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April 5, 2018

Mr. Peter Hays
Colorado Division of Reclamation, Mining and Safety
1313 Sherman St., Room 215
Denver, CO 80203

Re: Response to Comments
City of Greeley's 25th Ave Site (Poudre Ponds)

Dear Mr. Hays,

The City of Greeley (City) and its Consultant, Wenck Associates, Inc. (Wenck) appreciate your comments regarding the City's 25th Avenue Site (Permit No. M-2002-020), commonly referred to as the Poudre Ponds. This letter addresses the Division's (DRMS) comments dated August 30, 2017, on the Poudre Ponds technical revision submitted by the City on July 28, 2017. We thank you for approving our requests for extensions as we worked through some modeling issues that needed to be resolved. The Project Team's responses herein address the request for additional information on our hydraulic analysis for the spillway design and stability criteria of the riprap.

Our responses contain the most recent design analyses inclusive of additional cross sectional surveying along the Cache la Poudre River in the region of the spillway as we sought to address irregularities in the HEC-RAS model. These additional cross sections provided the Project Team with more reliable water surface elevations and velocities to advance into the spillway design.

The Project Team has prepared the following responses to each of the Division's comments:

Comment 1: *The Operator states parameters associated with flooding that lead to erosion (shear stress and velocity) were analyzed for a variety of design storms (2-year through 100-year) and an optimal spillway design was determined. Please provide the Division with a copy of the hydraulic analyses used to design the proposed spillway for review.*

Response to Comment 1:

The 2-year, 5-year, 10-year, 25-year, 50-year and 100-year recurrence intervals were analyzed as part of the proposed design. The 100-year flow was taken from the FEMA regulatory flood insurance study (FIS). The minor recurrence intervals were interpolated from the stream gauge records from the closest stream gauge (United States Geological Survey (USGS) gage 06752500). Consistent with Bulletin 17-B "Guidelines for Determining Flood Flow Frequency" developed by a committee of federal agencies (1976-1982), the minor recurrence interval discharges were calculated using a Log Pearson Type III distribution. The recurrence interval flows calculated and inputted into the project's hydraulic models are presented in **Table 1**.

Table 1: Calculated Flows

Recurrence Interval	Discharge (cfs)	Source
2-YR	1,424	Log P Type III analysis of CLAGRECO data
5-YR	2,734	Log P Type III analysis of CLAGRECO data
10-YR	3,651	Log P Type III analysis of CLAGRECO data
25-YR	4,793	Log P Type III analysis of CLAGRECO data
50-YR	5,608	Log P Type III analysis of CLAGRECO data
100-YR	11,700	Flood Insurance Study (FIS)

The project hydraulic modeling is built on the regulatory FIS's effective hydraulic (HEC-RAS) model. That model was updated and calibrated to create an existing conditions model, in order to incorporate updated topography and match historical observations on flows that overtop the riverbank into Poudre Ponds. The existing condition model results were used to design the spillway and determine the elevation of the proposed spillway.

A proposed conditions model was created by modifying the existing conditions model to reflect the proposed structure's topography. **Attachment A1** provides a table of the model predicted output from the hydraulic analysis at the location of the proposed inflow point (model cross-sections 51171, 51151, 51101, and 51081) for various recurrence intervals (2-year, 5-year, 10-year, 25-year, 50-year and the 100-year). **Figure 1** shows the location of the cross-sections in the table, which are located both upstream and downstream of the proposed spillway.

Consistent with FEMA 100-year floodplain regulations, uncertified levees are not incorporated into the 100-year analysis. Instead, flow is routed through the lowest points in each respective cross section. For the minor recurrence intervals, as allowed by FEMA floodplain regulations, levees were incorporated into the hydraulic model. This forces water to stay in the main channel causing increased water surface elevations and velocities, which reflects a conservative worst-case analysis for those recurrence intervals. Because of this difference, the increase in water surface elevations between the 50- and 100-year events are minimal or even negative in some locations.

To address the anticipated flow through the spillway during a 100-year flood event within the main channel, Wenck used the approach velocity method (Sturm, 2010). The basis of this method involves a broad-crested weir calculation without assuming a velocity coefficient of 1. By inserting variable approach velocities, a more accurate discharge through the weir can be calculated. The approach velocity at the spillway, typically, will not exceed the main channel velocities. This is due to the fact that the spillway flow direction will be perpendicular of the main channel flows. Rather than reduce the velocity for the analysis, we took the conservative approach of completing the analysis assuming the spillway approach velocity was equal to the highest predicted average channel velocity for the 100-year event of 13.25 fps (See HEC-RAS results table, Attachment A1). Because HEC-RAS is a one-dimensional, cross-sectionally averaged model, the average channel velocity is calculated. Therefore, the velocity in the thalweg is likely greater than 13.25 fps, and conversely, the velocity along the banks is likely less than 13.25 fps. While the perpendicular flow velocity will also be less than 13.25 fps, this value was used as the approach velocity to the spillway to insure a conservative design. **Attachment A2** displays the approximate discharges for a range of approach velocities. For our analysis, we evaluated the proposed spillway assuming a discharge of 933 cfs based on the 100-year in-channel velocity (13.25 fps) and assuming a conservative head of 2.5-feet. With the proposed addition of fill surrounding the spillway, the total head above the structure invert elevation will be approximately 2.5 feet for the 50-year event (the highest water surface elevation per the HEC-RAS model). This discharge through the spillway will result in a normal depth in the chute of 1.75 feet, resulting in approximately 0.15 feet of freeboard.

The design of the Poudre Ponds spillway extends below the ordinary high water level (OWHL) (4,665-ft per the 1988 North American Vertical Datum, NAVD88) of Poudre Ponds and consists of two portions of riprap (see the attached Sheet No. C-102). Grouted riprap (median grain size = d_{50} = 24-in.) extends from the invert of the spillway to the elevation in which the Pond is at two-thirds capacity (El. = 4,657-ft NAVD 88). After that point, graded riprap (d_{50} = 24-in.) will extend to the minimum pond elevation of 4,649 feet (NAVD 88).

Poudre ponds is managed as a "savings account" of water meaning that it is operated to remain as full as possible. Under normal conditions, the water in the Pond will dissipate energy as the spillway is activated. The graded riprap (d_{50} = 24-in.) portion of the spillway will become active only in the event the water surface elevation drops below El. = 4,657 feet (NAVD 88). Additionally, the minimum pond elevation of 4,649 feet (NAVD 88) was set through an agreement with Colorado Parks and Wildlife as the minimum water surface elevation (WSE) to sustain fish populations. The Pond cannot be lowered below 4,649 feet (NAVD 88).

Bentley FlowMaster V8i software was used for the hydraulic analysis for the spillway design. **Attachments A3** and **A4** are print outs of the inputs and results, for the grouted riprap and graded riprap portions of the proposed spillway. For the normal expected hydraulic conditions (933 cfs), shown in **Table 2**, in which the lower portion the spillway is under water, the depth in the spillway will be approximately 1.75 feet deep, as shown in the spillway section view presented in **Figure 2**. This assumes the addition of fill surrounding the spillway to 4,672 feet (NAVD 88). The velocity and shear stresses are 6.26 fps and 11.61 lb/ft^2, respectively, and within the acceptable range of the maximum permissible velocity (15-18 fps) and maximum shear stress (10-12 lb/ft^2) allowed by the Colorado Water Conservancy Board (CWCB) and Natural Resources Conservation Service (NRCS). These design criteria from CWCB and NRCS are presented in the response to Comment 3.

Table 2: Hydraulic Analysis Summary

	Discharge (cfs)	Side Slopes (_H:1V)	Bottom Width (ft)	Slope (%)	Flow Area (ft^2)	Maximum Velocity in Chute (ft/s)[1]	Normal Depth (ft)	Froude	Shear Stress (lb/ft2)
Greatest Expected Discharge based on Size of Weir - With Fill	933	20	50	0.15	148.9	6.26	1.75	0.99	11.61

1. Assumes Grouted Riprap spillway with Manning's "n" = 0.106

2. Spillway Invert = 4669.5

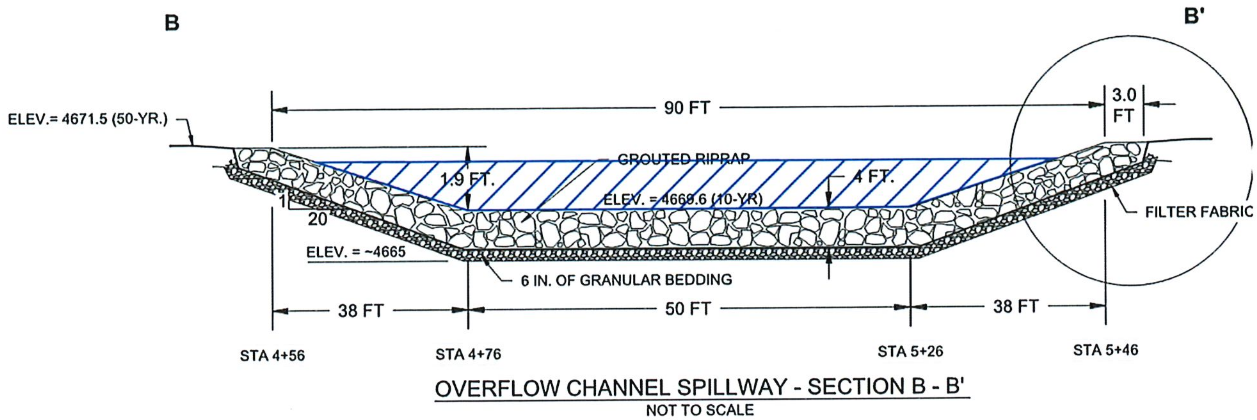

Figure 2: Schematic of spillway water depth when flow is 933-cfs (normal conditions)

During conditions in which the graded riprap section of the spillway is above water and is active, velocity and shear stress values would reach 6.35 fps and 11.51 lb/ft^2 respectively in that section. Both values are lower than the 9-10 fps maximum permissible velocity and less than the 14.0 lb/ft^2 maximum shear stress allowed by NRCS for graded riprap. The depth in this section of the spillway will be approximately 1.73-ft, resulting in approximately 0.17 feet of freeboard.

The riprap was sized using the results of the greatest expected hydraulic conditions presented in **Attachment A2**. Spillway hydraulic parameters that govern riprap sizing are dependent on the roughness of the riprap which varies by size, as such, an iterative approach was necessary.

- First, published values of Manning's roughness coefficient for grouted riprap and riprap with a $d_{50} \geq$ 12-in were assumed (n = 0.030 and n= 0.078, respectively). The respective unit flow values from the 933 cfs maximum flow within the respective spillway portions were used as input in the CSU Equation (Abt, et al, 1988) as suggested in the Urban Drainage Flood Control District's (UDFCD) "Urban Storm Drainage Criteria Manual Volume 1," Chapter 8, for steep slope conditions and angular stone. Using a safety factor of 1.5, this yielded a d_{50} of 1.94-ft and 2.04-ft for the grouted and graded riprap respectively.

- Assuming a d_{50} = 2.00-ft, Manning's roughness coefficient was calculated for the grouted riprap using the UDFCD "Urban Storm Drainage Criteria Manual Volume 2," Chapter 9 equation for grouted boulders with the upper one-third of the rock height left ungrouted. This equation uses depth of flow and rock diameter as inputs. The normal depth in the channel with n = 0.03 (0.90-ft) was used initially to update the roughness coefficient, which was then used as input in the open channel calculations to calculate a new normal depth. This approach was repeated iteratively to convergence with n = 0.106 and flow depth = 1.75 for the grouted riprap spillway.

- With a grouted riprap roughness coefficient (n= 0.106) now greater than that assumed for the graded riprap section, a similar technique was used to update the graded roughness coefficient, but using the equation with the upper one-half of the rock height left ungrouted. A similar iterative approach was used and converged with n = 0.120 and flow depth = 1.73 for the graded riprap spillway.

- With updated roughness coefficients and respective open channel hydraulic parameters, the CSU Equation for steep slope riprap size was re-calculated to confirm the riprap sizing. The increased channel roughness resulted in increased normal depths and therefore decreased unit discharges in the grouted and graded riprap channels, which effectively decreased the d_{50} to 1.71-ft and 1.94-ft respectively.

From the above analysis, with a safety factor of 1.5 and maximum assumed discharge in the spillway, the d_{50} was selected to be 24-inches after rounding up to the nearest 0.50-ft for both the grouted and graded riprap spillway sections.

Attachment A5 presents the range of shear stresses that could occur if the flow of 933 cfs were channelized across a smaller area of the grouted riprap section of the spillway. This could occur if there were debris accumulation blocking the full width of the spillway. The velocity and shear stresses increase to 6.56 fps and 12.45 lb/ft^2, respectively. While lower than the 15-18 fps maximum permissible velocity, the 10-12 lb/ft^2 maximum shear stress is exceeded for bottom widths between 5-ft to 30-ft. **Attachment A5** also shows for these channel widths, the channel depth is exceeded, thus these shear stresses in exceedance of the design criterion are unlikely to actually occur. Conversely, for bottom widths of 35-ft and above, the 933 cfs is fully contained within the channel and the shear stresses are within the acceptable range.

Comment 2: *The Operator states to confirm the stability of the design a worst-case scenario was determined to analyze the stability of the structure versus extreme velocities and shear stress. Please provide the Division with a copy of the worst-case scenario hydraulic analyses used to design the proposed spillway for review.*

Response to Comment 2:

The worst-case scenario was determined based on the greatest expected discharge that the spillway would receive assuming the Cachle la Poudre River 100 year in-channel velocity of 13.25 fps were directed into the spillway crest. This condition is shown in **Figure 2.** A hydraulic analysis of this scenario on the grouted riprap portion of the spillway was completed using Bentley FlowMaster V8i. The inputs and results of the analysis are summarized in **Table 2** and included in **Attachment A3**.

This greatest expected discharge is estimated to be 933 cfs. By creating this scenario, extreme velocities and shear stresses could be calculated along the spillway. The contingency involved with the larger (d_{50}= 24-inch) rock size required under normal (100-year and smaller) recurrence intervals as well as the incorporation of the grout provides suitable stability to prevent significant damage to the structure during these worst-case scenarios.

The calculated velocity and shear stresses in the grouted riprap portion of the spillway during this worst-case scenario are 6.26 fps and 11.61 lb/ft^2, respectively. Both values are within the 15-18 fps maximum permissible velocity and 10-12 lb/ft^2 maximum permissible shear stress allowable for a grouted riprap spillway. These design criteria are presented in the response to Comment 3.

During an event as large as the worst-case scenario, the spillway would be activated prior to the peak of the hydrograph and before the peak flow arrives. This would result in an increase in the Pond water surface elevations above the normal operating water surface elevation of the Pond. In this case, the Pond would act as a plunge pool, providing energy dissipation for the extreme hydraulic conditions, and the graded riprap portion of the spillway would be underwater. Though the graded riprap portion of the spillway was analyzed under this worst-case scenario, it is unlikely ever to receive it.

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Comment 3: *The Operator states the proposed grouted riprap design meets the stability criteria as determined by the Natural Resources Conservation Service and Colorado Water Conservation Board. The Division typically requires Operators to comply with the design criteria from the Urban Drainage and Flood Control District. Please provide the Division with the design criteria for the Natural Resources Conservation Service and Colorado Water Conservation Board for review. Please demonstrate how the proposed spillway design is designed in accordance with the Urban Drainage and Flood Control District criteria.*

Response to Comment 3:

We appreciate the additional explanation we received from Peter Hays via email on Sept. 5, 2017 regarding this comment. In that email, Peter explained that the Division does not have drainage criteria in its regulations, and that the Urban Drainage and Flood Control District's "Technical Review Guidelines for Gravel Mining and Water Storage Actives Within or Adjacent to 100-year Floodplains" (TRG) is used by the Division as a reference.

Wenck reviewed the TRG and have found the proposed design for the inflow point to be consistent with the TRG in most respects. For example, the TRG specifies (Figure 2.7) that for grouted sloping boulder slope protection for the proposed spillway slopes that the UDFCD drop structure design criteria be used to determine the minimum boulder size. Using this methodology and the spillway velocities obtained using the roughness coefficients of n = 0.106 and n = 0.120 (grouted and graded riprap respectively), the calculated rock-sizing parameters, R_p, were 3.26 and 3.47 respectively, which both correspond to a nominal size of 18-in. This is inline with the d_{50}= 24-inch size chosen for both the grouted and graded spillway riprap. Either Urban Drainage riprap designation Type VH, or CDOT riprap class 0.5 ton gradations will be accepted specified for construction.

Additionally, the attached spillway detail (Sheet No. C-102) has been updated to include additional TRG guidance to reduce the grout thickness to two-thirds of the total riprap thickness consistent with the grouted boulders material specifications outlined in Figure 9-15 of the UDFCD Criteria Manual Volume 2. The design has also been updated to use concrete cutoffs as identified in Figure 2.8 of the TRG.

In using the Colorado Water Conservancy Board (CWCB) and Natural Resources Conservation Service (NRCS) standards, the Project Team took a conservative approach to the design of the spillway. The CWCB and NRCS design criteria referenced are the maximum permissible velocities and shear stresses of grouted riprap and graded riprap as published in Table CH13-T103 of the CWCB's Floodplain and Stormwater Criteria Manual (provided as **Attachment B1**) and the NRCS Field Office Technical Guide (FOTG) for Streambank and Shoreline Protection Code 580 (provided as **Attachment B2**), the proposed spillway design was analyzed to ensure these recommended maximum values would not be exceeded.

For the grouted section, the maximum permissible velocity of 18.0 fps and shear stress of 12.0 lb/ft^2 (**Attachments B1 and B2**) exceed the expected values during events up to the 100-year event. The same can be said for the graded riprap, as the maximum permissible velocity of 9-10 fps and shear stress of 14.0 lb/ft^2 also exceed the expected values during events up to the 100-year event. As such, since the worst-case scenario far exceeds the normal expected flows, the approach to the shear stress analysis is conservative. The expected shear stresses will not exceed the maximum permissible shear stress of grouted or graded riprap even during a worst-case scenario.

The City and Wenck are both committed to ensuring the proposed project meets the requirements of the Division. If you have any questions or comments, please do not hesitate to contact Pamela Massaro (Wenck) at (970) 223-4705 or Daniel Moore (City of Greeley) at (970) 350-9814.

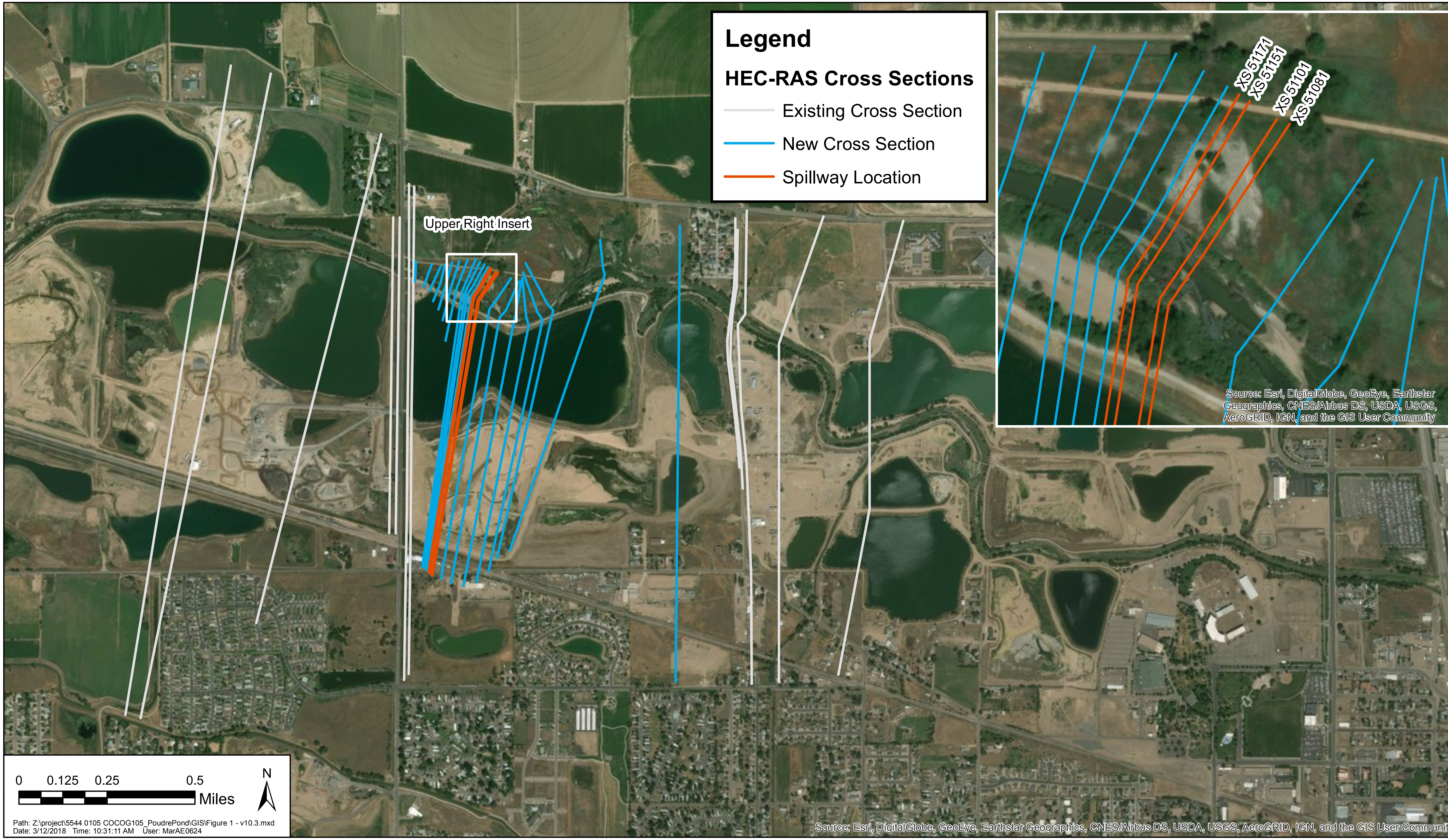
Sincerely,

Pamela A.K. Massaro, P.E.
Wenck Associates, Inc. – Water Resources Engineer

References

Abt, S.R., et al. (1988). Development of riprap design criteria by riprap testing in flumes: Phase II, Followup investigations. United States.

Sturm, T.W. (2010) Open Channel Hydraulics, 2nd edition, McGraw-Hill, New York.



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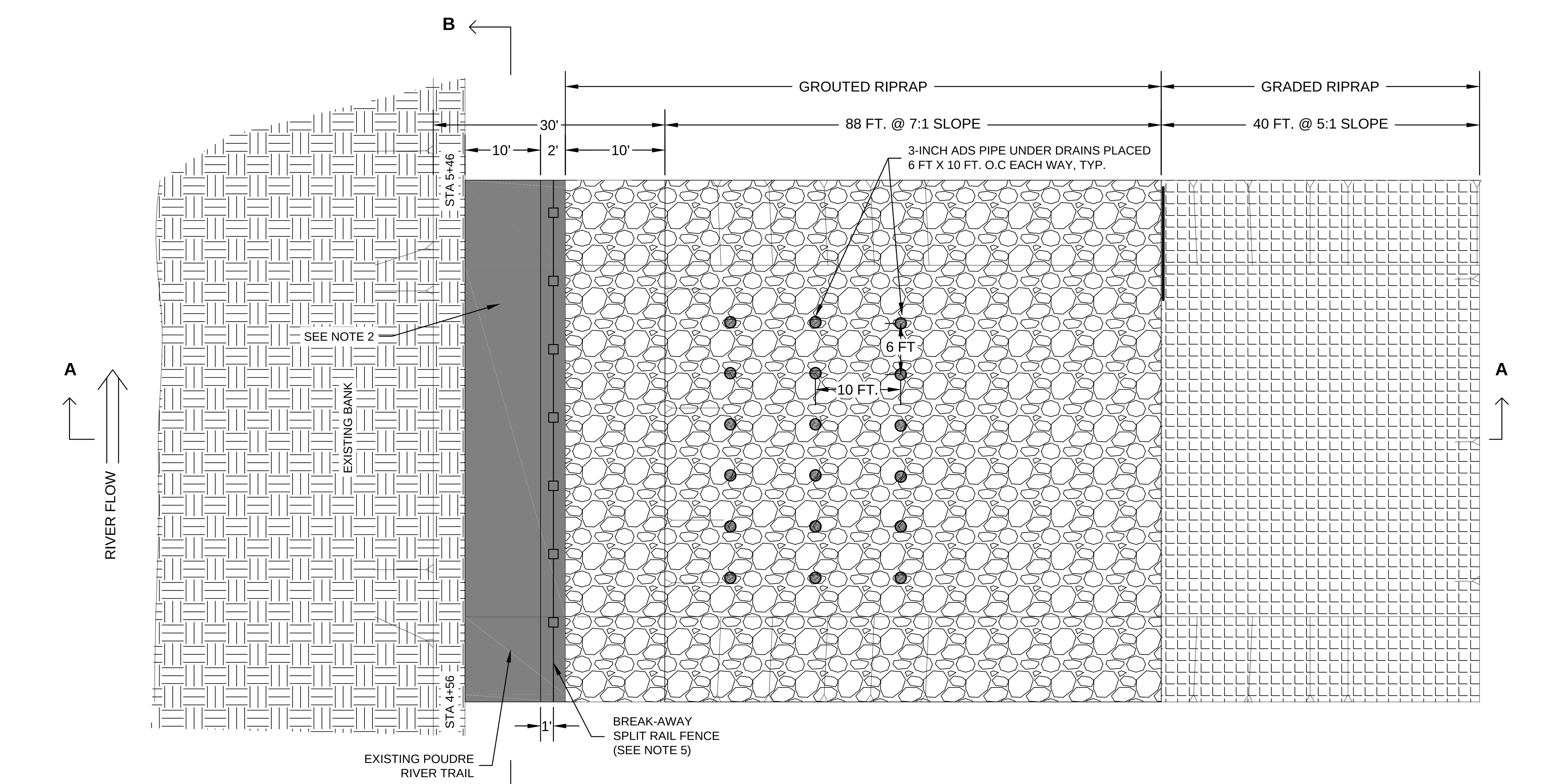
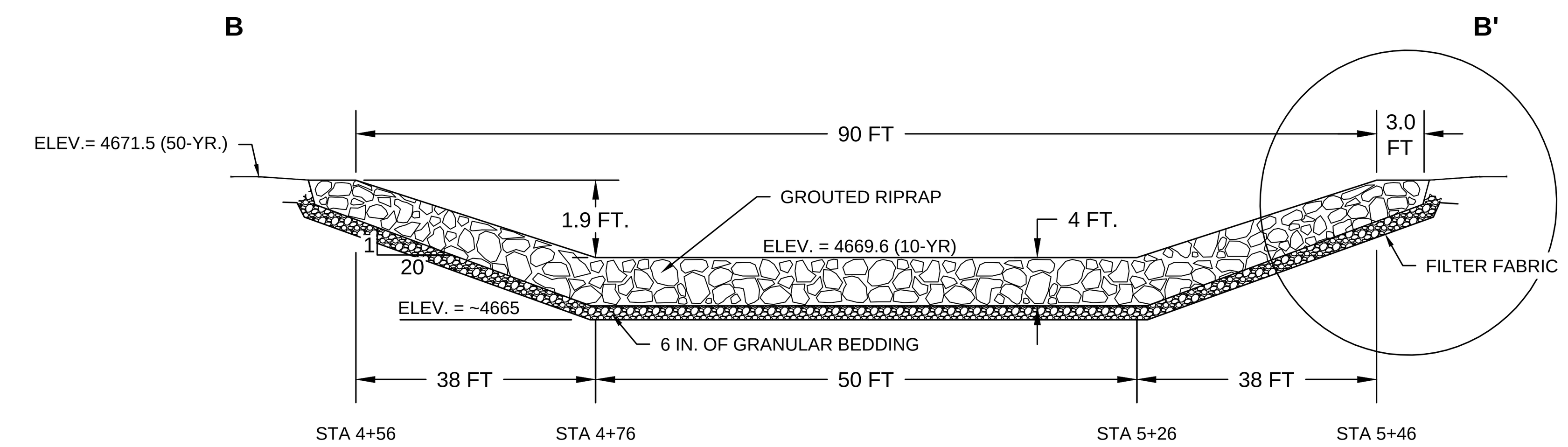
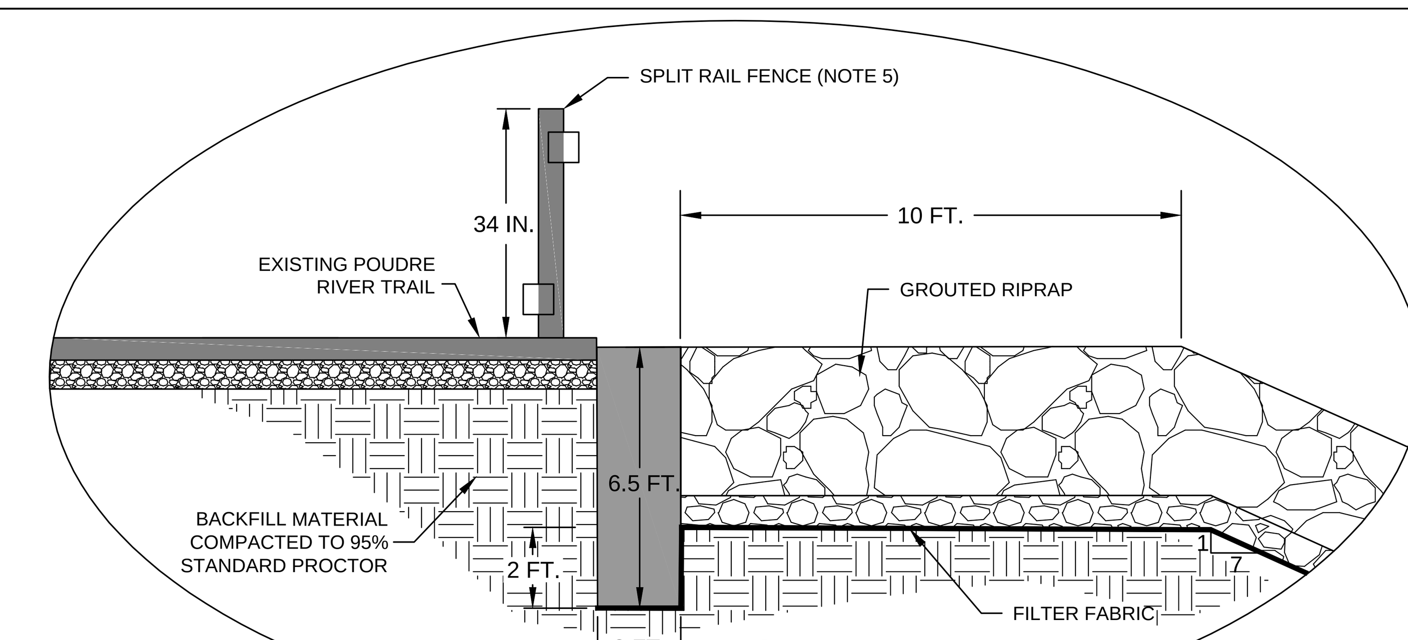
HEC-RAS Cross Section Locations		Figure 1



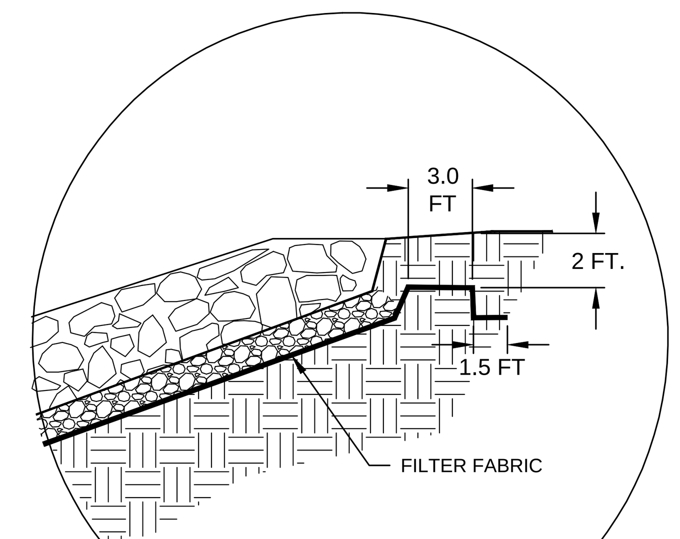
OVERFLOW CHANNEL SPILLWAY - PLAN VIEW
NOT TO SCALE



OVERFLOW CHANNEL SPILLWAY - SECTION B - B'
NOT TO SCALE



INSET A
NOT TO SCALE



INSET B
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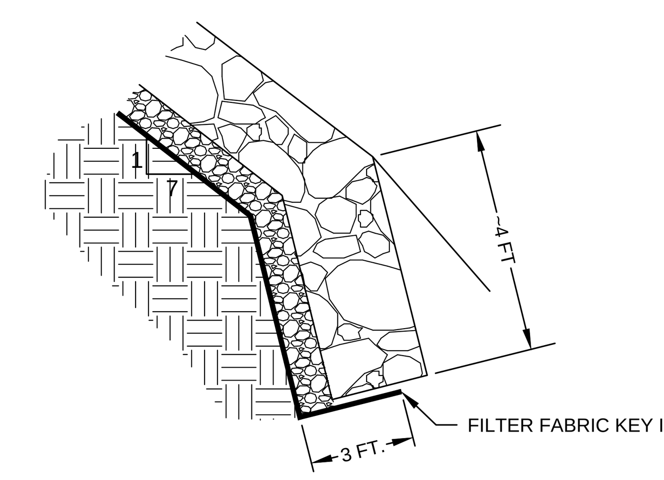
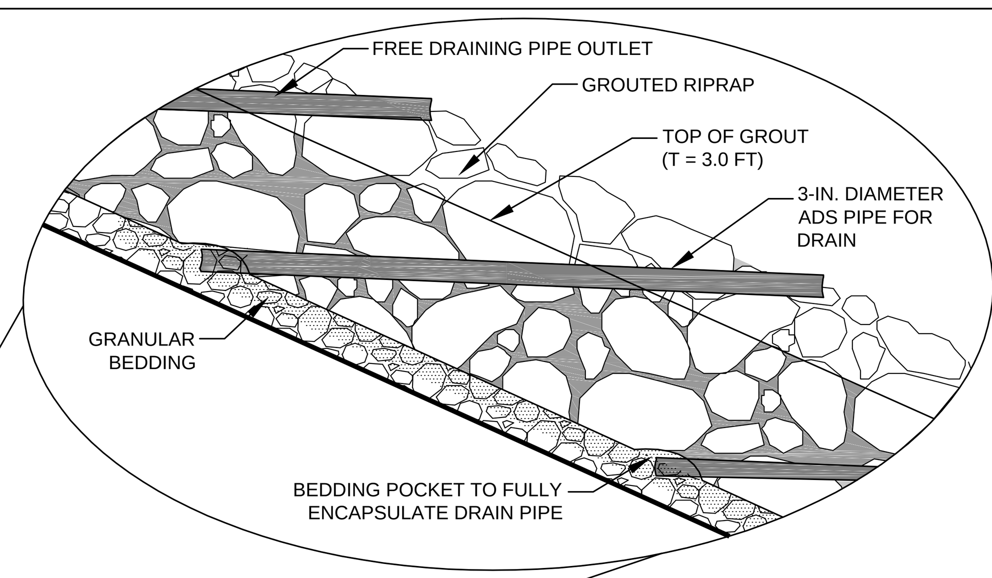
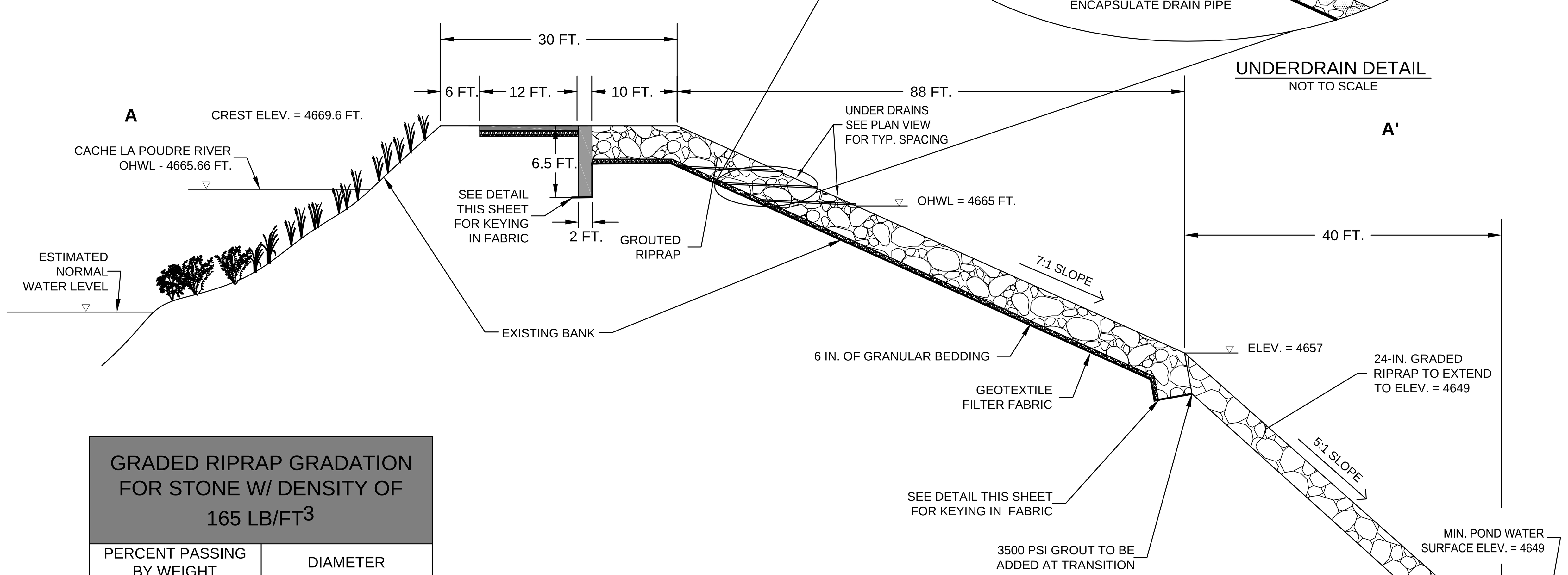


KEY IN/ANCHOR DETAIL
NOT TO SCALE



UNDERDRAIN DETAIL
NOT TO SCALE



OVERFLOW CHANNEL SPILLWAY - SECTION A-A'
NOT TO SCALE

GRADED RIPRAP GRADATION FOR STONE W/ DENSITY OF 165 LB/FT^3

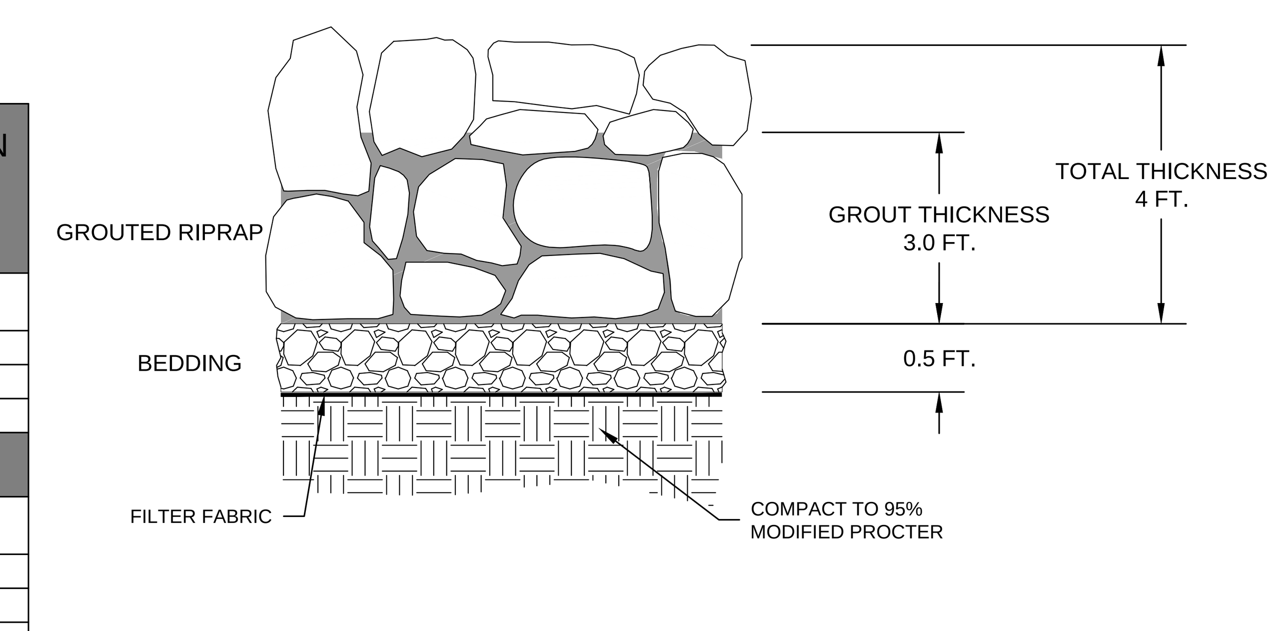
PERCENT PASSING BY WEIGHT	DIAMETER
95%	42 IN.
50%	24 IN.
10%	6 IN.

GROUTED RIPRAP GRADATION FOR STONE W/ DENSITY OF 165 LB/FT^3

PERCENT PASSING BY WEIGHT	DIAMETER
95%	42 IN.
50%	24 IN.
10%	9 IN.

BEDDING

PERCENT PASSING BY WEIGHT	SIZE
100%	1 IN.
85%	.75 IN.
50%	.25 IN.
15%	NO. 10
5%	NO. 200



* URBAN DRAINAGE RIPRAP DESIGNATION TYPE VH
CDOT RIPRAP CLASS 0.5 TON

GROUTED RIPRAP DETAIL
NOT TO SCALE

GENERAL NOTES:

1. WARNING SIGNS SHALL BE POSTED AT THE PROJECT LOCATION PRIOR TO CONSTRUCTION.
2. THE EXISTING POUDRE RIVER TRAIL IN THE VICINITY OF THE PROJECT AREA SHALL BE REMOVED PRIOR TO CONSTRUCTION. UPON COMPLETION OF THE SPILL STRUCTURE, THE TRAIL SHALL BE REPLACED AS SPECIFIED IN THE SPECIFICATIONS.
3. THE TOE OF THE SPILLWAY SIDE SLOPES SHALL BE ROUNDED ON THE TRAIL FOR BIKING/PEDESTRIANS.
4. AFTER CONSTRUCTION, THE REMOVED TRAIL SECTION SHALL BE REPLACED WITH FIBER-MESH REINFORCED CONCRETE.
5. A TWO-RAIL SPLIT RAIL FENCE FOLLOWING CITY OF GREELEY'S STANDARDS WILL BE CONSTRUCTED ALONG THE REPACED TRAIL SECTION PER THE PROJECT SPECIFICATIONS.

ELEVATION DATUM: NAVD 88

REV	REVISION DESCRIPTION	DWN	APP	REV DATE

CLIENT
CITY OF GREELEY
WATER AND SEWER DEPARTMENT

PRIME CONSULTANT
Lidstone and Associates, Inc.
WENCK
Lidstone and Associates - A Wenck Company

SUB CONSULTANT

SEAL

PROJECT TITLE
POUDRE PONDS SCOUR PROTECTION PROJECT

SHEET TITLE
GROUTED RIPRAP SPILLWAY DETAIL

DWN BY	CHK'D	APP'D	DWG DATE	SCALE
DJW	ZSB	PAKM	3/12/18	

PROJECT NO.	SHEET NO.	REV NO.
COCOG105	C-501	

Attachment A

Attachment A1: Main Channel HEC-RAS Model Results

River Sta	Profile	Q Total	Existing Condition				Proposed Condition				Proposed minus Existing**			
		Q Total	W.S. Elev	E.G. Elev	Vel Chnl	Froude # Chl	W.S. Elev	E.G. Elev	Vel Chnl	Froude # Chl	W.S. Elev	E.G. Elev	Vel Chnl	Froude # Chl
		CFS	FT	FT	FT/Sec	-	FT	FT	FT/Sec	-	FT	FT	FT/Sec	-
51171	2	1424	4666.79	4666.92	3.11	0.27	4666.79	4666.92	3.11	0.27	0.00	0.00	0.00	0.00
51171	5	2734	4668.63	4668.79	3.51	0.26	4668.63	4668.79	3.50	0.26	0.00	0.00	-0.01	0.00
51171	10	3651	4669.66	4669.83	3.76	0.26	4669.66	4669.83	3.75	0.26	0.00	0.00	-0.01	0.00
51171	25	4793	4670.73	4670.92	4.04	0.26	4670.73	4670.92	4.03	0.26	0.00	0.00	-0.01	0.00
51171	50	5608	4671.31	4671.52	4.26	0.26	4671.31	4671.52	4.26	0.26	0.00	0.00	0.00	0.00
51171	100 (FIS)	11700	4671.05	4672.07	9.32	0.58	4671.03	4672.06	9.32	0.58	-0.02	-0.01	0.00	0.00
51151	2	1424	4666.76	4666.90	3.13	0.27	4666.76	4666.90	3.13	0.27	0.00	0.00	0.00	0.00
51151	5	2734	4668.61	4668.76	3.53	0.26	4668.61	4668.76	3.54	0.26	0.00	0.00	0.01	0.00
51151	10	3651	4669.63	4669.81	3.77	0.26	4669.63	4669.81	3.77	0.26	0.00	0.00	0.00	0.00
51151	25	4793	4670.71	4670.90	4.03	0.26	4670.71	4670.90	3.99	0.25	0.00	0.00	-0.04	-0.01
51151	50	5608	4671.29	4671.50	4.25	0.26	4671.29	4671.50	4.20	0.26	0.00	0.00	-0.05	0.00
51151	100 (FIS)	11700	4670.91	4671.97	9.49	0.60	4670.92	4671.95	9.37	0.59	0.01	-0.02	-0.12	-0.01
51101	2	1424	4666.68	4666.83	3.21	0.28	4666.68	4666.83	3.21	0.28	0.00	0.00	0.00	0.00
51101	5	2734	4668.54	4668.71	3.57	0.27	4668.55	4668.71	3.56	0.27	0.01	0.00	-0.01	0.00
51101	10	3651	4669.58	4669.75	3.80	0.26	4669.58	4669.75	3.78	0.26	0.00	0.00	-0.02	0.00
51101	25	4793	4670.65	4670.85	4.06	0.26	4670.66	4670.85	4.00	0.26	0.01	0.00	-0.06	0.00
51101	50	5608	4671.24	4671.45	4.27	0.26	4671.24	4671.45	4.21	0.26	0.00	0.00	-0.06	0.00
51101	100 (FIS)	11700	4670.26	4671.63	10.66	0.70	4670.35	4671.63	10.38	0.68	0.09	0.00	-0.28	-0.02
51081	2	1424	4666.66	4666.80	3.20	0.28	4666.66	4666.80	3.20	0.28	0.00	0.00	0.00	0.00
51081	5	2734	4668.52	4668.68	3.58	0.27	4668.52	4668.68	3.58	0.27	0.00	0.00	0.00	0.00
51081	10	3651	4669.55	4669.73	3.82	0.26	4669.55	4669.73	3.82	0.26	0.00	0.00	0.00	0.00
51081	25	4793	4670.62	4670.83	4.14	0.26	4670.63	4670.83	4.09	0.26	0.01	0.00	-0.05	0.00
51081	50	5608	4671.21	4671.43	4.35	0.27	4671.21	4671.43	4.32	0.27	0.00	0.00	-0.03	0.00
51081	100 (FIS)	11700	4669.16	4671.35	13.25	0.94	4669.16	4671.35	13.25	0.94	0.00	0.00	0.00	0.00

*All Elevations listed in the NAVD 88 Datum

**A positive value in the Proposed minus Existing columns indicates that the proposed condition value is higher; conversely, a negative value indicates that the proposed condition value is lower

Note: Yellow highlight indicates that the model results include levees

Note: No highlight indicates that the model results do not include levees in compliance with FEMA floodplain standards.

Appendix 2: Approach Velocity Sensitivity Analysis

Spillway Width, b	50	ft
Crest Breadth (L)	30	ft
Crest Invert	4669.5	
HW Elev.	4670.5	
Total Head, H	2.5	ft
Discharge Coefficient, Cd	0.93833	
Gravitational Acceleration	32.2	ft/s^2

$$Q = C_v C_d \frac{2}{3}\left[\frac{2g}{3}\right]^{1/2} bH^{1.5}$$

Where:

$$C_v = \left(\frac{H + {v_0^2}/{2g}}{H}\right)^{2/3}$$

$$C_d = 0.93 + 0.1\left(\frac{H}{L}\right)$$

Note: Equation assumes rectangular, broad-crested weir

Approach Velocity (ft/s)	Velocity Coefficient, C_v	Discharge Coefficient, C_d	Discharge, Q (cfs)
10.50	1.41	0.938	808.66
11.00	1.45	0.938	829.67
11.50	1.49	0.938	851.37
12.00	1.52	0.938	873.73
12.50	1.56	0.938	896.74
13.00	1.61	0.938	920.36
13.25	1.63	0.938	932.40
13.50	1.65	0.938	944.59
14.00	1.69	0.938	969.40
14.50	1.74	0.938	994.76
15.00	1.78	0.938	1020.68
15.50	1.83	0.938	1047.11
16.00	1.87	0.938	1074.06

Attachment A3: Grouted Riprap Portion Hydraulics

Spillway Structure - Grouted Riprap

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.106	
Channel Slope		0.15000	ft/ft
Left Side Slope		20.00	ft/ft (H:V)
Right Side Slope		20.00	ft/ft (H:V)
Bottom Width		50.00	ft
Discharge		933.00	ft³/s

Results

Normal Depth		1.75	ft
Flow Area		148.93	ft²
Wetted Perimeter		120.15	ft
Hydraulic Radius		1.24	ft
Top Width		120.06	ft
Critical Depth		1.74	ft
Critical Slope		0.15272	ft/ft
Velocity		6.26	ft/s
Velocity Head		0.61	ft
Specific Energy		2.36	ft
Froude Number		0.99	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		0.00	ft
Length		0.00	ft
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	ft
Profile Description			
Profile Headloss		0.00	ft
Downstream Velocity		Infinity	ft/s
Upstream Velocity		Infinity	ft/s
Normal Depth		1.75	ft
Critical Depth		1.74	ft
Channel Slope		0.15000	ft/ft

Spillway Structure - Grouted Riprap

GVF Output Data

Critical Slope	0.15272	ft/ft

Attachment A4: Graded Riprap Portion Hydraulics

Spillway Structure - Graded Riprap

Project Description

Friction Method	Manning Formula	
Solve For	Normal Depth	

Input Data

Roughness Coefficient	0.120	
Channel Slope	0.20000	ft/ft
Left Side Slope	20.00	ft/ft (H:V)
Right Side Slope	20.00	ft/ft (H:V)
Bottom Width	50.00	ft
Discharge	933.00	ft^3/s

Results

Normal Depth	1.73	ft
Flow Area	146.85	ft^2
Wetted Perimeter	119.45	ft
Hydraulic Radius	1.23	ft
Top Width	119.36	ft
Critical Depth	1.74	ft
Critical Slope	0.19573	ft/ft
Velocity	6.35	ft/s
Velocity Head	0.63	ft
Specific Energy	2.36	ft
Froude Number	1.01	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.73	ft
Critical Depth	1.74	ft
Channel Slope	0.20000	ft/ft

Spillway Structure - Graded Riprap

GVF Output Data

Critical Slope	0.19573	ft/ft

Attachment A5: Hydraulics for Variable Bottom Widths

Bottom Width (ft)	Discharge, Q (cfs)	Flow Area (ft^2)	Hydraulic Radius (ft)	Normal Depth (ft)	Velocity in Chute (ft/s)	Froude	Shear Stress (psf)
5	933.00	142.12	1.33	2.54	6.56	1	12.45
10	933.00	142.32	1.33	2.43	6.56	1	12.45
15	933.00	142.72	1.32	2.32	6.54	1	12.36
20	933.00	143.26	1.31	2.22	6.51	1	12.26
25	933.00	143.88	1.3	2.13	6.48	1	12.17
30	933.00	144.69	1.29	2.04	6.45	1	12.07
35	933.00	145.6	1.28	1.96	6.41	1	11.98
40	933.00	146.60	1.27	1.89	6.36	1	11.89
45	933.00	147.75	1.25	1.82	6.31	0.99	11.70
50	933.00	148.93	1.24	1.75	6.26	0.99	11.61

Side Slopes of Spillway	20:1	ft/ft (H:V)
Manning's "n"	0.106	
Slope of Spillway	0.15	ft/ft
Unit Weight, γ	62.4	lb/ft^3
Gravitational Acceleration	32.2	ft/s^2

Attachment B

Attachment B1

MAXIMUM PERMISSIBLE MEAN CHANNEL VELOCITY

MATERIAL / LINING	MAXIMUM PERMISSIBLE MEAN VELOCITY (fps)
NATURAL & IMPROVED UNLINED CHANNELS	
Erosive Soils:	
Loams, Sands, Noncolloidal Silts	3.0
Less Erosive Soils:	
Clays, Shales, Cobbles, Gravel	5.0
FULLY LINED CHANNELS	
Unreinforced Vegetation	5.5
Loose Riprap	10.0
Grouted Riprap	15.0
Gibbons	15.0
Soil-Cement	15.0
Concrete	35.0

NOTES:

1. For composite lined channels, use the lowest of the maximum mean velocities for the materials used in the composite lining.
2. Deviations from the above values are only allowed with appropriate engineering analysis and/or suitable agreements for maintenance responsibilities.
3. Maximum permissible velocities based upon non-clear water conditions.

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ALLOWABLE VELOCITY AND MAXIMUM SHEAR STRESS
Streambank and Shoreland Protection Code 580

Type of Treatment	Allowable Shear lb/sq ft	Velocity ft/sec
Brush Mattresses[1]		
Staked only w/ rock riprap toe (initial)	0.8 - 4.1	5
Staked only w/ rock riprap toe (grown)	4.0 - 8.0	12
Coir Geotextile Roll[2]		
Roll with coir rope mesh staked only without rock riprap toe	0.2 - 0.8	< 5
Roll with Polypropylene rope mesh staked only without rock riprap toe	0.8 - 3.0	< 8
Roll with Polypropylene rope mesh staked and with rock riprap toe	3.0 - 4.0	< 12
Live Fascine[3]		
LF Bundle w/ rock riprap toe	2.0 - 3.1	8
Soils[4]		
Fine colloidal sand	0.02-0.03	1.5
Sandy loam (noncolloidal)	0.03-0.04	1.75
Alluvial silt (noncolloidal)	0.045-0.05	2
Silty loam (noncolloidal)	0.045-0.05	1.75-2.25
Firm loam	0.075	2.5
Fine gravels	0.075	2.5
Stiff clay	0.26	3-4.5
Alluvial silt (colloidal)	0.26	3.75
Graded loam to cobbles	0.38	3.75
Graded silts to cobbles	0.43	4
Shales and hardpan	0.67	6
Gravel/Cobble[4]		
1-inch	0.33	2.5-5
2-inch	0.67	3-6
6-inch	2	4-7.5
12-inch	4	5.5-12
Vegetation[4]		
Class A turf (ret class)	3.7	6-8
Class B turf (ret class)	2.1	4-7
Class C turf (ret class)	1	3.5
Retardance Class D	0.6	Design of roadside channels HEC-15
Retardance Class E	0.35	
Long native grasses	1.2-1.7	4-6
Short native and bunch grass	0.7-0.95	3-4

Attachment B2

Type of Treatment	Allowable Shear lb/sq ft	Velocity ft/sec
Soil Bioengineering[4]		
Wattles	0.2-1.0	3
Reed fascine	0.6-1.25	5
Coir roll	3-5	8
Vegetated coir mat	4-8	9.5
Live brush mattress (initial)	0.4-4.1	4
Live brush mattress (grown)	3.90-8.2	12
Brush layering (initial/grown)	0.4-6.25	12
Live fascine	1.25-3.10	6-8
Live willow stakes	2.10-3.10	3-10
Hard Surfacing[4]		
Gabions	10	14-19
Concrete	12.5	>18
Boulder Clusters[5]		
Boulder		
Very large (>80-inch diameter)	37.4	25
Large (>40-in diameter)	18.7	19
Medium (>20-inch diameter)	9.3	14
Small (>10-inch diameter)	4.7	10
Cobble		
Large (>5-inch diameter)	2.3	7
Small (>2.5-inch diameter)	1.1	5
Gravel		
Very Course (>1.25-inch diameter)	0.54	3
Course (>.63-inch diameter)	0.25	2.5

[1] Brush mattresses (ERDC TN EMRRP-SR-23): http://el.erdc.usace.army.mil/emrrp/pdf/sr23.pdf.
[2] Coir Geotextile roll (ERDC TN EMRRP-SR-04): http://el.erdc.usace.army.mil/emrrp/pdf/sr04.pdf.
[3] Live Fascine (ERDC TN EMRRP-SR-31): http://el.erdc.usace.army.mil/emrrp/pdf/sr31.pdf.
[4] Stream Restoration Materials (ERDC TN EMRRP-SR-29): http://el.erdc.usace.army.mil/emrrp/pdf/sr29.pdf.
[5] Boulder Clusters (ERDC TN EMRRP-SR-11): http://el.erdc.usace.army.mil/emrrp/pdf/sr11.pdf.

Additional Sources:

Wisconsin Department of Transportation, Erosion Control - Product Acceptability List (PAL):
http://www.dot.wisconsin.gov/library/research/docs/finalreports/tau-finalreports/erosion.pdf

Texas Department of Transportation, Approved Products List:
http://www.dot.state.tx.us/mnt/erosion/contents.htm