent on how much water they can access. Plantings close to the low water line will have access to groundwater, but those placed higher on the bank may require temporary irrigation for success.

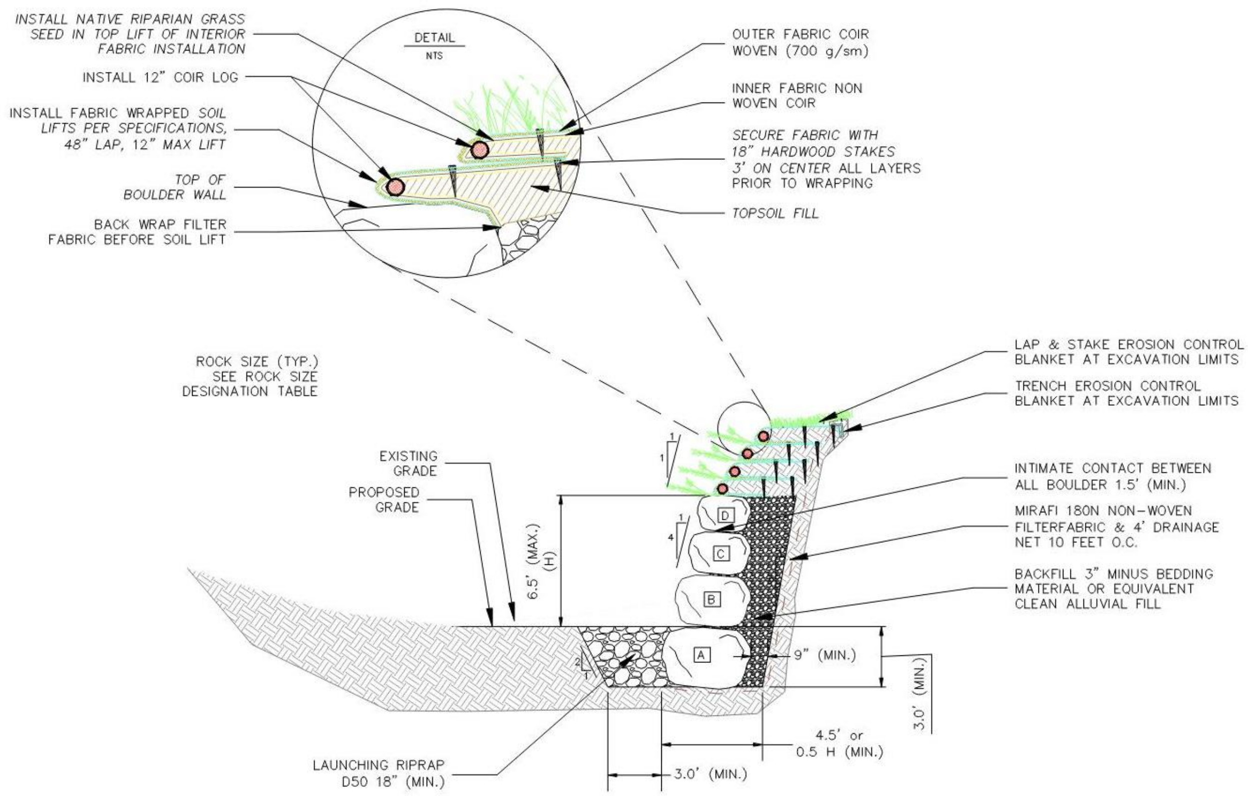

Figure 8: Conceptual Sketch of Boulder Wall Bank Treatment Option

3.1.2. Option 2

This option includes a similar treatment to Option 1 on the right bank, with the addition of matching the floodplain width similar to the width upstream and downstream of the Project area. The floodplain width in the project reach is approximately 120 feet, whereas the floodplain width in the meander bend upstream of the project reach is approximately 270 feet and the floodplain width in the downstream meander is approximately 330 feet (Figure 9).

3.1.3. Option 3

This option involves full riprap treatment of the bend. For purposes of this high-level comparison, it is assumed that riprap will be 24 inches in diameter and be placed at a slope of 1.5(H):1(V).

Otak also considered using rock vanes (a.k.a. stream barbs) to redirect the flow along the bank. Sometimes this approach is less expensive than full revetment of the bank. However, the height of the eroded bank, combined with other constraints (the trail, the slurry wall, etc.), make the installation of rock vanes more expensive and less resilient than other treatments.

Figure 9: Floodplain Widths in the Vicinity of the Project Area

3.2. Hydraulic Analysis

The Existing Conditions model was updated to assess the feasibility and effectiveness of Option 2, widening the floodplain width in the project reach to be consistent with the upstream and downstream bends. Option 3 was not evaluated in the hydraulic model, given the intent was to maintain existing bank lines.

The hydraulic modeling results indicate that widening the floodplain reduces the flow velocity along the bank from 7.3 ft/s to 5.5 ft/s for the 10-year event, and from 6.3 to 5.3 for the 100-year event ("PROP Project Meander" in Figure 7). While these reductions would be a benefit to the stability of the outside bank and to the river function in this area, they do not appear to be large enough to justify a softer approach to the bank treatment.

Furthermore, given the multiple flow paths present in the Risk MAP model for this reach of the Poudre River, any changes to the capacity of the main channel will also cause changes in the flow path through the ponds (to the south). In order to meet a no-rise condition the proposed bank treatment will have to maintain the existing conveyance capacity that can be achieved by not increasing the bank roughness or obstructing flow more than the existing bank. The boulder wall (Option 1) or traditional riprap (Option 3) can be configured to achieve this.

3.3. Recommendations

When evaluating the potential bank stabilization treatments for the project area, the following factors were considered:

1) Constraints of the site associated with the narrow strip of land between the river and the slurry wall and the private land to the north,

2) Risk to adjacent infrastructure (slurry wall/storage pond, Poudre River Trail),

3) Dynamic hydraulics associated with an outside bend of the river, and

4) Improvements to natural environment (e.g., channel, wetlands, riparian corridor).

Given the constraints with modifying the channel and banks in the vicinity of the bend, the existing hydraulics within the river at this bend, and risk to adjacent infrastructure, a hardened solution to stabilize the eroding bank was warranted. The preliminary hydraulics assessment indicated that Option 2, with the addition of a floodplain bench, does not reduce the velocities enough at the bank to warrant the additional cost. Furthermore, while the costs of Option 1 and Option 3 are comparable, there are added ecological components (i.e., willow plantings, seeded floodplain benching) associated with Option 1.

Therefore, given these considerations the recommended alternative to move further in the design process was Option 1, a boulder wall with soil lifts and incorporated woody material and plantings.

4. Bank Stabilization Design

As the design was developed and the associated construction costing updated, it was determined that the boulder wall with soil lifts concept would need to be refined to reduce the overall cost of the project. The resulting modified design includes two walls ("bottom wall" and "middle wall") with narrow planting benches in between (Figure 10). The final design plans can be found in Appendix A.

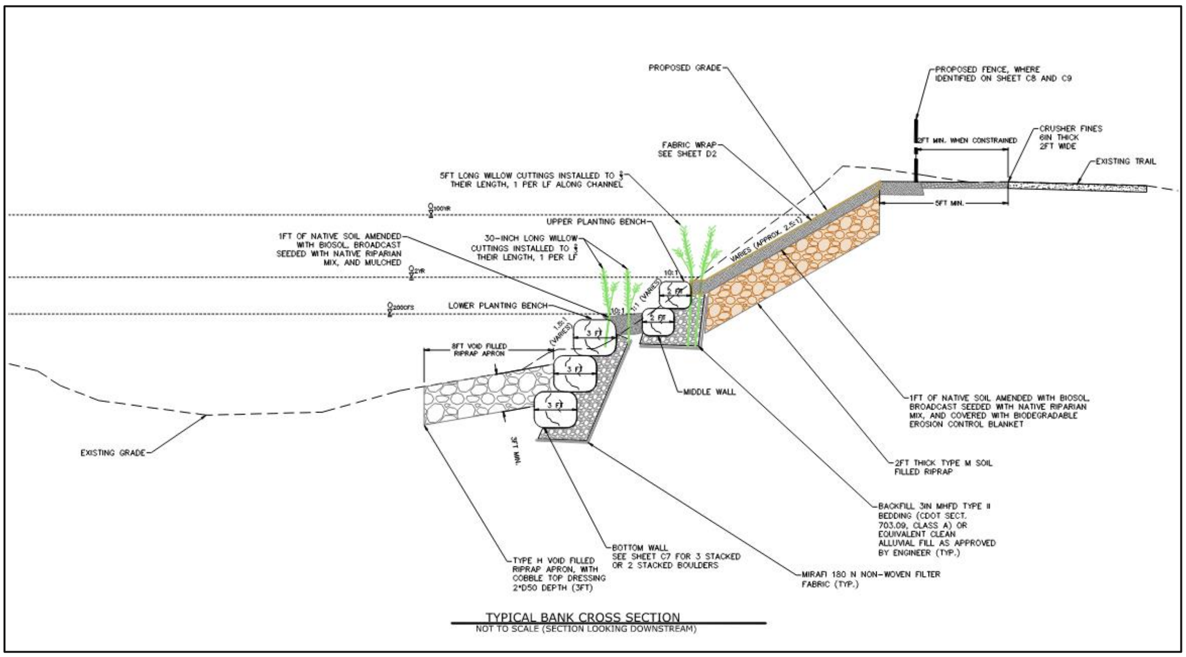

Figure 10: Typical Bank Cross Section

The following sections summarize the design analyses associated with the final boulder wall design.

4.1. Scour Analysis

Bend scour was calculated using Equation 4.5 from HEC-23 (Scour at Protected Bendways; FHWA, 2009). This analysis was performed for the 10-yr, 25-yr, 50-yr and 100-yr discharges. The application of this equation is limited to discharge rates that result in a flow depth on the floodplain that is 20% or less of the flow depth in the main channel. Often this restricts calculations to lower, more frequent discharges, but at this location the 100-year flow event is contained within the channel (on the outside of the bend) due to the confined and incised nature of the Poudre River through this reach. The results of this analysis are presented in Table 3. The negative scour depth values for the 10-, 50-, and 100-year flows indicate that there is no additional scour expected, while the 25-year flow shows a half a foot of expected scour.

Table 3: Bend Scour Calculations and Results

Variable	10-yr Flow	25-yr Flow	50-yr Flow	100-yr Flow
Top width, W (ft)	117.7	117.7	123.0	123.4
Centerline radius of curvature, R_c (ft)	425.0	425.0	425.0	425.0
Initial Depth, D_0 (ft)	11.1	12.1	13.4	15.6
Approach Flow Area, A_1 (ft)	1169.0	1355.1	1374.0	1553.0
Approach Flow Width, W_1 (ft)	190.6	191.9	192.0	193.3
Average approach channel depth $D_{mnc} = A_1/W_1$ (ft)	6.1	7.1	7.2	8.0
R_c/W	3.6	3.6	3.5	3.4
W/D_{mnc}	20.0	20.0	20.0	20.0
D_{mxb}/D_{mnc}	1.8	1.8	1.8	1.8
Depth after scour, D_{mxb} (ft)	10.9	12.6	12.8	14.4
Water surface elevation (ft)	4668.2	4669.2	4669.3	4670.4
Depth of scour $Ds = D_{mxb}-D_0$ (ft)	**-0.2**	**0.5**	**-0.6**	**-1.2**

These results are expected considering the bank of the river is currently armored and has been subject to bendway scour for decades. Based on this analysis the bottom of the lower boulder wall is at a minimum elevation of 4656.5 at the upstream and downstream ends where the wall is only two boulders tall, and a depth of 4653.9 for the approximately 450-foot middle section of the three-boulder-tall wall.

Additional scour protection is provided by an apron of void filled riprap which extends 8 feet out from the wall and is designed as launching riprap. Launching riprap is placed at the surface (in this case flush with existing ground). If additional scour occurs along the toe of the apron the riprap will “launch” into the scour hole. Assuming an angle of reposed of 1:1 this riprap apron should provide protection for up to an additional 5.5 ft of scour depth (elevation 4648.4 for 3-boulder-high and 4651 for the 2-boulder-high wall). The lowest point of the existing channel along the wall length is 4656, therefore the design protects against a total scour of between 5 and 7.6 feet of scour below that elevation.

4.2. Boulder Wall Design

The preliminary design of the boulder wall was based on simplified design relationships developed by FHWA for Guanella Pass Road project in the Pike and Arapaho National Forests in Colorado (FHWA, 2003; FHWA and CFLHD, 2003). Coulomb’s lateral earth pressure theory was then used to finalize the design of the walls. Figure 11, Table 4, and Table 5 show the input variables for the upper and lower walls. The results of this analysis are summarized in Table 6. The minimum factor of safety for overturning and sliding are 5 and 6, respectively.

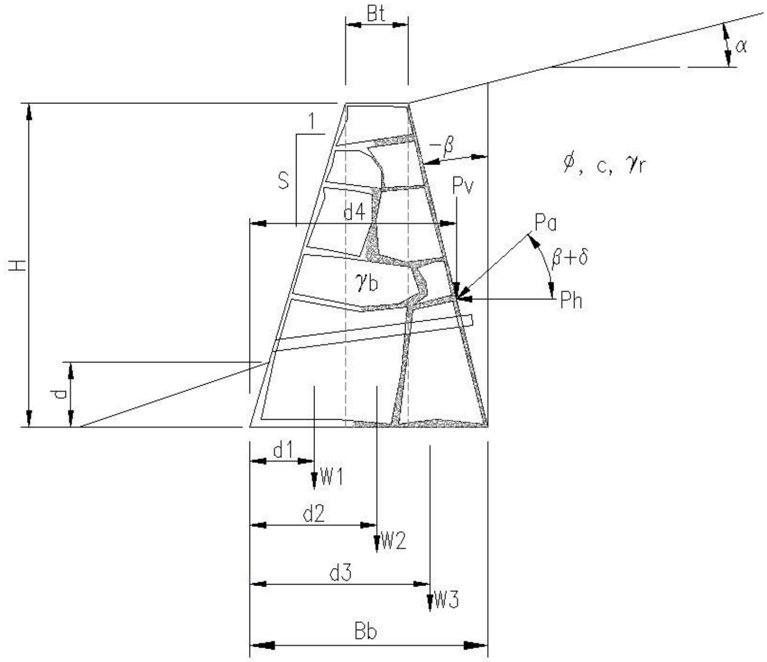

Figure 11: Coulomb's Theory Variable Definition Diagram

Table 4: Wall Design Variables-Upper Wall

Variable	Symbol	Value	Unit	Value	Unit
Wall Height	H	4	ft		
Top Wall Thickness	B_t	2	ft		
Front Face Slope	S_{ff}	1.2	vert : 1 horiz		
Back Face Slope	S_{bf}	-1.2	vert : 1 horiz		
Wall Embedment Depth	d	2	ft		
Bottom Wall Thickness	B_b	2.00	ft	**0.50**	***H**
Surcharge Pressure	q	300	psf		
Allowable Bearing Pressure	Q_a	2000	psf		
Soil Friction Angle	f	34	deg	0.59	rad
Soil Cohesion	c	0	psf		
Soil Backslope Angle	a	1	deg	0.02	rad
Soil Unit Weight	g_r	125	pcf		
Boulder Unit Weight	g_b	150	pcf		
Void Ratio	V	3	%		
Boulder Wall Net Unit Weight	g_w	145.5	pcf		
Wall Backface Batter	b	39	deg	0.68	rad
Angle of Wall Friction	d	23	deg	0.40	rad

Table 5: Wall Design Variables-Lower Wall

Variable	Symbol	Value	Unit	Value	Unit
Lower Tier Offset	X	2	ft	(from back face of lower tier to front face of upper tier)	
Lower Tier Wall Height	H	9	ft		
Lower Tier Top Wall Thickness	B_t	3	ft		
Lower Tier Front Face Slope	S_{ff}	1.1	vert : 1 horiz		
Lower Tier Back Face Slope	S_{bf}	-1.1	vert : 1 horiz		
Lower Tier Wall Embedment Depth	d	3	ft		
Lower Tier Bottom Wall Thickness	B_b	3.00	ft	**0.33**	***H**
Lower Tier Allowable Bearing Pressure	Q_a	2000	psf		
Lower Tier Soil Friction Angle	f	34	deg	0.59	rad
Lower Tier Soil Cohesion	c	0	psf		
Lower Tier Soil Backslope Angle	a	18.43	deg	0.32	rad
Lower Tier Soil Unit Weight	g_r	125	pcf		
Boulder Unit Weight	g_b	150	pcf		
Void Ratio	V	3%			
Boulder Wall Net Unit Weight	g_w	145.5	pcf		
Lower Tier Wall Backface Batter	b	41	deg	0.72	rad
Lower Tier Angle of Wall Friction	d	23	deg	0.40	rad

Table 6: Coulomb's Theory Results

Location/Condition	Overturning Factor of Safety	Sliding Factor of Safety
Upper Wall 4 ft	24	31
Lower Wall 9 ft	5	6
Lower Wall 6 ft	20	16

4.3. Rock Sizing

Three different methods were used to verify the riprap size as follows:

1. Abt & Johnson Equation (Abt, et al., 1988). This method was developed for steep slopes from 2 to 20 percent.
2. Hughes Equation (Hughes, et al., 1983).
3. USACE Steep Slope Riprap Equation (1994). This method is applicable for slopes from 2 to 20 percent.

The results of this analysis are presented in Table 7. Maximum D_{50} ranged from 0.07 to 1.7 ft, depending on the location and discharge used. Based on this analysis it is recommended that the riprap apron be constructed out of Type H Void Filled Riprap (D_{50}= 1.5 ft), and the slope above the walls should be constructed out of Type M Soil Filled Riprap (D_{50}= 1.0 ft).

Table 7: Riprap Size Calculation Results

Location	Riprap Size, D_{50} (ft)		
	Abt & Johnson	Hughes	USACE Steep Slope
50877 10-Year	0.29	0.09	0.76
50877 25-Year	0.33	0.12	0.98
50877 50-Year	0.33	0.12	1.00
50877 100-Year	0.34	0.11	1.22
50877 500-Year	0.37	0.09	1.69
50668 10-Year	0.28	0.09	0.76
50668 25-Year	0.30	0.09	0.98
50668 50-Year	0.30	0.09	1.00
50668 100-Year	0.31	0.07	1.22
50668 500-Year	0.34	0.07	1.69
50488 10-Year	0.32	0.16	0.76
50488 25-Year	0.33	0.13	0.98
50488 50-Year	0.33	0.12	1.00
50488 100-Year	0.32	0.08	1.22
50488 500-Year	0.34	0.07	1.69
MAX	**0.37**	**0.16**	**1.69**
AVG	**0.32**	**0.10**	**1.13**

4.4. Planting Plan

A planting plan was developed by AlpineEco and can be found in Appendix A and summarized in Figure 10. The plan consists of live cuttings (to be collected on-site or nearby) installed on the lower and upper planting benches, and riparian mix seeding.

5. References

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APPENDIX A

Poudre Ponds Bank Stabilization Project Final Design Drawings