Calculation Cover Sheet

Project: Hemphill Diversion Project

Client: Nevada Irrigation District - NID **Proj. No.** 21-125

Title: Hemphill Hydraulics - 90% Submittal

Prepared By, Name: J. Burgi

Prepared By, Signature: ______________ **Date:** 3/4/2022

Peer Reviewed By, Name: M. Cerucci

Peer Reviewed, Signature: ______________ **Date:** ______________

SUBJECT:	Nevada Irrigation District - NID	BY: J. Burgi	CHK'D BY: M. Cerucci
	Hemphill Diversion Project	DATE: 3/4/2022	
	Hemphill Hydraulics - 90% Submittal	PROJECT NO.: 21-125	

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Hemphill Diversion Project
Hemphill Canal Entrance Head Loss

BY: J. Burgi CHK'D BY: M. Cerucci
DATE: 3/4/2022
PROJECT NO.: 21-125

Purpose

The purpose of this calculation sheet is to identify the hydraulic grade line from Auburn Ravine to the canal.

References

- *Lindeburg, Michael. (2003). Civil Engineering Reference Manual, California, Professional Publications, Inc.*
- *Mefford, Brent. (2014). Pocket Guide to Screening Small Water Diversions, USFS, USFWS, USBOR*

Information - Input

Hemphill canal flow conditions

Q_{min}	3	cfs
Q_{design}	6	cfs
Q_{max}	18	cfs

Auburn Ravine Water Surface Elevation

$WSEL_{min}$	198.5
$WSEL_{design}$	
$WSEL_{max}$	

Calculation

Headloss through Cone Screen

$$h = \frac{10V_a^2}{64}$$ (Mefford, 2014)

Va 0.33 fps Max screen approach velocity

h 0.017 ft headloss through cone screen

Headloss through pipe from cone screen to canal.

$$h = \frac{3.022v^{1.85}L}{C^{1.85}D^{1.17}}$$ (Lindburg, 2003) eq. 17.30

v		fps	velocity
L	75	ft	length
C	140		roughness coefficienet
D	3	ft	diameter

Q (cfs)	v (fps)	h (ft)
3	0.424	0.0014
6	0.849	0.0050
18	2.546	0.0378

Minor Headloss in pipe from cone screen to canal

$$h_m = K * \frac{v^2}{2g}$$ (Lindburg, 2003) eq. 17.41

Q (cfs)	v (fps)	$v^2/2g$
3	0.424	0.0028
6	0.849	0.0112
18	2.546	0.1007

K

0.9	90 elbow
1	exit
1	entrance

Minor losses in pipe include one 90 degree bend, two 45 degree bends and one exit.

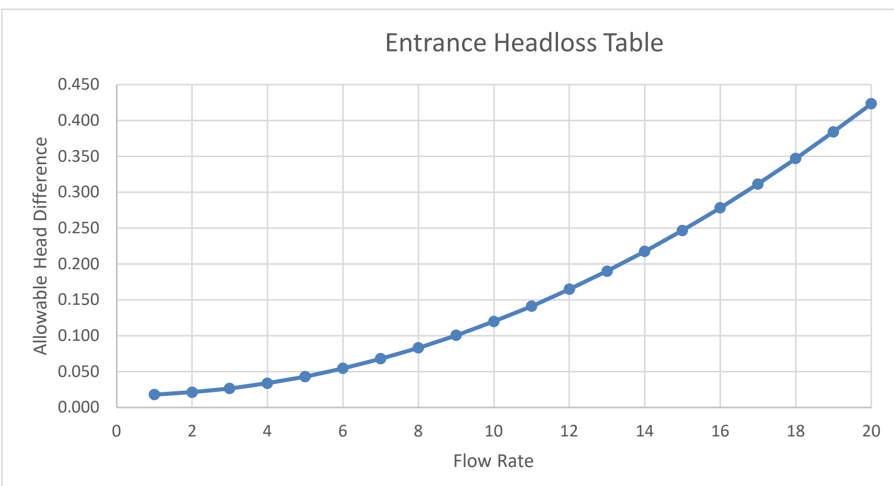
Q (cfs)	Total minor loss (ft)
3	0.008
6	0.032
18	0.292

Conclusion

Total losses from Auburn Ravine to the Canal include a weir, the cone screen, pipe, minor losses through the

	3 cfs	6 cfs	18 cfs
Cone loss (ft)	0.02	0.02	0.02
Pipe loss (ft)	0.00	0.00	0.04
Minor Loss (ft)	0.01	0.03	0.29
Total (ft)	**0.03**	**0.05**	**0.35**

Q	h1	h2	h3	Total
1	0.017	0.000	0.001	0.018
2	0.017	0.001	0.004	0.021
3	0.017	0.001	0.008	0.027
4	0.017	0.002	0.014	0.034
5	0.017	0.004	0.023	0.043
6	0.017	0.005	0.032	0.054
7	0.017	0.007	0.044	0.068
8	0.017	0.008	0.058	0.083
9	0.017	0.010	0.073	0.101
10	0.017	0.013	0.090	0.120
11	0.017	0.015	0.109	0.141
12	0.017	0.018	0.130	0.165
13	0.017	0.021	0.152	0.190
14	0.017	0.024	0.177	0.217
15	0.017	0.027	0.203	0.247
16	0.017	0.030	0.231	0.278
17	0.017	0.034	0.260	0.312
18	0.017	0.038	0.292	0.347
19	0.017	0.042	0.325	0.384
20	0.017	0.046	0.361	0.423



SUBJECT: Nevada Irrigation District - NID
Hemphill Diversion Project
Hemphill Canal Head Loss

BY: J. Burgi **CHK'D BY:** M. Cerucci
DATE: 3/4/2022
PROJECT NO.: 21-125

Purpose

The purpose of this calculation sheet is to identify the hydraulic grade line between the first Turkey Creek Golf Club culvert and the outlet from the fish screen.

References

- *Tullis, J. Paul. (1989). Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients. New York: John Wiley & Sons.*
- *Miller, D.S. (1990). Internal Flow Systems, Design and Performance Prediction. Houston: Gulf Publishing Company.*

Information - Input

Hemphill canal flow conditions

Q_{min}	3	cfs
Q_{design}	6	cfs
Q_{max}	18	cfs

Hemphill Canal

W_b	7	ft	bot width
z	1	:1	side slope
S	0.0002	ft/ft	Slope for end of proposed fish screen to culvert
L	790	ft	Distance from culvert to outlet
h	0.158		Change in elevation at the bottom of canal
n	0.025		Manning's coefficient

Calculation

Based on HY-8 analysis of first culvert (located approximately 790 feet downstream from the proposed fish screen, flow in the culvert is outlet controled, and the WSE at the entrance of the culvert is calculated as:

Flow cfs	WSE ft	depth ft
3	197.26	0.92
6	197.65	1.31
18	198.72	2.38

Analysis will start from the hydraulically controled downstream end. The first culvert on Hemphill canal is approximatlet 790 feet downstream.

Calculation of Normal Depth between fish screen and culvert.

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

$$A = \frac{Q * n}{1.49 * S^{1/2} * R^{2/3}}$$

$$A = d * (w + zd)$$

$$P = w + 2d\,(1 + t^2)^{0.5}$$

Q, cfs	Normal Depth d, ft	A, ft^2	P, ft	R, fr	V , fps
3.00	0.81	6.34	9.29	0.68	0.47
6.00	1.09	8.77	10.07	0.87	0.68
18.00	1.72	15.03	11.87	1.27	1.20

Standard step backwater calculation to find length of canal required to transition from flow depth at the culvert entrance to normal depth.

Backwater calc for 6 cfs

d (ft)	A (ft2)	V (ft/s)	E (ft)	delta E	R (ft)	Sf	Avg -Sf	So-Avg Sf	dl (ft)	Cum Dist	Elev.
1.31	10.89	0.55	1.31		1.01689457	8.41E-05					197.65
				-0.02			8.63E-05	1.14E-04	-174.46	-174.46	
1.29	10.69	0.56	1.29		1.00426617	8.86E-05					197.63
				-0.02			9.10E-05	1.09E-04	-181.85	-356.30	
1.27	10.50	0.57	1.28		0.9915784	9.34E-05					197.61
				-0.02			9.60E-05	1.04E-04	-190.49	-546.80	
1.25	10.31	0.58	1.26		0.97883032	9.86E-05					197.59
				-0.02			1.01E-04	9.86E-05	-200.73	-747.53	
1.23	10.12	0.59	1.24		0.96602094	1.04E-04					197.57
				-0.02			1.07E-04	9.29E-05	-213.03	-960.56	
1.21	9.93	0.60	1.22		0.95314928	1.10E-04					197.55
				-0.02			1.13E-04	8.67E-05	-228.05	-1188.61	
1.19	9.75	0.62	1.20		0.9402143	1.16E-04					197.53
				-0.02			1.20E-04	8.01E-05	-246.78	-1435.38	
1.17	9.56	0.63	1.18		0.92721497	1.23E-04					197.51
				-0.02			1.27E-04	7.30E-05	-270.75	-1706.14	
1.15	9.37	0.64	1.16		0.91415023	1.31E-04					197.49
				-0.02			1.35E-04	6.53E-05	-302.46	-2008.60	
1.13	9.19	0.65	1.14		0.90101898	1.39E-04					197.47
				-0.02			1.43E-04	5.70E-05	-346.31	-2354.90	
1.11	9.00	0.67	1.12		0.88782011	1.47E-04					197.45
				-0.02			1.52E-04	4.80E-05	-410.76	-2765.66	
1.09	8.82	0.68	1.10		0.87455248	1.57E-04					197.43

Backwater calc for 18 cfs

d (ft)	A (ft2)	V (ft/s)	E (ft)	delta E	R (ft)	Sf	Avg -Sf	So-Avg Sf	dl (ft)	Cum Dist	Elev.
2.38	22.32	0.81	2.39		1.6257616	1.07E-05					198.72
				-0.05			1.11E-05	1.89E-04	-261.79	-261.79	
2.33	21.74	0.83	2.34		1.59959704	1.15E-05					198.67
				-0.05			1.20E-05	1.88E-04	-262.78	-524.57	
2.28	21.16	0.85	2.29		1.57325399	1.24E-05					198.62
				-0.05			1.29E-05	1.87E-04	-263.89	-788.46	
2.23	20.58	0.87	2.24		1.54672675	1.34E-05					198.57
				-0.05			1.40E-05	1.86E-04	-265.13	-1053.59	
2.18	20.01	0.90	2.19		1.52000941	1.46E-05					198.52
				-0.05			1.52E-05	1.85E-04	-266.51	-1320.10	
2.13	19.45	0.93	2.14		1.49309576	1.58E-05					198.47
				-0.05			1.65E-05	1.84E-04	-268.07	-1588.17	
2.08	18.89	0.95	2.09		1.46597933	1.72E-05					198.42
				-0.05			1.79E-05	1.82E-04	-269.83	-1858.00	
2.03	18.33	0.98	2.04		1.43865339	1.87E-05					198.37
				-0.05			1.95E-05	1.80E-04	-271.82	-2129.82	
1.98	17.78	1.01	2.00		1.41111086	2.04E-05					198.32
				-0.05			2.13E-05	1.79E-04	-274.09	-2403.90	
1.93	17.23	1.04	1.95		1.38334438	2.23E-05					198.27
				-0.05			2.33E-05	1.77E-04	-276.68	-2680.58	
1.88	16.69	1.08	1.90		1.35534624	2.44E-05					198.22
				-0.05			2.56E-05	1.74E-04	-279.67	-2960.25	
1.83	16.16	1.11	1.85		1.32710835	2.68E-05					198.17
				-0.05			2.81E-05	1.72E-04	-283.13	-3243.38	
1.78	15.63	1.15	1.80		1.29862228	2.94E-05					198.12
				-0.05			3.10E-05	1.69E-04	-287.17	-3530.55	
1.73	15.10	1.19	1.75		1.26987915	3.25E-05					198.07

Conclusion

The cumulative distance to transition from known flow depth of 1.31 feet (6 cfs) at the Turkey Creek Golf Club culvert to a calculated normal depth of 1.09 feet (6 cfs) results in a length of 2,765.66 feet. The proposed fish screen will be located approximately 790 feet upstream of the culvert. Therefore, normal depth will not be reached, and flow at the outlet of the fish screen will be controled by the flow conditions in the culvert. The calculated depth at normal flow of 6 cfs at the outlet from the fish screen is 1.23 feet. For the max flow of 18 cfs, the depth at the outlet from the fish screen is 2.23 ft.

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Nevada Irrigation District - NID	**BY:** J. Burgi	**CHK'D BY:**
	Hemphill Diversion Project	**DATE:** 3/4/2022	
	Roughened Channel - Rock sizing	**PROJECT NO.:** 21-125	

Purpose

The purpose of this calcualtion sheet is to compare different methods of calculating D50 based on CDFW XII methods for rock ramps and the Bureau of Reclemation Rock Ramp sizing methods.

References

- *CDFW. (2009). California Salmonid Stream Habitat Restoration Manual - Part XII Fish Passage and Implementation. CDFW.*
- *USBR. (2007). Rock Ramp Design Guidelines. Denver: U.S. Department of Interior, Bureau of Reclamation*

Information - Input

Q_{MAX} =	5000	ft^3/s	Estimated bank full flow
$Q_{100\ YR}$ =	15000	ft^3/s	From FEMA
Channel Width$_{MAX}$ =	83	ft	Bank full width
Channel Width$_{100\ YR}$ =	600	ft	Approximate floodplain width
S =	0.028	ft/ft	Roughened Channel Slope
q_{MAX} =	60.24	ft^2/s/ft	
$q_{100\ YR}$ =	25.00	ft^2/s/ft	

Calculation

CDFW XII
Equation XII-I ACOE(1994)

$$D_{30-ACOE} = \frac{1.95 * S^{0.555} * (1.25q)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$D_{84-ESM} = 1.5 * D_{30-ASCOE}$$

$$D_{50-ESM} = 0.4 * D_{84-ESM}$$

Where:
$D_{30\text{-}ACOE}$ = D30 stable particle size based on rock gradation provided by ACOE 1994 (ft)
S = Hydraulic slope (ft/ft)
q = unit discharge within active channel at stable bed design flow (cfs/ft)
g = gravitation acceleration (32.2 ft/s^2)

Max			100 Year		
S =	0.028	ft/ft	S =	0.028	
q_{MAX} =	60.24	ft^3/s/ft	$q_{100\ YR}$ =	25.00	ft3/s/ft
g =	32.2	ft/s^2	g =	32.2	ft/s^2
$D_{30\text{-}ACOE\ MAX}$ =	1.502	ft	$D_{30\text{-}ACOE\ 100\ YR}$ =	0.836	ft
$D_{84\text{-}ESM\ MAX}$ =	2.254	ft	$D_{84\text{-}ESM\ 100\ YR}$ =	1.254	ft
$D_{50\text{-}ESM\ MAX}$ =	0.901	ft	$D_{50\text{-}ESM\ 100\ YR}$ =	0.502	ft

BOR
Abt and Johnson (1991) Equation 4-2

$$q_{sizing} = 1.35 * q_{design}$$

$$D_{50} = \varphi_e * \varphi_c * a * 5.23 * S^{0.43} q_{sizing}{}^{0.56}$$

Where:
D_{50} = D_{50} median diameter of rock layer (in)
ϕe = coefficient for empirical envelope on the regression relationship =1.2
ϕc = coefficient of flow concentration due to channelization within revetment
a = shape factor for rounded versus angular material
S = profile slope of rock ramp (ft/ft)
q_{sizing} = design unit discharge (ft^3/s/ft)

Max			100 Year		
$q_{Sizing\ MAX}$ =	81.3253012	ft^3/s/ft	$q_{Sizing\ 100\ YR}$ =	33.8	ft^3/s/ft
ϕe =	1.2		ϕe =	1.2	
ϕc =	1.2	assuming sheet flow	ϕc =	1.2	assuming sheet flow
a =	1.4	rounded material	a =	1.4	rounded material
S =	0.028	ft/ft	S =	0.028	ft/ft
$D_{50\ MAX}$ =	26.61	in	$D_{50\ 100\ YR}$ =	16.26	in
$D_{50\ MAX}$ =	2.22	ft	$D_{50\ 100\ YR}$ =	1.35	ft

BOR
Ullmann (2000) Equation 4-5

$$q_{sizing} = 1.35 * q_{design}$$

$$D_{50} = 6.84 * S^{0.43} * q^{0.56} * C_u^{0.25} * (1.12 * R + 0.39)$$

Where:
D_{50} = D_{50} median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
q_{sizing} = design unit discharge (ft^3/s/ft)
C_u = Coefficient of uniformity, D_{60}/D_{10}
R = percent roundness in decimal form

Max		
$q_{Sizing\ MAX}$ =	81.3253012	ft^3/s/ft
S =	0.028	ft/ft
C_u =	2.4	
R =	0.7	
$D_{50\ MAX}$ =	25.22	in
$D_{50\ MAX}$ =	2.10	ft

100 Year		
$q_{Sizing\ 100\ YR}$ =	33.8	ft^3/s/ft
S =	0.028	ft/ft
C_u =	2.4	
R =	0.7	
$D_{50\ 100\ YR}$ =	15.41	in
$D_{50\ 100\ YR}$ =	1.28	ft

BOR
Ferro (1999) Equation 4-6

$$D_{50} = B * (\varphi_e * \frac{0.95}{(\sigma_g^2)^{0.562}} * (\frac{Q * S}{B^{\frac{5}{2}} * g^{\frac{1}{2}}} * \frac{\gamma_s - \gamma}{\gamma})^{\frac{1}{2}}$$

Where:
D_{50} = D_{50} median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
Q = total discharge (ft^3/s)
ϕe = coefficient for empirical data in regression relationship =1.4
σ_g^2 = geometric variance of gradation, D_{84}/D_{16}
g = gravitation acceleration (32.2 ft/s^2)
γ_s = specific weight of stone (lbs/ft^3)
γ = specific weight of water (lbs/ft^3)

Max		
B =	83	ft
S =	0.028	ft/ft
Q =	5000	ft^3/s
ϕe =	1.4	
σ_g^2 =	4	
g =	32.2	ft/s^2
γ_s =	156.075	lbs/ft^3
γ =	62.43	lbs/ft^3
$D_{50\ MAX}$ =	1.230	ft

100 Year		
B =	600	ft
S =	0.028	ft/ft
Q =	15000	ft^3/s
ϕe =	1.4	
σ_g^2 =	4	
g =	32.2	ft/s^2
γ_s =	156.075	lbs/ft^3
γ =	62.43	lbs/ft^3
$D_{50\ 100\ YR}$ =	1.299	ft

BOR
Robinson et al. (1998) Equation 10-6

$$q_{sizing} = 1.35 * q_{design}$$

$$D_{50} = (\frac{q_{sizing}}{9.76 * 10^{-7} * S^{-1.50}})^{\frac{1}{1.89}}$$

Where:
D_{50} = D_{50} median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
q_{sizing} = design unit discharge (ft^3/s/ft)

Max		
S =	0.028	ft^3/s/ft
q_{sizing} =	81.3253012	ft/ft
$D_{50\ MAX}$ =	258.49	mm
$D_{50\ MAX}$ =	10.18	in
$D_{50\ MAX}$ =	0.85	ft

100 Year		
S =	0.028	ft^3/s/ft
q_{sizing} =	33.75	ft/ft
$D_{50\ 100\ YR}$ =	162.315	mm
$D_{50\ 100\ YR}$ =	6.390	in
$D_{50\ 100\ YR}$ =	0.533	ft

BOR
USACE Bed (1991) Equation 4-8 and 4-9

$$D_{30} = \frac{1.95 * S^{0.555} * (1.25q)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$D_{50} = D_{30} * (\frac{D_{85}}{D_{15}})^{\frac{1}{3}}$$

Where:
D_{30} = Rock diameter for which 30% is smaller by mass (ft)
S = Slope of rock ramp (ft/ft)
q = unit discharge within active channel at stable bed design flow (cfs/ft)
g = gravitation acceleration (32.2 ft/s^2)
D_{85} = Rock diameter for which 85% is smaller by mass (ft)
D_{15} = Rock diameter for which 85% is smaller by mass (ft)

Max

S =	0.028	ft/ft
q_{MAX} =	60.24	ft^3/s/ft
g =	32.2	ft/s^2
$D_{30\ MAX}$ =	1.502	ft
D_{85}/D_{15} =	2.7	
$D_{50\ MAX}$ =	2.092	ft

100 Year

S =	0.028	
$q_{100\ YR}$ =	25.00	ft3/s/ft
g =	32.2	ft/s^2
$D_{30\ 100\ YR}$ =	0.836	ft
D_{85}/D_{15} =	2.7	
$D_{50\ 100\ YR}$ =	1.164	ft

Conclusion

Reference	Equation	D_{50} (ft)		D_{50} (in)	
		Max	100 Yr	Max	100 Yr
CDFW XII	Equation XII-I ACOE(1994)	0.90	0.50	10.82	6.02
BOR	Abt and Johnson (1991) Equation 4-2	2.22	1.35	26.61	16.26
BOR	Ullmann (2000) Equation 4-5	2.10	1.28	25.22	15.41
BOR	Ferro (1999) Equation 4-6	1.23	1.30	14.76	15.59
BOR	Robinson et al. (1998) Equation 10-6	0.85	0.53	10.18	6.39
BOR	USACE Bed (1991) Equation 4-8 and 4-9	2.09	1.16	25.10	13.97

$D_{50\text{-}ESM}$ 0.90

The CDFW rock sizing equation was compared with five other rock sizing equations for both the 100-year flow as defined by FEMA and a "max channel" flow estimating the maximum flow at bankfull flow. Due to the spread of the water for the 100-yr flow, the channel velocities may be lower than that of the bankfull flow. As a result the rock sizes for the max flow are greater than those for the 100-yr.

The D50's ranged from 0.90 ft - 2.22 ft with the CDFW method returning the smallest rock. The average rock size for the max flow is 1.5' (18 inches).

The CDFW D_{50} is smaller partially due to the specific gadation for Engineered Streambed Material (ESM). While the D_{50} is smaller, the gradiation calls for a larger D_{100} (as shown in the following pages) than what would be found in a normal rock gradation. These larger D_{100} rocks will provide sufficient stability for the smaller rock and as such, will provide a stable roughened channel.

Design will move forward with $D_{50\text{-}ESM}$ of 0.90 ft.

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Nevada Irrigation District - NID	**BY:** R. Hudson	**CHK'D BY:** J. Burgi
	Hemphill Diversion Project	**DATE:** 3/4/2022	
	Roughened Channel - Rock Gradation	**PROJECT NO.:** 21-125	

Purpose

The purpose of this calcualtion sheet is to calculate the Rock Gradation for the roughened channel based on the D50 calculated on the preveous calculation sheet.

References

- *CDFW. (2009). California Salmonid Stream Habitat Restoration Manual - Part XII Fish Passage and Implementation. CDFW.*

Information - Input

Q_{MAX} =	5000	ft^3/s	Estimated bank full flow
Channel $Width_{MAX}$ =	83	ft	Bank full width
S =	0.028	ft/ft	Roughened Channel Slope
q_{MAX} =	60.24	$ft^2/s/ft$	
$D_{50\ ESM}$ =	0.90	ft	

Calculation

CDFW XII
Equation XII-I ACOE(1994)

$$D_{30-ACOE} = \frac{1.95 * S^{0.555} * (1.25q)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$D_{84-ESM} = 1.5 * D_{30-ASCOE}$$

$$D_{50-ESM} = 0.4 * D_{84-ESM}$$

$$D_{100-ESM} = 2.5 * D_{84-ESM}$$

Where:
$D_{30\text{-}ACOE}$ = D30 stable particle size based on rock gradation provided by ACOE 1994 (ft)
S = Hydraulic slope (ft/ft)
q = unit discharge within active channel at stable bed design flow (cfs/ft)
g = gravitation acceleration (32.2 ft/s^2)

	Max	
S =	0.028	ft/ft
q_{MAX} =	60.24	$ft^3/s/ft$
g =	32.2	ft/s^2
$D_{30\text{-}ACOE\ MAX}$ =	1.502	ft
$D_{84\text{-}ESM\ MAX}$ =	2.254	ft
$D_{50\text{-}ESM\ MAX}$ =	0.901	ft
$D_{100\text{-}ESM\ MAX}$ =	5.634	ft

CDFW XII
Equation XII-2 ACOE(1994)

$$D_{16-ESM} = 0.32^{\frac{1}{n}} D_{50-ESM}$$

Where:
$D_{50\text{-}ESM}$ = D_{50} median diameter of rock layer (ft)
n = design partical-size distribution curve between 0.45-0.7. n value should result in D8-ESM to be approximately 2 mm. If it fails to, additional fines should be added to the mix to achieve the recommended 5 to 10% fines in the final mix

	Max	
n =	0.45	$ft^3/s/ft$
$D_{50\text{-}ESM\ MAX}$ =	0.90	ft
$D_{16\text{-}ESM\ MAX}$ =	0.072	ft

CDFW XII
Equation XII-3 ACOE(1994)

$$D_{8-ESM} = 0.16^{\frac{1}{n}} D_{50-ESM}$$

Where:
$D_{50\text{-}ESM}$ = D_{50} median diameter of rock layer (ft)
n = design partical-size distribution curve between 0.45-0.7. n value should result in D8-ESM to be approximately 2 mm. If it fails to, additional fines should be added to the mix to achieve the recommended 5 to 10% fines in the final mix

	Max	
n =	0.45	ft^3/s/ft
$D_{50\text{-}ESM\ MAX}$ =	0.90	ft
$D_{8\text{-}ESM\ MAX}$ =	0.015	ft

Additional fines requirec

CDFW XII
Engineered Streambed Material Thickness

$$ESM_{Thickness} = 0.67 D_{100-ESM}$$

Where:
$D_{100\text{-}ESM}$ = D_{100} median diameter of rock layer (ft)

	Max	
$D_{100\text{-}ESM\ MAX}$ =	5.63	ft
$ESM_{Thickness\text{-}MAX}$ =	3.77	ft

Conclusion

Rock Gradation	Max Diameter (ft)	Max Diameter (in)
D100-ESM MAX =	5.63	67.61
D84-ESM MAX =	2.25	27.04
D50-ESM MAX =	0.90	10.82
D30-ACOE MAX =	1.50	18.03
D16-ESM MAX =	0.07	0.86
D8-ESM MAX =	0.02	0.18
ESMThickness =	3.77	45.30

Rock Group	%	% passing 100	inches	Installation Method
A	8	92	50-67	A
B	4	88	35-50	A
C	4	84	24-35	A
D	8	76	18-24	A
E	9	67	9-18	A
F	17	50	9-18	B
G	34	16	2-9	B
H	8	8	2mm-2in	B
I	8	0	<2mm	B

The CDFW guidance for the sizing of the D100 material results in rocks that are excessivly large for the scale of this project.

An analysis of the stability of the rocks was completed on these large rocks (next page) finding that rock with diameter of 5.63 ft results in a factor of safety of 5.4. The D84 material with diameter of 2.25 ft, results in a factor of safety of 2.4. (see Large Rock Stability calcs).

Based on the results of the analysis of the factor of safety, we have reduced the diameter of the D100 material to 42" (3.5 ft) with

Rock Group	%	% passing 100	inches	Installation Method
A	8	92	42-35	A
B	7	85	24-35	A
C	7	78	18-24	A
D	10	68	11-18	A
E	18	50	11-18	B
F	34	16	2-9	B
G	8	8	2mm-2in	B
H	8	0	<2mm	B

SUBJECT: Nevada Irrigation District - NID | BY: J. Burgi | CHK'D BY:
Hemphill Diversion Project | DATE: 3/4/2022
Roughened Channel - Large Boulder Analysis | PROJECT NO.: 21-125

Purpose

The purpose of this calcualtion sheet is to calculate the factor of safety for the larger boulders

References

• *USBR. (2007). Rock Ramp Design Guidelines. Denver: U.S. Department of Interior, Bureau of Reclamation*

Information - Input

Q_{MAX} =	5000	ft^3/s	Estimated bank full flow
$Q_{100\ YR}$ =	15000	ft^3/s	From FEMA
Channel $Width_{MAX}$ =	83	ft	Bank full width
Channel $Width_{100\ YR}$ =	600	ft	Approximate floodplain width
S =	0.028	ft/ft	Roughened Channel Slope
q_{MAX} =	60.24	$ft^2/s/ft$	
$q_{100\ YR}$ =	25.00	$ft^2/s/ft$	
$D_{50\ MAX}$ =	0.90	ft	
$D_{84\ MAX}$ =	2.25	ft	
P =	85.2	ft	wetted perimeter
A =	241.4	ft^2	Area
R =	2.8	ft	Hydarulic Radius
W =	83.00	ft	Top Width
D =	2.91	ft	Hydraulic Depth

Calculation

$$SF = \frac{a_\theta \cdot \tan(\phi)}{\eta_1 \cdot \tan(\phi) + \sqrt{1 - a_\theta^{\,2}} \cdot \cos(\beta)}$$

Equation 7-1

$$a_\theta = \sqrt{\cos^2(\theta_1) - \sin^2(\theta_0)}$$

$$\theta = \tan^{-1}\left(\frac{\sin(\theta_0)}{\sin(\theta_1)}\right)$$

$$\eta_1 = \eta_0 \left[\frac{\left(\frac{A}{B}\right) + \sin(\lambda + \beta + \theta)}{1 + \left(\frac{A}{B}\right)}\right] \cong \eta_0 \left[\frac{1 + \sin(\lambda + \beta + \theta)}{2}\right]$$

$$\eta_0 \cong \frac{18 \cdot \tau_0}{(\gamma_s - \gamma_w) \cdot D_s}$$

$$\beta = \tan^{-1}\left(\frac{\cos(\lambda + \theta)}{\frac{(A+B) \cdot \sqrt{1 - a_0^{\,2}}}{B \cdot \eta_0 \cdot \tan(\phi)} + \sin(\lambda + \theta)}\right) \cong \tan^{-1}\left(\frac{\cos(\lambda + \theta)}{\frac{2 \cdot \sqrt{1 - a_0^{\,2}}}{\eta_0 \cdot \tan(\phi)} + \sin(\lambda + \theta)}\right)$$

$$A = \left(\frac{l_4}{l_2}\right) \cdot \left(\frac{F_L}{F_S}\right)$$

$$B = \left(\frac{l_3}{l_4}\right) \cdot \left(\frac{F_D}{F_S}\right)$$

$$\frac{A}{B} \approx 1$$

Where,

- SF = Safety factor;
- D_s = rock diameter;
- θ_0 = longitudinal bed slope;
- θ_1 = bank side slope;
- ϕ = Angle of repose (≅42 Degrees);
- λ = angle of vertical stream line deviation from horizontal, must be ≥ 0 (outside of a bend);
- τ_0 = bed shear stress = $\gamma \cdot R \cdot S_f$;
- γ = unit weight of water;
- R = hydraulic radius;
- S_f = friction slope;
- θ = down-slope angle including bed and bank slope;
- η_0 = shear force acting on the rock;
- β = correction for side slope, bed slope, and secondary currents;
- η_1 = correction for side slope, bed slope, and secondary currents; and
- $l_{1,2,3,4}$ = moment arms between riprap particles (canceled through lift and drag assumptions.
- A, B = lever arm ratios. The ratio A/B is assumed to equal 1.

$$\tau = \gamma * R * S$$

Using the above equations:

Diameter of large rocks (ft)	D_s =	1	2	2.25	3	3.5	5.6
	τ =	4.9	4.9	4.9	4.9	4.9	4.9
	η_o =	0.868	0.434	0.386	0.289	0.248	0.155
Assuming parallel streamlines	λ =	0	0	0	0	0	0
Side slope	θ_1 =	0	0	0	0	0	0
	θ_0 =	1.60	1.60	1.60	1.60	1.60	1.60
Since the SIN of zero is zero	θ =	90	90	90	90	90	90
	a_q =	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996
	β =	0	0	0	0	0	0
	η_1 =	0.868	0.434	0.386	0.289	0.248	0.155
	SF =	1.1	2.1	2.4	3.1	3.6	5.4

Scour around large rocks - similar to pier scour

$$y_{s,p} = 2.0 * K_1 * K_2 * K_3 * K_4 * a^{0.65} * y_1^{0.35} * Fr_1^{0.43}$$ CSU Pier Scour Eq.

$$Fr = \frac{v}{(g * h_m)^{1/2}}$$

Fr = 1.03

$y_{s,p}$ = 0.83

v =	10	Velocity (from Hec-Ras, fps)
h_m =	2.91	Hydarulic Depth
K1	1.1	
K2	1	
K3	1.1	
K4	0.2	
a	1	Pier Diameter (Rock Diameter, ft)
y1	4.5	Flow depth (from Hec_Ras)

Conclusion

Based on the analysis above, Rock larger than the $D_{84\text{-}ESM}$ would be stable with a factor of safety of 2.4 on the roughened rock ramp. A minimum recommended factor of safety of 1.1 is reached with a rock diameter of 1.0 feet.

Assuming pier type scour around larger boulders, there is the potential of 1.4 ft scour. Since the overall thickness of the roughened ramp will be approximately 3.75 feet, the system is sufficiently armored.

SUBJECT: Nevada Irrigation District - NID
Hemphill Diversion Project
Roughened Channel - Manning's Coefficient

BY: J. Burgi CHK'D BY:
DATE: 3/4/2022
PROJECT NO.: 21-125

Purpose

The purpose of this calcualtion sheet is to calculate the manning's coefficient for the roughened channel at high flow.

References

• *USBR. (2007). Rock Ramp Design Guidelines. Denver: U.S. Department of Interior, Bureau of Reclamation*

Information - Input

Q_{MAX} =	5000	ft^3/s	Estimated bank full flow
$Q_{100\ YR}$ =	15000	ft^3/s	From FEMA
Channel $Width_{MAX}$ =	83	ft	Bank full width
Channel $Width_{100\ YR}$ =	600	ft	Approximate floodplain width
S =	0.028	ft/ft	Roughened Channel Slope
q_{MAX} =	60.24	$ft^2/s/ft$	
$q_{100\ YR}$ =	25.00	$ft^2/s/ft$	
$D_{50\ MAX}$ =	0.90	ft	
$D_{84\ MAX}$ =	2.25	ft	
P =	85.2		wetted perimeter
A =	241.4		Area
R =	2.8		Hydarulic Radius
W =	83.00		Top Width
D =	2.91		Hydraulic Depth

Calculation

Depth Independent Roughness for Mild Gradients

$$n = K_u * D_x^{1/6}$$ Equation 3-1 (BOR)

n = Channel Roughness
K_u = dimensional coefficient
D_x = representative grain diameter

Author	$K_{u\ (m)}$	$K_{u\ (ft)}$	X for D_x	Dx (ft)	Dx (m)	n
Henderson (1966)	0.038		75	1.96	0.60	0.035
Lane and Carlson (1955)	0.0473	0.0388	75	1.96	0.60	0.043
Strickler (1923)	0.041		50	0.90	0.27	0.033
USACOE (1991)	0.046	0.038	90	3.33	1.01	0.046

Depth Based Roughness

$$\frac{1}{\sqrt{f}} = -2.0 \cdot \log_{10}\left(\frac{k_s}{3.71 \cdot D} + \frac{2.51}{\text{Re} \cdot \sqrt{f}}\right)$$ Equation 3-2 (BOR)

$$\text{Re} = \frac{V \cdot D}{\nu}$$ Equation 3-3 (BOR)

V	4	fps	velocity
D	2.91		Average hydraulic depth (flow area divided by top with)
v	0.000014081	ft2/s	kinematic viscosity
Re	826,098.2		
ks	0.5	height of roughness element (0.5*D50)	
right side	2.757958605	Calculate right side of equation 3-2	
left side	2.8	Calculate left side of equation 3-2	
Diff	(0.0)	Use to goal seek	
f	0.127		

$$n = R^{1/6} \cdot \sqrt{\frac{f}{8 \cdot g}}$$

Equation 3-4 (BOR)

R = 2.8

n = 0.03

Steep Slope Roughness Estimation

$$n = 0.029 * (D_{50} * S_o)^{0.147}$$

D_{50} = 0.90 Medium grain size
S_o = 0.028 Slope of ramp

n = 0.0391

Conclusion

Of the analysis listed above for the estimation of the manning's coefficient for a roughened rock channel, the Army Corps of Engineergs appears to be the most conservative. It is recommended that a manning's n - coefficient of **0.045** be used in the hydraulic modeling of the roughened channel.

SUBJECT:	Nevada Irrigation District - NID	**BY:** J. Burgi	**CHK'D BY:** M. Cerucci
	Hemphill Diversion Project	**DATE:** 3/4/2022	
	Low Flow Channel	**PROJECT NO.:** 21-125	

Purpose

The purpose of this calculation sheet is to analyze the proposed low flow channel for the roughened channel.

References

- *Lindeburg, Michael. (2003). Civil Engineering Reference Manual, California, Professional Publications, Inc.*
- *Mefford, Brent. (2014). Pocket Guide to Screening Small Water Diversions, USFS, USFWS, USBOR*

Information - Input

Auburn Ravine Design flows:

Q95	13.3	cfs

Calculation

Calculate normal depth of flow in low flow channel at low flow

The low flow channel will be a triangular channel with side slopes of 2:1 and depth of 1 ft.

$$Q = \left(\frac{1.49}{n}\right) A \left(R_h^{\frac{2}{3}}\right) S^{\frac{1}{2}}$$

Manning Coeff	n	0.045	
hydraulic grade line	S	0.028	ft/ft
bottom width	btm	0	ft
sideslope right	zr	2	
sideslope left	zl	2	
depth	d	1	ft
Area	A	2.00	sqft
Wetted Perimeter	W	4.47	ft
Hydraulic Radius	R_h	0.45	ft
Velocity	V	3.23	fps
Flow	Q	6.46	cfs

Calculate normal depth of flow in trapazoidal medium flow channel

The medium flow channel will be a trapazoidal channel with 2:1 side slopes, a bottom width of 8 ft, and a depth of 0.75 feet.

$$Q = \left(\frac{1.49}{n}\right) A \left(R_h^{\frac{2}{3}}\right) S^{\frac{1}{2}}$$

Manning Coeff	n	0.045	
hydraulic grade line	S	0.028	ft/ft
bottom width	btm	8	ft
sideslope right	zr	2	
sideslope left	zl	2	
depth	d	0.75	ft
Area	A	7.13	sqft
Wetted Perimeter	W	11.35	ft
Hydraulic Radius	R_h	0.63	ft
Velocity	V	4.05	fps
Flow	Q	28.86	cfs

Conclusion

The low flow triangual channel will reach capacity at normal flow depth of 1 foot at a flow rate of 6.5 cfs. The trapazoidal medium flow channel will reach capacity at normal flow depth of 0.75 at a flow rate of 28.7 cfs. Adding the triangular flow to the trapazoidal channel flow results in a flow rate of 35.2 cfs.

SUBJECT:	Nevada Irrigation District - NID	BY:	R. Hudson	CHK'D BY:	J. Burgi
	Hemphill Diversion Project	DATE:	3/4/2022		
	Transition Rock Sizing	PROJECT NO.:	21-125		

Purpose

The purpose of this calcualtion sheet is to calculate the Rock Sizing for the section of river downstream of the roughened channel.

References

- *USBR. (2009). Hydraulic Design of Stilling Basin and Energy Dissipators Engineering Monograph No. 25. USBR.*

Information - Input

Q_{MAX} =	5000	ft^3/s	Estimated bank full flow
$Q_{100\ YR}$ =	15000	ft^3/s	From FEMA
V_{MAX} =	10.61	ft/s	From HEC-RAS model at location where rock channel ends
$V_{100\ YR}$ =	13.94	ft/s	From HEC-RAS model at location where rock channel ends

Calculation

Hydraulic modeling of the roughened channel and transition into the downstream natrual stream channel indicate that there will not be a hydraulic jump at the 100-yr flow condition. Therefore the transition from the roughened channel to the natural stream will be protected using design criteria for the protection of stream channel downstream of a stilling basin where most of the energy has been removed from the flow, however flow is characterized by high velocity flow including surges and waves. The following chart has been developed by the USBR(2009) to guide the designer in sizing riprap downstream from stilling baisins.

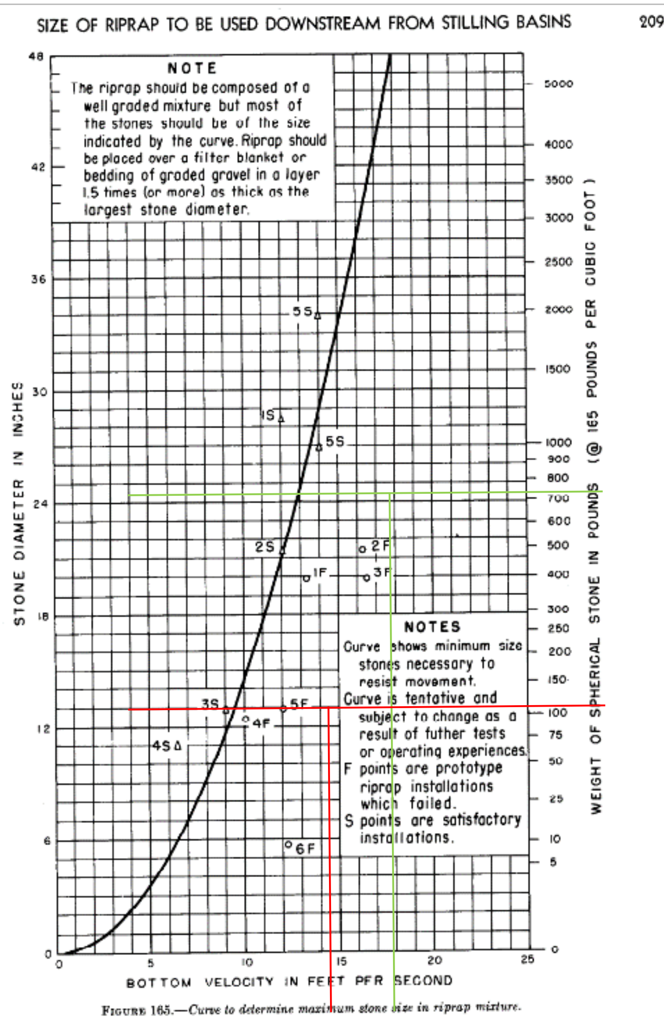


Figure 1. Minimum Riprap Sizing

Using Figure 1 above, the required stone size is as follows:

$d_{100\ YR}$ =	27.8	in
$d_{BANKFULL}$ =	16.1	in
$W_{100\ YR}$ =	1100	lbs
$W_{BANKFULL}$ =	210	lbs

Conclusion

Research conducted by USBR in the use of this chart established that a well-graded riprap layer containing about 40 percent of the rock pices smaller than the required size as shown above was as stable, or more stable than a single stone of the required size. The proposed streambed rock distribution shown in the Rock Gradation calculation for the roughened channel suggests a rock gradation that would meet the required stone size calaculated above.

SUBJECT: Nevada Irrigation District - NID
Hemphill Diversion Project
Fish Screen Approach Velocity

BY: J. Burgi **CHK'D BY:**
DATE: 3/14/2022
PROJECT NO.: 21-125

Purpose

The purpose of this calcualtion sheet is to analize the approach velocity at the cone fish screen at varying depths of water and varying flow rates.

References

• *ISI (2022) Data Sheet provided to McMillen Jacobs via e-mail on 2/23/2022*

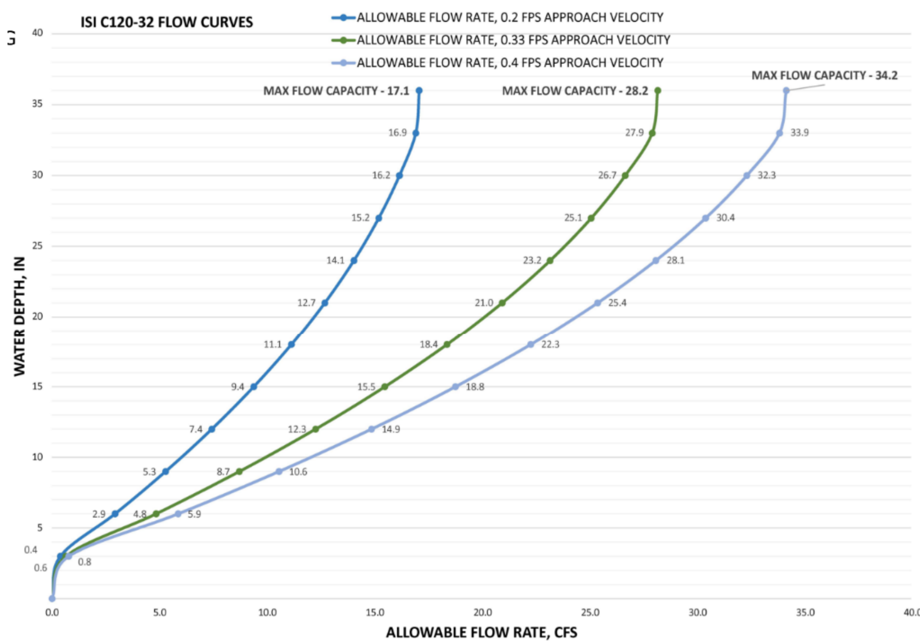
Information - Input

Q_{min}	3	cfs
Q_{design}	6	cfs
Q_{max}	18	cfs

Calculation

Water depth and approach velocity calculations in front of the fish screen throughout range of diversion flows

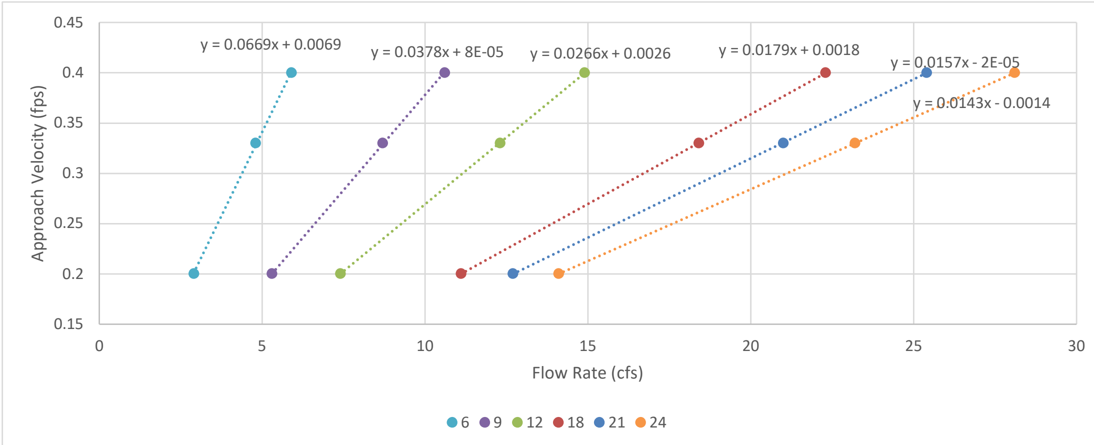
Based on flow data provided by manufacturer (ISI 2022):



The following table is made up of data from the figure above.

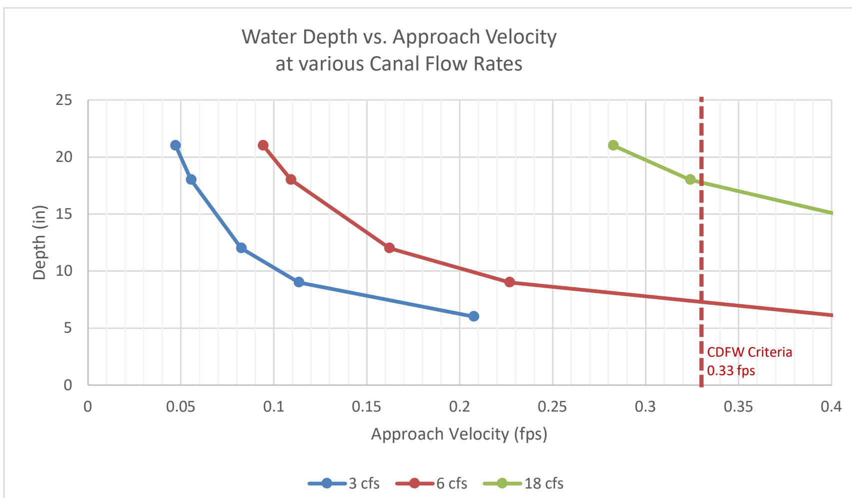
	Approach Vel.		
Depth	0.2	0.33	0.4
3	0.4	0.6	0.8
6	2.9	4.8	5.9
9	5.3	8.7	10.6
12	7.4	12.3	14.9
15	9.4	15.5	18.8
18	11.1	18.4	22.3
21	12.7	21	25.4
24	14.1	23.2	28.1
27	15.2	25.1	30.4
30	16.2	26.7	32.3
33	16.9	27.9	33.9
36	17.1	28.2	34.2

This data can be re-plotted for Approach Velocity vs. Flow Rate at differing depths of water at the screen.



The equation for the linear relationship between approach velocity and flow rate at varying depths of water are used to develop the following table and figure.

	Flow Rate (cfs)								
Depth	3	4	5	6	8	10	12	16	18
6	0.2076	0.2745	0.3414	0.4083	0.5421	0.6759	0.8097	1.0773	1.2111
9	0.1135	0.1513	0.1891	0.2269	0.3025	0.3781	0.4537	0.6049	0.6805
12	0.0824	0.1090	0.1356	0.1622	0.2154	0.2686	0.3218	0.4282	0.4814
18	0.0555	0.0734	0.0913	0.1092	0.1450	0.1808	0.2166	0.2882	0.3240
21	0.0471	0.0628	0.0785	0.0942	0.1256	0.1570	0.1884	0.2512	0.2826



Conclusion

For the minimum diversion of 3 cfs, approach velocities will be significantly below the CDFW criteria of 0.33 fps throughout the range of depths. For the normal diversion of 6 cfs, approch velocities will be below the CDFW criteria once the depth of water on the cone screen is above 7 inches (WSEL 197.08 ft). At the max flow rate of 18 cfs, approach velocities will be below the CDFW criteria when the depth of water at the cone screen is greater than 17 inches (WSEL 197.92 ft).

SUBJECT:	Nevada Irrigation District - NID	BY: J. Burgi	CHK'D BY:
	Hemphill Diversion Project	DATE:	
	Stream Flow vs. Depth Rating Curve	PROJECT NO.: 21-125	

Purpose

The purpose of this calcualtion sheet is to develop a stream flow rating curve.

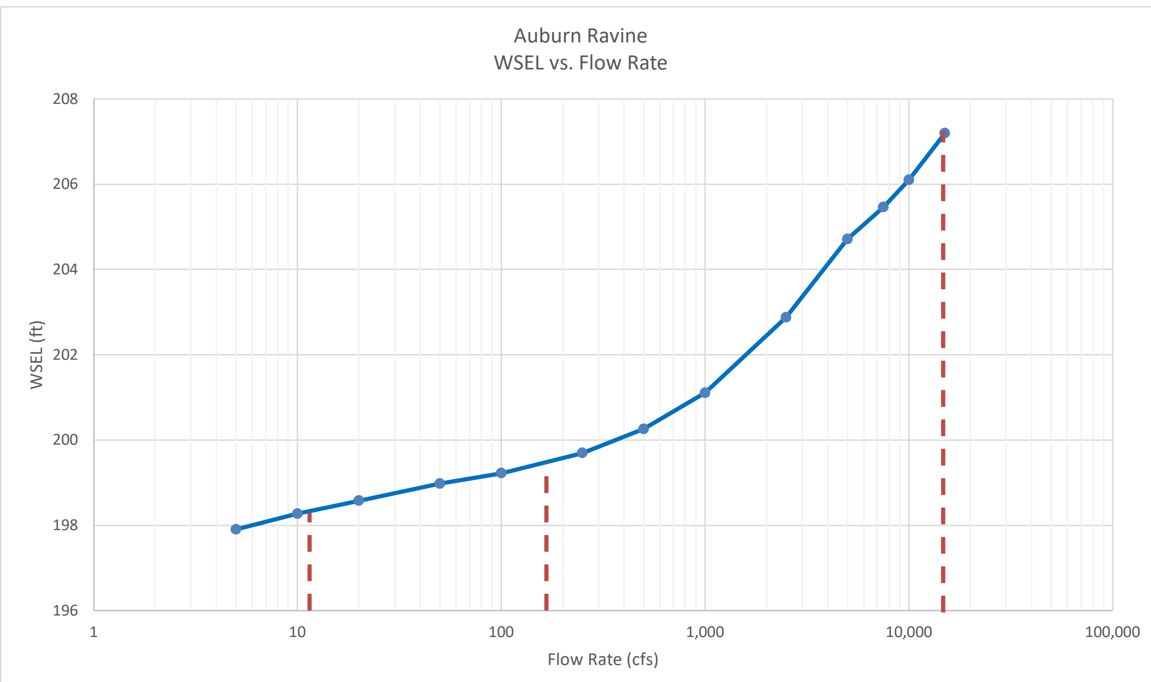
References

Information - Input

Based on 2-D HEC-RAS Model,

Flow (cfs)	WSEL (ft)
5	197.906
10	198.274
20	198.577
50	198.977
100	199.221
250	199.694
500	200.259
1000	201.108
2500	202.875
5000	204.716
7500	205.459
10000	206.103
15000	207.193

Calculation



Conclusion

Based on output from the current 2-D HEC-RAS model for the proposed roughend channel, the above rating curve estimates the water surface elevation at the Cone Screen Structure for flows in Auburn Ravine ranging from 10 cfs to 15,000 cfs.