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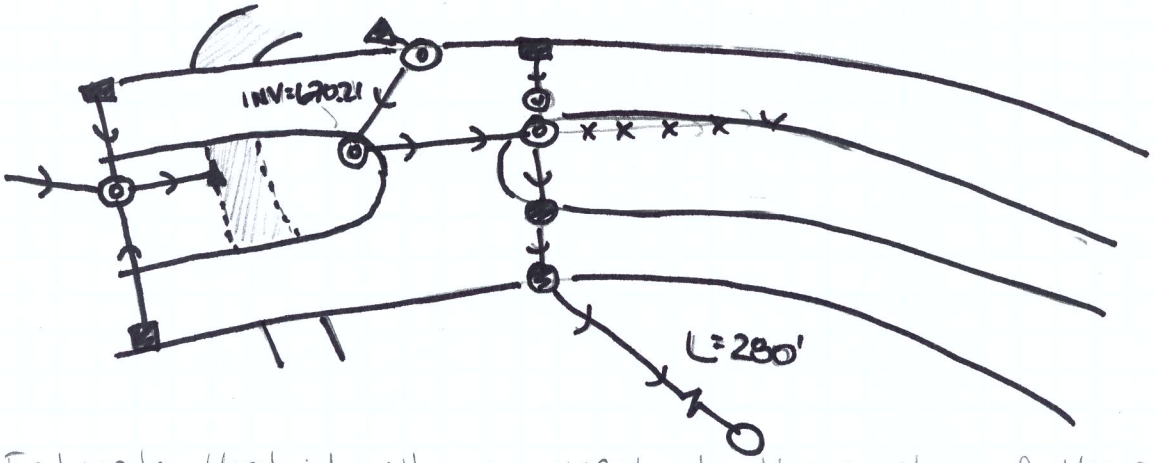
	STRUCTURE TITLE: Scotts/Hutchinson Ravine	CHECKED BY:	DATE:	CONTRACT NO:	

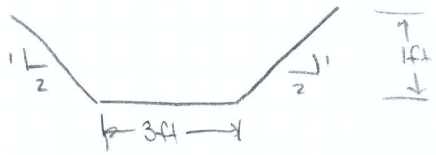
Hutchinson Realignment

* Replace P76 & raise to 1% slope
  - ↳ US INV = 670.39
  - ↳ Length = 36 × 0.005 = 0.16 ft
  - ↳ DS INV = 670.21
* Replace J76 w/ MUCH shallowes structure
* Add structure in island across intersection & truncate/replace P78
* Run sewes across street & around/through trees
* Meander sewes through trees & park until can do straight run.
* Run sewes straight until approx 2-3' of coves remaining
  - ↳ Daylight sewes outfall & use swale to continue divession to detention basin



* Estimate that it will run nearly to the center of the parade grounds until able to daylight.

* Swale: U/S elev = 663.03
  D/S elev = 654.02
  Length = 400 ft
  $V_{max\text{-}allow}$ = 2.5 ft/s



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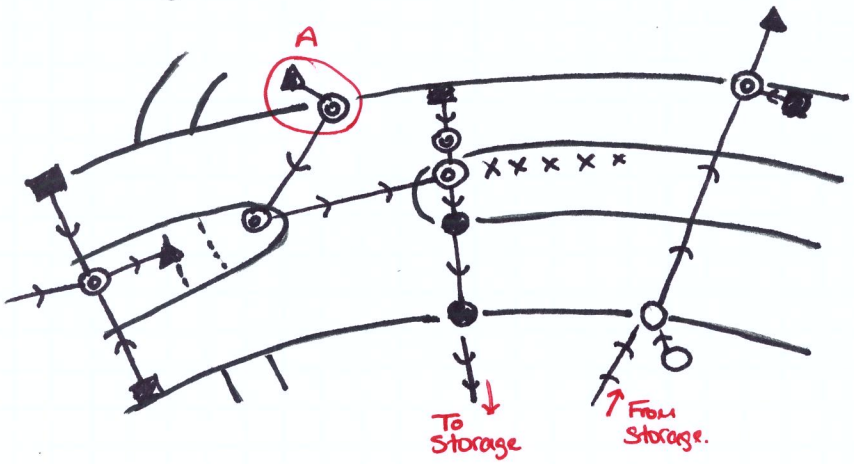
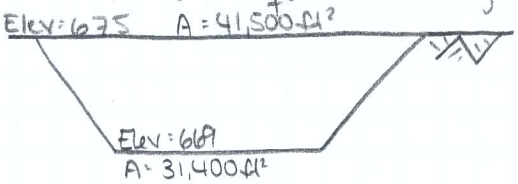
	STRUCTURE TITLE: Hutchinson Ravine	CHECKED BY:	DATE:	CONTRACT NO:	

Hutchinson - Option #2: Detention Basin by Entrance

* This measure uses the same realignment detailed in Hutchinson Option #1, but terminates in the field to the south.
* Place storage in field, then reroute back to original outfall



* Location A problematic due to low rim elevation
* Initial estimate of required storage volume



* Using 12" restrictor, $Q_{max} = 4.92$ cfs
* Uses 74% of the available volume

For Underground Storage:

Need $\forall = 218,700 ft^3$
Use 5'x10' structure 400' long
↳ volume = $20,000 ft^3$ each
↳ Need 11

If using 12" outlet, $Q_{max} = 4.68$ cfs
Uses 82% of available volume

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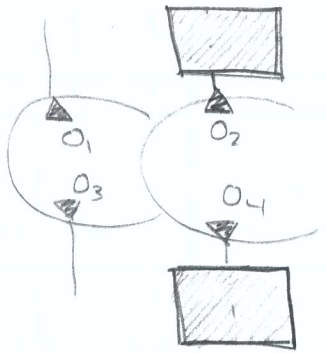
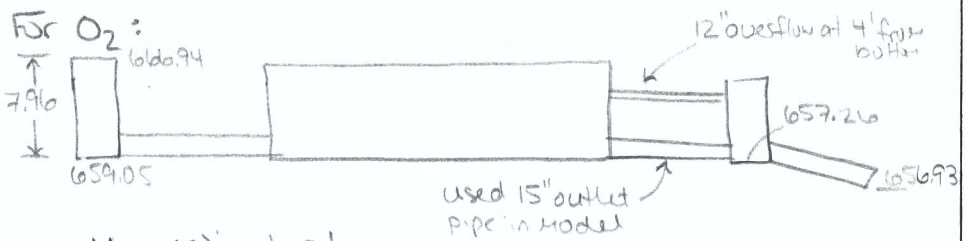
Detention Pond Modification #1



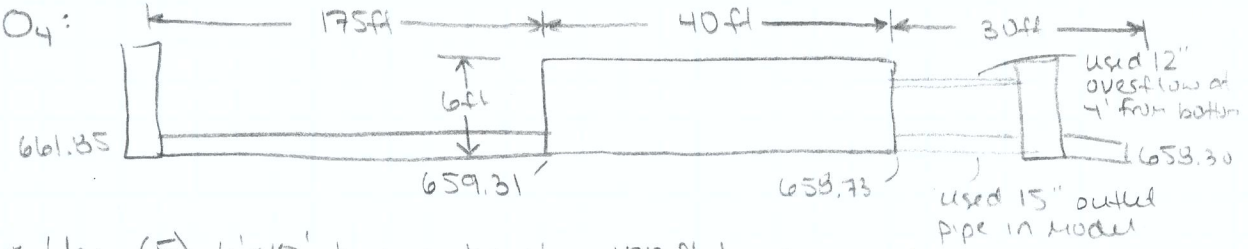
For O_1 & O_4: Consides abandoning?

For O_2:



* Use (8) 6'×10' box culverts 100ft long
* Uses 81% of the proposed storage volume
* $Q_{max\text{-}out}$ = 15 cfs

For O_4:



* Use (5) 6'×10' box culverts 100ft long
* Uses 82% of the proposed storage volume
* $Q_{max\text{-}out}$ = 10.68 cfs

Note: These are inadequate to provide sufficient storage to mitigate the overall discharge from the detention ponds.

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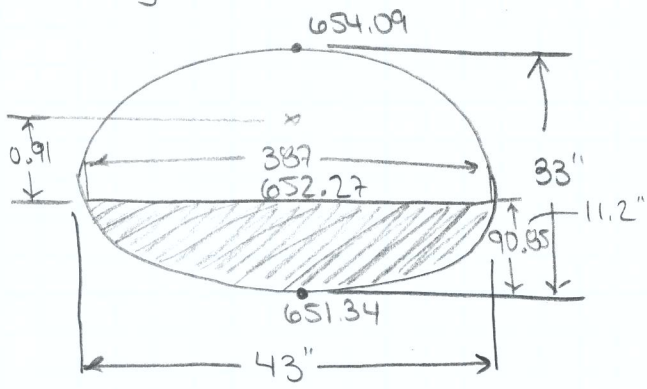
Sizing of Weirs



$A_{tot} = 7.74\,ft^2$
$A_{block} = 2.23\,ft^2$ } $A_{onf} = 5.5\,ft^2$

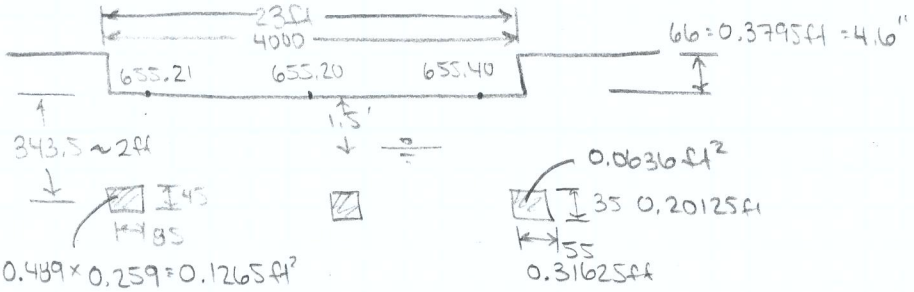
Weir

$Q = 1.92 L H^{3/2}$
$= 1.92 (6.67\,ft) H^{3/2}$
$= 12.81\,ft\, H^{3/2}$

Orifice

$Q = 0.62 (2 \times 5.5\,ft^2) \sqrt{2 \cdot 32.2 \frac{ft}{s^2} \cdot H}$
$= 54.73\, H^{1/2}$

Detention Basin #1 Weir



h	Q_{onf}
0	0
0.1	0.2
0.5	0.445
0.75	0.545
1.0	0.629
1.25	0.704
1.50	0.77
1.75	0.833
2.00	0.89

h	Q_w
0	0
0.1	1.8
0.2	5.1
0.3	9.4
0.4	14.7
1.3	126.15
2.3	387.7

H	Q_o	Q_w	Q_{tot}
0	0	-	0
0.1	0.2	-	0.2
0.5	0.445	-	0.445
0.75	0.545	-	0.545
1.0	0.629	-	0.629
1.5	0.77	-	0.77
2.0	0.99	0	0.99
2.1	0.912	1.8	2.71
2.2	0.934	5.1	6.034
2.3	0.955	9.4	10.36
2.4	0.975	14.7	15.675
3.3	1.14	126.15	127.29
4.3	1.31	387.7	399.01

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Detention Pond Modification #2: Enlarge Detention #1

Increase Storage Volume

Original		Modified +1	Modified +2
0	15835	20410	34410
1.5	16331	20906.5	34907
1.8	17161	21737	35737
2.3	18555	23311	37311
3.3	20005	25942	39942
4.3	24229	30166	44166
5.3	31752	37689	51689
Q_1 = 31.11 cfs		Q_1 = 31.1 cfs	Q_1 = 31.08 cfs
Q_2 = 35.5 cfs		Q_2 = 35.5 cfs	Q_2 = 35.43 cfs

Note: Increasing the storage volume of the facility upstream does not significantly improve the overall discharge.

Detention Pond Modification #3: Reduce Weir Length Btwn Basins

* Try 1/3 current length

$Q = 1.92LH^{3/2} = 1.92(7ft)H^{3/2}$

h	Q
0	0
0.1	0.425
0.2	1.202
0.3	2.208
0.4	3.4
0.5	24.44
1.3	126.98
2.3	273.22

H	Q_{tot}
0	0
0.1	0.2
0.5	0.45
1.0	0.63
2	0.89
2.1	1.32
2.2	2.14
2.3	3.163
2.4	4.38
2.5	29.44
3.3	129.12
4.3	274.53

* Reduce opening of culverts by 0.5.

ORIGINAL

Q_{max} = 35.46 for 15 min

33-35.5 for 1.5 hrs.

MOD

Q_{max} = 34.55 for 15 min

33-35 for 1.75 hrs.

Note: Reducing weir openings might delay the peak, but functionally only increases storage volume utilized while NOT significantly reducing the peak discharge.

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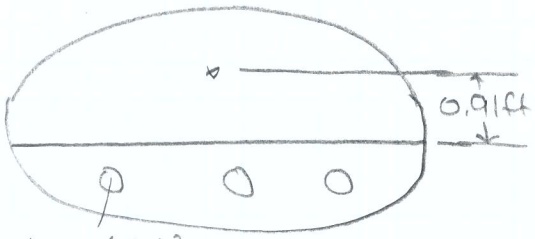
Detention Pond Modification #4: Modify Weirs to Include Orifice



$A = \frac{\pi (2/12)^2}{4} = 0.022 ft^2$

Weir

$Q = 12.31 ft\ H^{3/2}$

Orifice #1

$Q = 0.62 (2 \times 5.5 ft^2) \sqrt{2 (32.2 \frac{ft}{s^2}) H}$

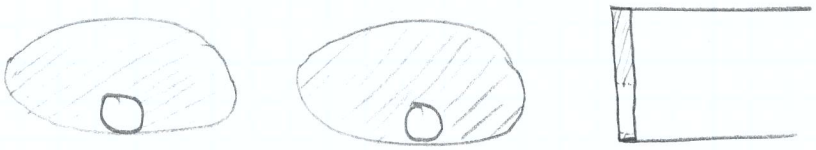
Orifice #2

$Q = 6 \times 0.62 (0.022 ft^2) \sqrt{2 (32.2 \frac{ft}{s^2}) H}$

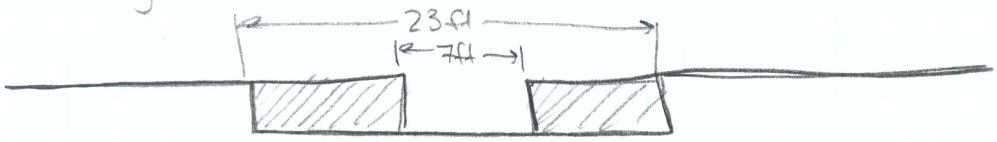
Note: Not enough to make significant impact!

Detention Pond Modification #5: Modify Culvert Inlet

* Provide 12" orifice into culvert



* Alter configuration of weirs between Basins #1 & #2



Note: Flow backs up until stage such that ~30 cfs reached. Can block outlet to reduce discharge, but not enough storage to contain resulting volume

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Detention Pond Modification #6: Drop Structure at Head

From SWMM model,

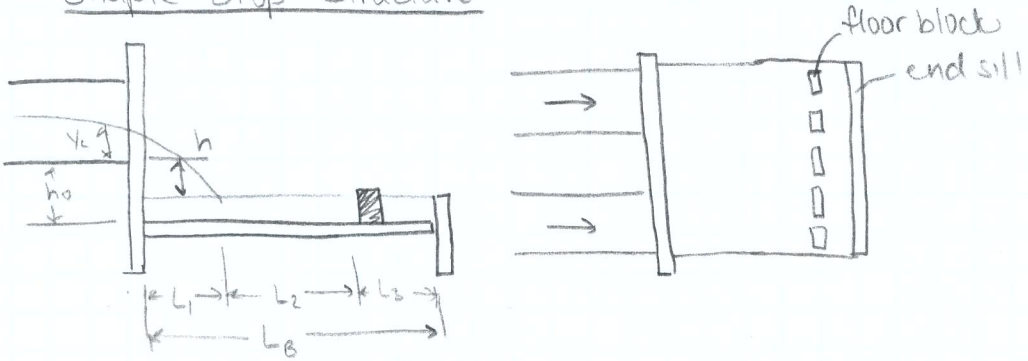
$Q_{max} = 35.46$ cfs for $t = 1.5$ hrs.

$$q = \frac{35.46 \text{ cfs}}{7.2 \text{ ft}} = 4.95 \frac{ft^2}{s}$$

From HEC-14, Section 11

$$N_d = \frac{q^2}{g h_0^3} = \frac{(4.95 ft^2/s)^2}{(32.2 \frac{ft}{s^2})(6 ft)^3} = 0.0035$$

Simple Drop Structure



$$Y_c = \left(\frac{q^2}{g}\right)^{1/2} = \left[\frac{(4.95 ft^2/s)^2}{(32.2 ft/s^2)}\right]^{1/2} = 0.87 ft$$

$Y_3 = 2.15 Y_c = 2.15(0.87 ft) = 1.87 ft$

$h_2 = -(h - Y_0)z - (5.13 - 0.87) = -4.26 ft$

$h_0 = h_2 - Y_3 = (-4.26 ft) - (1.87 ft) = -6.13 ft$

Solve for Basin Length

$$L_f = \left(-0.406 + \sqrt{3.195 - 4.368 \frac{h_0}{Y_c}}\right) Y_c = \left(-0.406 + \sqrt{3.195 - 4.368\left(\frac{-6.13}{0.87}\right)}\right)(0.87) = 4.74'$$

$$L_t = \left(-0.406 + \sqrt{3.195 - 4.368\left(\frac{h_2}{Y_c}\right)}\right) Y_c = \left[-0.406 + \sqrt{3.195 - 4.368\left(\frac{-4.26}{0.87}\right)}\right](0.87 ft) = 3.96 ft$$

$$L_s = \frac{\left[0.691 + 0.228\left(\frac{L_t}{Y_c}\right)^2 - \left(\frac{h_0}{Y_c}\right)\right] Y_c}{0.185 + 0.456\left(L_t / Y_c\right)} = 4.8 ft$$

$$L_1 = \frac{L_f + L_s}{2} = \frac{(4.74\,ft) + (4.8\,ft)}{2} = 4.8\,ft$$

$L_2 = 0.8\,y_c = 0.8(0.87\,ft) = 0.7\,ft$

$L_3 \geq 1.75\,y_c = 1.75(0.87\,ft) = 1.53\,ft$

$L_B \Sigma L_i = (4.8\,ft) + (0.7\,ft) + (1.6\,ft) = 7.1\,ft \rightarrow$ use 8 ft

Basin Blocks & End Sill

Block Height = $0.8\,y_c = 0.7\,ft$

Block Width = Block Spacing = $0.4\,y_c = 0.35\,ft \sim 0.4\,ft$.

End Sill Height = $0.4\,y_c = 0.4\,ft$

Design Basin Entrance & Exit

Sidewall Height above Tailwater Elevation = $3y_c$

$= 2.64\,ft$

$\approx 32"$

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Riffle Sizing

For XS 821 to 669: Not Needed ($S_o \approx 0.022$)

For XS 669 to 547:

$H = (0.051 - 0.022)(669 - 547) = 3.5\,ft$

Use 6" riffles $\frac{H}{n} = \frac{3.5}{0.5} = 7.0 \rightarrow$ need 7 structures

$V_d = 1.6V = 1.6(5\,ft/s) = 8\,ft/s$

$D_{30} = C\left[(0.78)\left(\frac{V}{\sqrt{K_1 g d}}\right)\right]^{2.5} = 0.525\left[0.78\left(\frac{8\,ft/s}{\sqrt{32.2\,ft/s^2 \cdot 0.7'}}\right)\right]^{2.5}$

$= 1.1\,ft.$

$Spacing = \frac{669 - 547}{7} = 17\,ft.$

For XS 547 - 437:

$H = (0.127 - 0.022)(111.9) = 11.75\,ft.$

Use 12" riffles $\frac{H}{n} = \frac{11.75}{1} = 11.75 \rightarrow$ need 12 structures.

$V_d = 1.6V = 1.6(5\,ft/s) = 8\,ft/s$

$D_{30} = 0.45\left[(0.78)\left(\frac{8.0\,ft/s}{\sqrt{32.2\,ft/s^2 \cdot 0.6'}}\right)\right]^{2.5} = 1.56\,ft$

$Spacing = \frac{111.9\,ft}{12} = 9.3\,ft.$

For XS 437 to 271:

$H = (0.026 - 0.022)(165.8) = 0.67\,ft$

Use 6" riffles $\frac{H}{n} = \frac{0.67}{0.5} = 1.34 \rightarrow$ need 2 structures

$V_d = 1.6V = 1.6(3.6\,ft/s) = 5.8\,ft/s$

$D_{30} = 0.6\left[(0.78)\left(\frac{5.8\,ft/s}{\sqrt{32.2\,ft/s^2 \cdot 0.8'}}\right)\right]^{2.5} = 0.5\,ft.$

$Spacing = \frac{165.8\,ft}{2} = 83\,ft.$

* Assume All Riffle Widths = 25 feet
* Replace existing checkdam w/ 1' stone riffle using D_{50} = 1' stones.
* Consider replacing inlet to storm sewer (specifically the trash grate)

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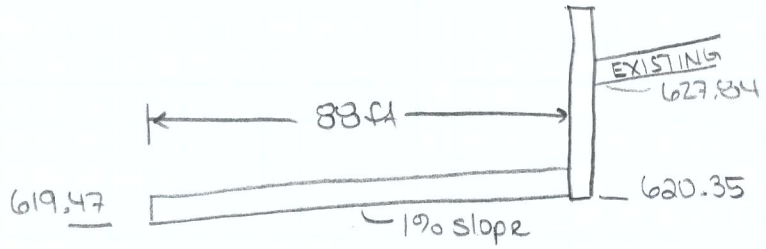
North Tributary

* Remove existing headwall & advance ~70ft towards main stretch
* Backfill behind new headwall
* Add new manhole at outfall & length of 24" pipe to discharge directly to main reach



* Include Plunge Pool at Confluence

$Q_{MAX} = (4.16\,cfs) + (43.8\,cfs) = 48\,cfs$

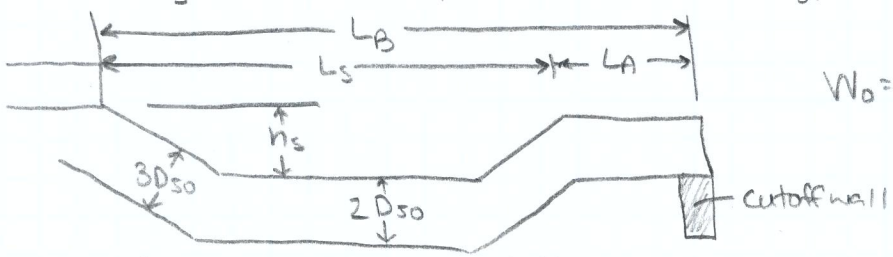
$V_{MAX} = 5.43\,ft/s \rightarrow V_d = 1.6V = 8.7\,ft/s$

$Y_e = 0.61\,ft$

$h_s \sim 1\,ft.$

$$\frac{h_s}{Y_e} = 0.86\left(\frac{D_{50}}{Y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gY_e}}\right) - C_o$$

Use $D_{50} = 0.5ft$, $h_s = 1ft$. $\left\{\frac{D_{50}}{Y_e} > 0.1$? OK! & $\frac{h_s}{D_{50}} = 2 \geq 2$ OK!



$W_o = 2ft$

$L_S = 10h_s = 10ft$ or $3W_o = 6ft$

$L_A = 5h_s = 5ft$ or $W_o = 2ft$

$L_B = 15h_s = 15ft$ or $4W_o = 8ft$

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Storm Sewer Near Mouth of Ravine

* Remove existing rubish & debris
* From SWMM Model
  $Q_{max} = 3.41$ cfs

Option #1: Cobble Chute

From EM-1110-2-1601, Eqn. 3-5

$$q = \frac{3.41 \text{ cfs}}{2 \text{ ft}} = 1.7 \frac{\text{ft}^2}{\text{s}} \Rightarrow q_d = 1.25 q = 2.14 \frac{\text{ft}^2}{\text{s}}$$

$$S_o = \frac{628 - 596.5}{185} = 0.17$$

$$D_{30} = \frac{1.95 S_o^{0.555} q^{2/3}}{g^{1/3}} = \frac{1.95(0.17)^{0.555}(2.14 \text{ ft}^2/\text{s})^{2/3}}{(32.2 \text{ ft/s}^2)^{1/3}} = 0.4 \text{ ft}$$

Use 6" stone w/ thickness = 9"

Option #2: In-Ravine Pipe

From SWMM model,

* Install structure at outfall
* Install 180' of 18" pipe to concrete chute near ravine floor
* Covers w/ 1' stone & 1' clean backfill

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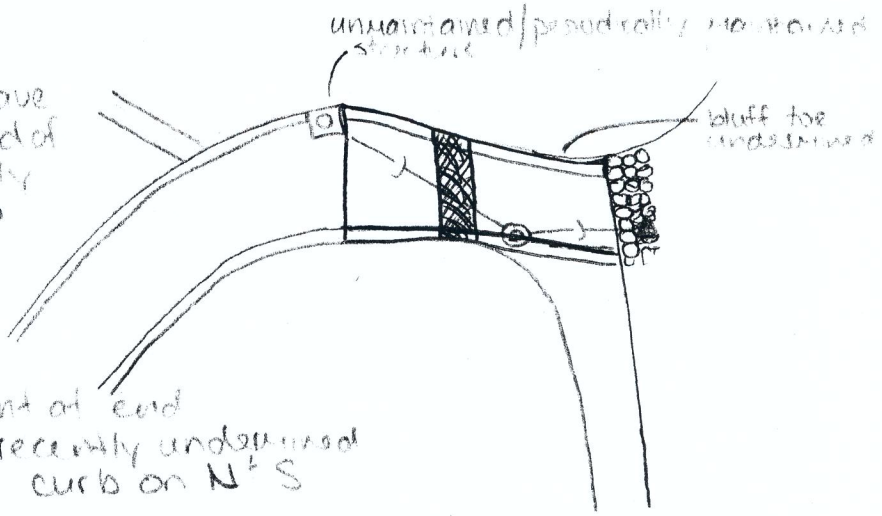
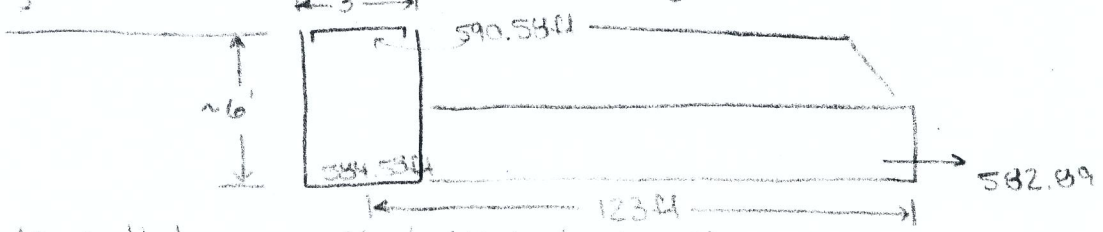
Mouth of Ravine



* During site visits, have found structure at end of north gutter constantly blocked, causing flow to bypass. Runs along edge of street, undercutting toe of bluff to north; running over pavement at end of ravine. Pavement recently undermined
* Install vertical faced curb on N & S sides of pavement.
* Over ~30ft stretch, transition from c/g to shallow v-shaped pavement at N structure → S, beginning at
  ↳ Note: Length will be controlled by what is necessary to incorporate S. gutter
* Install grated inlet from curb to curb over trench that intersects existing storm sewer line. Approx 35' long x 3' wide.



* Provide outlet pipes that drain to beach
  ↳ Assume flow at 0.8D → D = 2.5ft

$$Q = \frac{1.49}{n} A R_h^{2/3} S_o^{1/2} \qquad n = 0.013 \qquad R_h = 0.76\text{ft} \qquad A = 4.2\text{ft}^2$$

$$= \frac{1.49}{0.013}(4.2\text{ft}^2)(0.76\text{ft})^{2/3}(0.014)^{1/2}$$

$$= 47.5\text{ cfs}$$

$Q_{max} = 100.9$ cfs → need (3) pipes

* Provide riprap apron on slope downstream of grate

$$q = \frac{101\text{ cfs}}{20\text{ft}} \quad 5.05\ \frac{\text{ft}^2}{\text{s}} \rightarrow q_c \quad 6.32\ \frac{\text{ft}^2}{\text{s}}$$

$$D_{50} = \frac{1.95(0.2)^{0.555}(6.5^{\text{ft}^2/\text{s}})^{2/3}}{(32.2)^{1/3}} = 0.9\text{ft}$$

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Size Plunge Pool at Base of Slope/Outfall

- Assume pipe flows at 0.8D
- For $d/D = 0.8$, $A/D^2 = 0.67336 \Rightarrow A = 0.67336(2.5\,ft)^2 = 4.206\,ft^2$

Estimate velocity: $V = \frac{Q}{A} = \frac{47.5\,ft^3/s}{4.21\,ft^2} = 11.28\,\frac{ft}{s}$

From HEC-14 Equation 10.1,

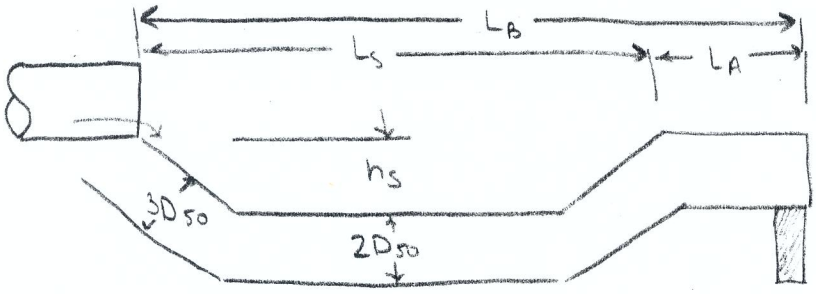
$$\frac{h_s}{Y_e} = 0.86\left(\frac{D_{50}}{Y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gY_e}}\right) - C_o$$

$Y_e \sim 2\,ft$

$h_s \sim 2\,ft$

$$\frac{h_s}{Y_e} = 0.86\left(\frac{D_{50}}{Y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gY_e}}\right) - C_o$$

Use $D_{50} = 1\,ft$ $h_s = 2\,ft$.



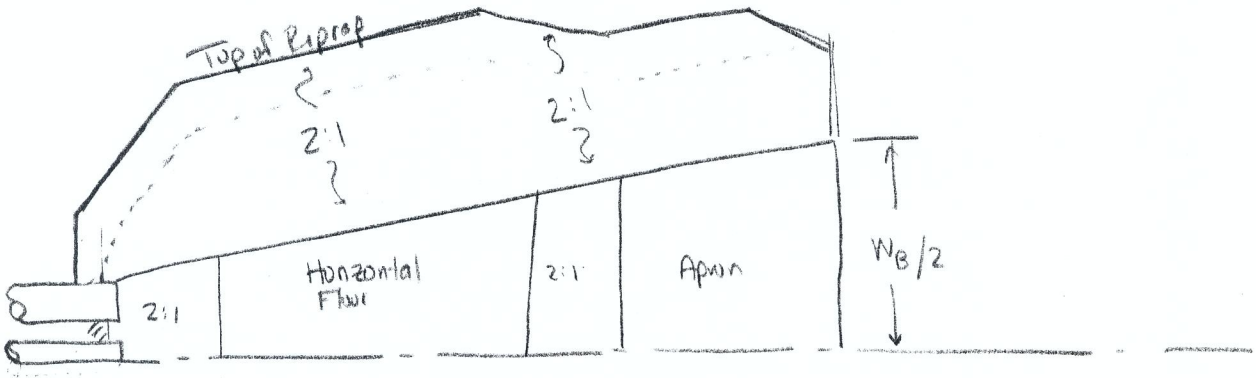
$L_S = 10h_s = 20\,ft$

$L_A = 5h_s = 10\,ft$

$L_B = 15h_s = 30\,ft$

Since using (3) 2.5ft diameter pipes, assume spacing of D/2; total width = 12.5ft

$$W_B = W_o + 2(L_B/3) = 12.5\,ft + 2\left(\frac{30\,ft}{3}\right) = 32.5\,ft$$



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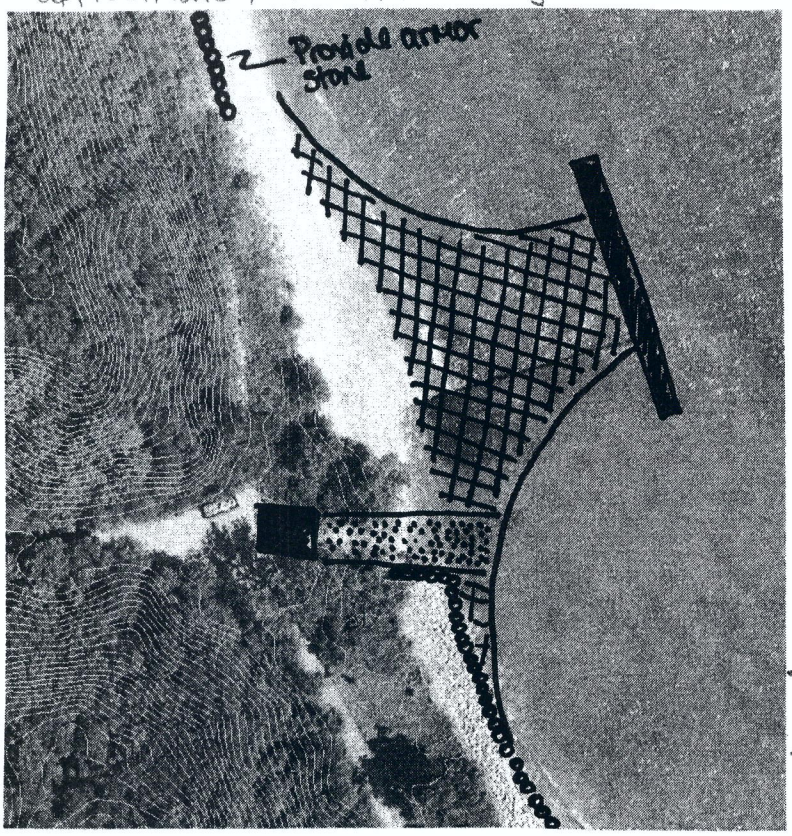
Integrate Into Shoreline Measures

* Use variation of Measure DE, Bartlett Ravine
* Remove groin north of Bartlett Ravine
* Locate midpoint of central nearshore breakwater of system DE approximately 54' north of groin.



* As before, structure is approximately 250 ft long and 130 ft offshore.
* Assume that stable shoreline configuration consistent w/ calculation sheet 7
* Field verify that existing stone south of mouth is adequate based on sizings shown in calculation sheet 13.
  - $D_{50} = 1.2$ ft
  - $W_{50} = 285$ #
  - $d_s = 2.5$ ft
* Provide 80 ft of stone at toe of berm per calculation sheet 13.
* Locate plunge pool at mouth of ravine
* Provide channel to lake.

* Stabilize designated banks using stone, tree trunks, root wads, etc. out to waterline. It's expected that the outlet will periodically silt in, but will open up during the next storm event. Bank stabilization will reduce the likelihood of the channel undercutting the southern revetment

Estimate velocity from end of plunge pool to waterline

Elev outfall = 582.89 ft
Elev MWL = 578.33 ft
$\Rightarrow S_o = \frac{582.89 - 578.33}{120 \text{ ft}} = 0.038$

$Q = \frac{1.49}{n} A R_h^{2/3} S_o^{1/2}$ where $A = By = (45\text{ft})y$, $P = B + 2y = (45\text{ft}) + 2y$, $R_h = A/P$

$V = Q/A$

$V_{S_o=0.038} = 5.35 \frac{ft}{s}$ → Will likely erode channel. Estimate required slope

$V_{S_o=0.009} = 3.47 \frac{ft}{s}$ → Use $S_o = 0.009$

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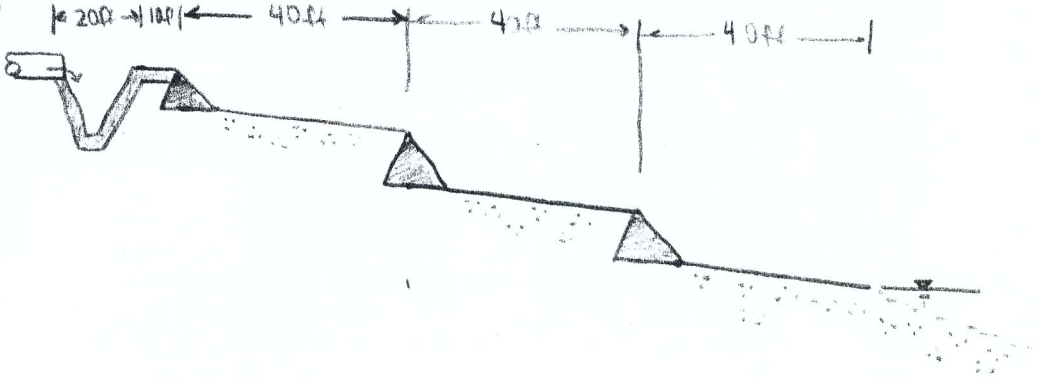
If multiple drop structures used,

$H = (0.038 - 0.009)(120ft) = 3.48ft$

Use (3) riffles total, (1) 12" riffle immediately downstream of plunge pool and (2) 1.25ft riffles spaced every 40ft thereafter



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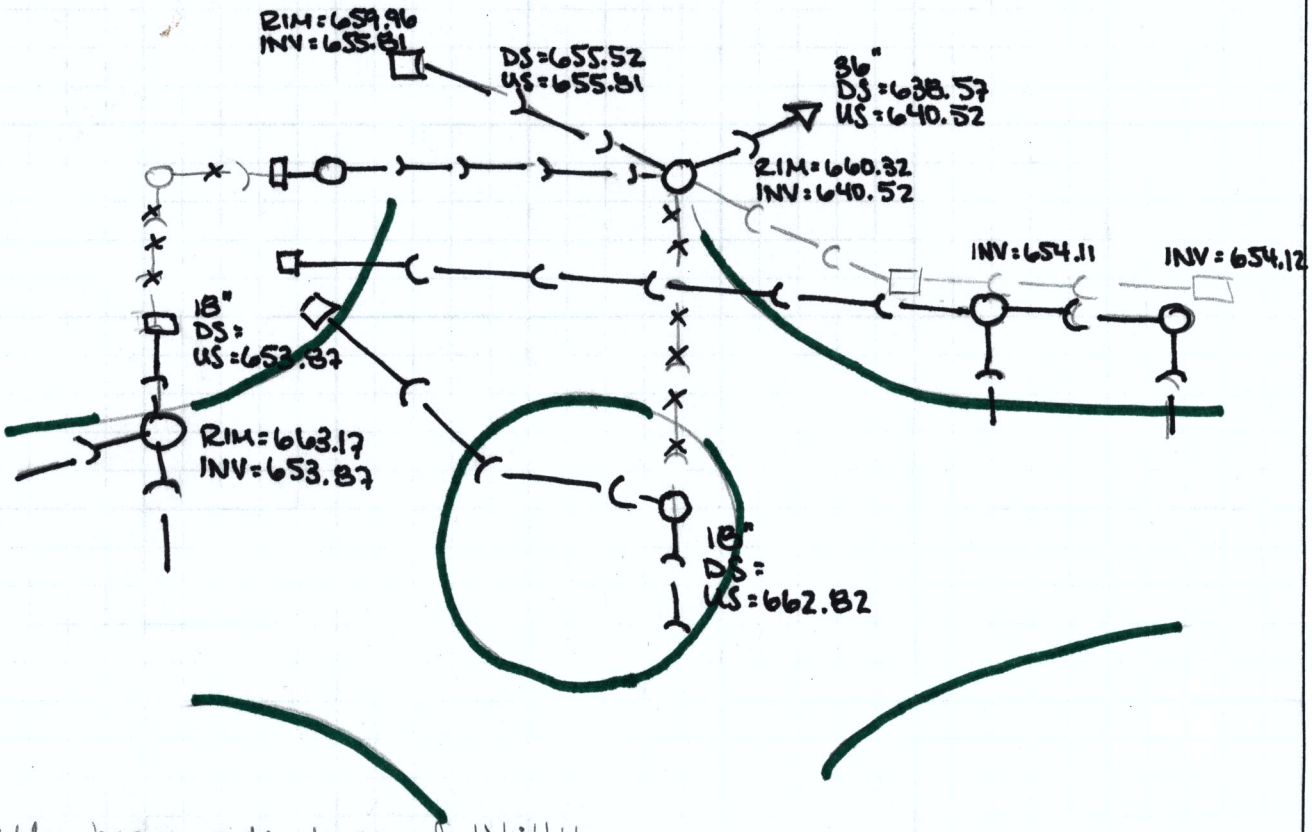
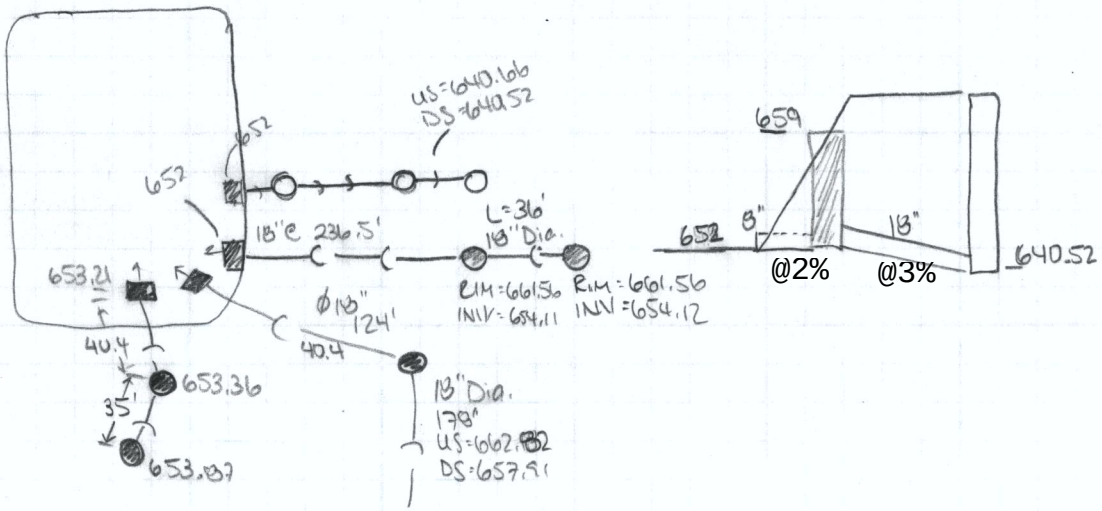
Option #1: Detention Storage at Head of Ravine

* Allowable Discharge: Target = 2 cfs
  Allowable = 7 cfs
* Need to realign storm sewess through the basin



* Use basin side-slopes of 1V:4H
* Use 1% slope for outlet pipe ~ 110 feet length



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For Current + Approx. 50% Undeveloped Area

	Elev	Area
0	648	17,573
4	652	28,250
8	656	40,176
12	660	55,300

* Uses 90% of storage volume
* Max depth = 11.20 ft.
* Provide beehive grate at top of structure for overflow.

For Current Only

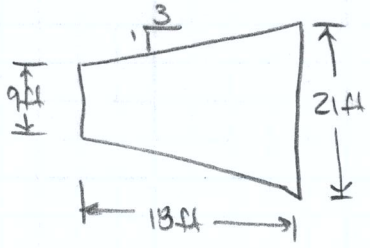
0	652	10,600
4	656	15,070
8	660	20,900

* Uses 77% of storage volume
* Max depth = 6.6ft.

Plunge Pool at Head of Ravine

$V_{max} = 8\frac{ft}{s} \rightarrow V_d = 1.2\,V_{max} = 9.6\frac{ft}{s} \sim 10\frac{ft}{s}$

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{10\frac{ft}{s}^2}{2 \cdot 32.2\frac{ft}{s^2}}\right) = 0.65ft.$$



Thickness $= 1.5 D_{50} = 1.0ft.$

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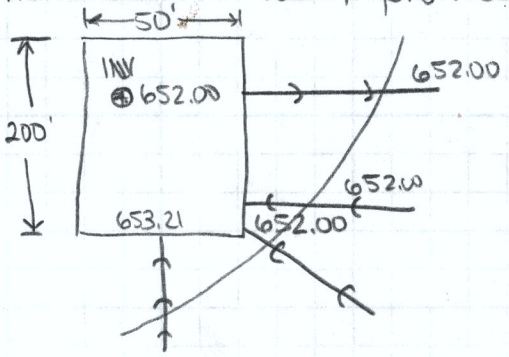
Option #2: Underground Detention Storage

Allowable Discharge: Target = 2cfs
Allowable = 7cfs.

From SWMM model, provide (5) 8'×10' structures 200' long



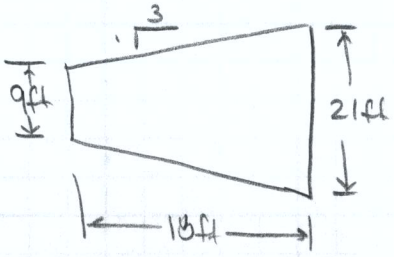
$Q_{max} = 4.47$ cfs
$Q_d = 1.2Q = 5.4$ cfs

$V_{max} = 8 \frac{ft}{s}$
$V_d = 1.2V = 9.6 \frac{ft}{s} \sim 10 \frac{ft}{s}$

Plunge pool at head of ravine

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{10^2}{2 \cdot 32.2}\right) = 0.65 ft$$

$$D_{50} = 0.02D\left(\frac{Q}{D^{2.5}}\right)^{4/3}\left(\frac{1}{0.4}\right) = 0.02(3ft)\left(\frac{5.4}{3^{2.5}}\right)^{4/3}\left(\frac{1}{0.4}\right) = 0.04 ft.$$



Thickness = $1.5D_{50}$ = 1ft.

Note: Design does not take into account additional development

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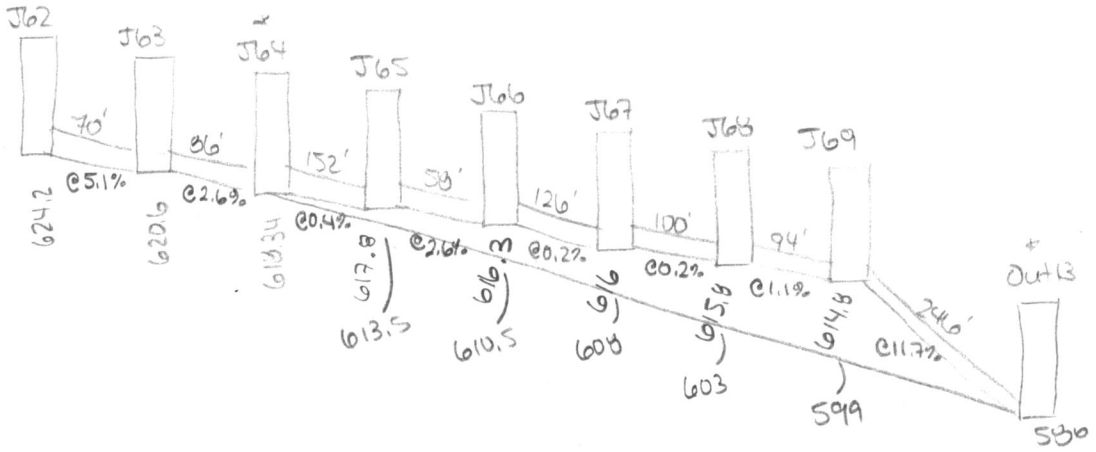
Option #3: In-Ravine Pipe

* Eliminate the existing detention basin by adding structures/pipe & backfilling. Enlarge its connection from 6" to 24"
* At head of ravine, drop crown of pipe to existing grade; assume any filling will be done to repair incision
* Since only single outfall, tie in w/ new structure; must be sure to bury it in the head slope.
* At mouth of ravine:
  - Connect the sewer to the existing culvert under the slope (use single large structure). Provide a beehive grate at the structure to drain the ravine. Allow minor ponding.
  - Excavate the mouth. Cut the floor to reduce the severity of the drop at the end... use riffles to transition & reduce the slope. Armor the beach at the mouth
  - Excavate the mouth. Leave floor as is, but still the pipe to existing culvert. Build cascade down face instead.



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Riffle Design

For XS1063 to 370

*use riffles to reduce velocity by raising channel bed

→ Current = $\frac{631.2 - 617.90}{1113.6 - 607.8} = 0.026$

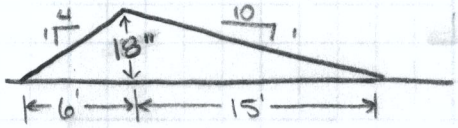
→ Try $S_f = 0.017$

$H = (S_o - S_f)X = (0.026 - 0.017)(730) = 6.57\text{ft} \sim 6.6\text{ft}.$

No. of structures = $\frac{H}{h} = \frac{6.6\text{ft}}{0.5} = 13.2$ (use 14)

Required Spacing = $\frac{730\text{ft}}{14} = 52\text{ft}$

Estimated as 15' wide



Estimate Stone Size

$V_d = 1.6(3\text{ft/s}) = 4.8\text{ft/s}$

$C = 0.6$

$d = 0.5\text{ft}$

$$D_{30} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\left(\frac{V}{\sqrt{K_1 g d}}\right)\right]^{2.5} = 0.875\left[(0.78)\left(\frac{4.8\frac{ft}{s}}{\sqrt{(32.2\frac{ft}{s^2})(0.5ft)}}\right)\right]^{2.5}$$

$= 0.32\text{ft}$

Note: The following options are measured relative to mouth, so some overlap may occur w/ the above riffles. Where this occurs, the riffle below supersede those above.

If Fill Excavated & Ravine Floor Dropped

$S_f = 0.008$

$S_o = \frac{621.53 - 584.69}{790} = \frac{36.84}{790} = 0.047$

$H = (S_o - S_f)X = (0.047 - 0.008)(790) = 30.52\text{ft}.$

Use (31) 12" riffles

Spacing = $\frac{790}{31} = 26\text{ft}.$

Assume width = 10ft.

If Fill Excavated & Ravine Floor NOT dropped (Sloped at Culvert)

$S_f = 0.008$

$S_o = \frac{620 - 584.69}{240\ ft} = 0.147$

$H = (S_f - S_o)X = (0.147 - 0.008)(240ft) = 33.4ft.$

* If using 12" riffles, would need 34 structures at 7.1 foot spacing... consides alternative configuration
* Try stone weirs.

Estimate Required stone size

* Use $V = 5ft/s \rightarrow V_d = 1.6V = 8ft/s$
* $C = 0.375$ (from Schenck analysis)
* $D_{30} = 0.375\left[(0.78)\left(\frac{8\frac{ft}{s}}{\sqrt{32.2\frac{ft}{s^2} \cdot 0.5ft}}\right)\right]^{2.5} = 1.13ft$

Estimate Toe Scour

* channel width ~ 3ft
* $D_{MNC} = 0.5ft$
* $D_{MXB} = 1.14 D_{MNC}\left(1.72 + \frac{0.0084W}{D_{MNC}}\right)$
  $= 1.14(1ft)\left(1.72 + \frac{0.0084 \cdot 3ft}{1ft}\right) = 1.4ft.$
* Scour depth $= D_{MXB} - D_{MNC} = 1.4 - 0.5 = 0.9ft.$

Total Rock Weir Height

* Assume height above bed = 0.5ft
* Minimum embedment = 0.9 ft
* Use 18" base stone to prevent piping & increase stability

Rock Weir Geometry

Crest Width $= 2D_{50} \approx 1.5ft$
$R = 1.625ft.$

* Provide bank stabilization up ravine slope 3' either side
* Last weir tapers at 1:10

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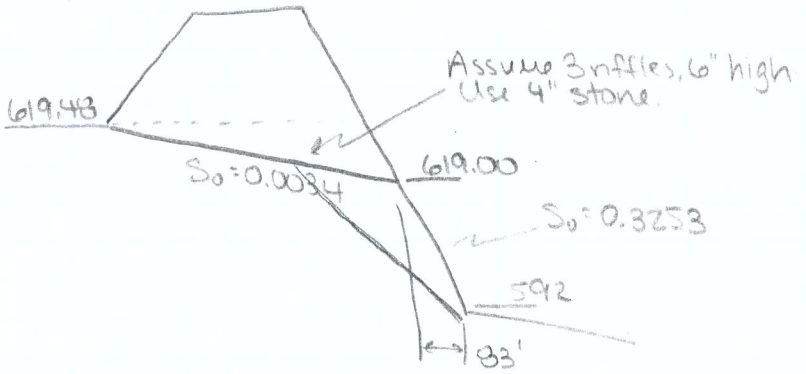
If Fill Excavated & Ravine Floor NOT dropped (Earth Plug Only)



Using straight chute, from EM1110-2-1601, Eqn. 3-5 (Steep Slope Riprap)

$$D_{30} = \frac{1.95 S_o^{0.555} q^{2/3}}{g^{1/3}} = \frac{1.95(0.3253)(3.7\,ft^2/s)^{2/3}}{(32.2\,ft/s^2)^{1/3}} = 0.79\,ft$$

$\approx 1\,ft$

Note: This value exceeds the permissible slope for this equation's applicability. Stone would therefore need to be grouted in place

If Tunnel Pushed & Ravine Floor NOT dropped (Sloped at Culvert)

* Use Stone Weir design (weirs go in tunnel)

If Tunnel Pushed & Ravine Floor NOT dropped (Earth Plug Only)

* Use Grouted Cobble Chute

No Plug Removal & Shallow Slope w/ Excavation

* Riffle height & spacing will depend on depth of excavation
* Assume depth = 10ft & height = 12" w/ 50ft spacing
  ∴ 500ft long excavation (need 10 riffle)
* Stabilize slopes of side/rear walls (1V:3H or 1V:4H ?)
* Assume D_{50} = 6" stone
* Assume width = 10ft
* Situate toe of last riffle at mouth of culvert.

No Plug Removal & Steep Slope w/ Wing Walls

* Assume depth = 10ft
* Estimate q ... assume width = 5'

$$q = \frac{Q}{b} = \frac{18.5\,cfs}{5ft} = 3.7\,\frac{ft^2}{s}$$

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From EM 1110-2-1601, Eqn. 3-5 (Steep Slope Riprap)

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}} = \frac{1.95(0.2)^{0.555}\left(3.7\frac{ft^2}{s}\right)^{2/3}}{\left(32.2\frac{ft}{s^2}\right)^{1/3}} = 0.6\,ft.$$

From EM 1110-2-1601, Eqn. 3-3 (Basic Stone Sizing)

$V_{max} = 3\frac{ft}{s}$ $\quad V_d = 1.6V = 4.8\frac{ft}{s}$

$$D_{30} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2} \frac{V}{\sqrt{Kgd'}}\right]^{2.5} = 1.25\left[(0.78)\frac{4.8\frac{ft}{s}}{\sqrt{(32.2)(1ft)}}\right]^{2.5} = 0.44\,ft$$

Thickness = $2D_{30} = 2(0.6ft) = 1.2ft$.

+ Provide headwall & wingwall around the culvert inlet

<u>No Plug Removal & Tunnel Installed to Toe of Bluff</u>

Assume tunnel width = 5ft

$Q = 18.5cfs \therefore q = 1.6\left(\frac{Q}{b}\right) = 1.6\left(\frac{18.5cfs}{5ft}\right) = 5.92\frac{ft^2}{s}$

$S_o = \frac{619.48 - 584}{265} = 0.133$

From EM 1110-2-1601 Eqn. 3-5 (Steep slope riprap)

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}} = \frac{1.95(0.133)^{0.555}\left(5.92\frac{ft^2}{s}\right)}{\left(32.2\frac{ft}{s^2}\right)^{1/3}} = 0.65\,ft$$

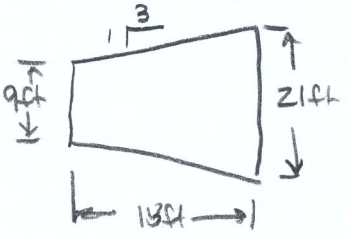
From EM-1110-2-1601, Eqn. 3-3 (Basic Stone Sizing)
Using Manning's equation, $D = 5'$, $Q = 18.5\,cfs$, $S_o = 0.133$, $n = 0.033$
$d_N = 2.35ft$, $A_N = 3.022ft^2$ $V_N = 2.04\frac{ft}{s}$

$V_d = 1.6\,V_N = 1.6(2.04\frac{ft}{s}) = 3.2\frac{ft}{s}$

$$D_{30} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2} \frac{V}{\sqrt{Kgd'}}\right]^{2.5} = 1.25\left[(0.78)\frac{3.2\frac{ft}{s}}{\sqrt{(32.2\frac{ft}{s^2})(2.35ft)}}\right]^{2.5}$$

$= 0.06\,ft.$

Provide stone apron at base, $D_{50} = 0.65ft$ & thickness = 1 ft



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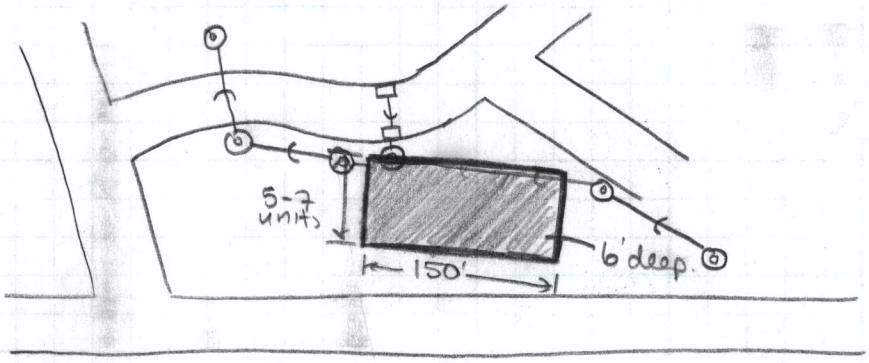
Ravine Head - Option #1

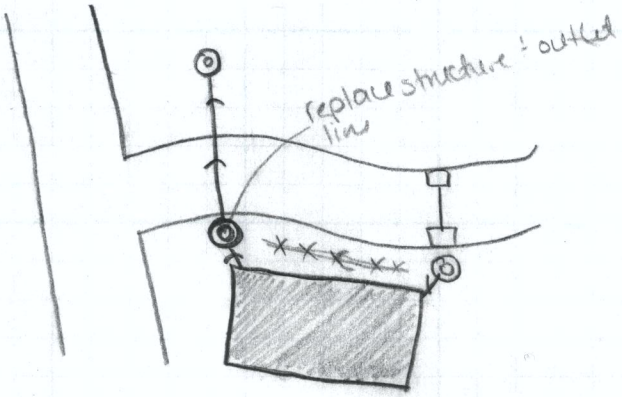
* Assumptions:
  - Assume place structure in 'Pipe 8' of SWMM model
  - Need to provide ~49,200 ft³ of storage
  - From HMS, allowable discharge = 1.2 cfs (2yr-12hr.)
  - If using box culverts, 200' x 10' x 6', would need (4) → 48,000 ft³
    150' x 10' x 6', would need (5) → 45,000 ft³

* Pipe 7 shortened to 100' ∴ outlet invert adjusted to 652.17
* Pipe 7 ∴ 10 routed through storage vault (Pipe 10 kept same)
* Junction 11 relocated to immediately adjacent to prior location
  ↳ 4" outlet connected to base
  ↳ 12" overflow provided ~1' below top (@ 657.73 - 1.0')



Ravine Head - Option #2

* Use (7) 8x10 culverts 90' long set on end
* Replace outlet structure; set invest to 644.9
  ↳ 4" outlet at base
  ↳ 12" outlet 2ft. below top
* Uses 79% of volume during 100-year 24-hour event



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Plunge Pool at Head for Option #1 or #2

* Find peak velocity from each line (100yr-6hr)
  → Bridge = 11.52 ft/s
  → Reved. Outfall = 7.18 ft/s

* Find total peak flow for outlet
  → Bridge = 1.42 cfs
  → Reved Outfall = 2.02 cfs

* Size based on Velocity
  $V_d = 1.2V = 1.2(11.52\ ft/s) = 13.8\ ft/s$

  From HEC-14, Eqn. D.3

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left[\frac{(13.8\ ft/s)^2}{2(32.2\ ft/s^2)}\right] = 1.25\ ft \approx 15"$$

* Size based on Flow
  $Q_d = 1.2Q = 1.2(8.5\ cfs) = 4.2\ cfs$

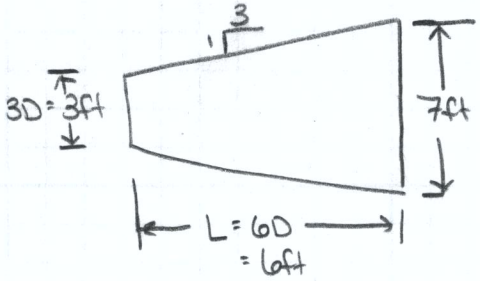
  From HEC-14, Eqn. D.5

$$D_{50} = 0.2D\left[\frac{Q}{\sqrt{g}\,D^{2.5}}\right]^{4/3}\left(\frac{D}{TW}\right) = 0.02D\left[\frac{Q}{D^{2.5}}\right]^{4/3}\left(\frac{D}{0.4D}\right)$$

$$= 0.02(1.5ft)\left[\frac{(4.2\ cfs)}{(1.5ft)^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right) = 0.13\ ft$$

→ Use $D_{50L} = 1.25ft$
$W_{50L} = 322\ lbs$

* Thickness = D_{100} or $1.5D_{50\text{-}u} \approx 2.5ft$
* Calculate length using HEC-14 standard



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Riffle Placement

* Channel velocities from XS 1120 to 431 warrant use of riffles
* Channel incision warrants use to head

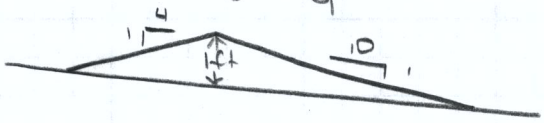
Current Slope ~ $\frac{23}{639} = 0.036$ / Target Slope = $\frac{612.4 - 605.8}{430.93 - 141.6} = \frac{6.6}{289.4} = 0.023$

From EMSR-4-XX,

$H = (S_o - S_f)X = (0.036 - 0.023)(639) = 8.3$ ft.

No. of structures = $\frac{H}{h} = \frac{8.3\text{ ft}}{1\text{ ft}} = 8.3 \rightarrow$ use 9

Required Spacing = $\frac{639}{9} = 71$ ft.



Width ~ 10-15'

* Estimated stone size

$V_d = 1.6V = (1.6)(3.0\text{ ft/s}) = 4.8\text{ ft/s}$

$$D_{30} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\left(\frac{V}{\sqrt{kgd}}\right)\right]^{2.5} = 0.375\left[\left(\frac{62.4}{165 - 62.4}\right)^{0.5}\left(\frac{4.8}{\sqrt{32.2 \cdot 0.5}}\right)\right]^{2.5}$$

$$= 0.375\left[(0.78)(1.20)\right]^{2.5} = 0.32\text{ ft.}$$

* Use an additional (4) to raise the channel bed from the mouth to XS 132

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Design of Stone Weirs at Mouth

Assumptions:
* Begin weirs at XS 192 → length ~ 183 ft
* Elev. drop = 605.56 - 582.77 = 22.8 ft.
* Length btwn structures ~ 8 ft.

Estimate Required Stone Size

* Use $V = 5$ ft/s → $V_d = 1.6V = 1.6(5\text{ ft/s}) = 8$ ft/s
* $C = 2(0.3)(1.25)(1)(1)(0.5) = 0.375$

From EM-1110-2-1601 Eqn. 3-3

$$D_{30} = 0.375\left[(0.78)\left(\frac{8\text{ ft/s}}{\sqrt{32.2\frac{\text{ft}}{\text{s}^2}(0.5\text{ft})}}\right)\right]^{2.5} = 1.13\text{ ft} \sim 9$$

Estimate Toe Scour

Channel Width ~ 3ft.

$D_{MNC} = 0.5$ft

$$D_{MXB} = 1.14 D_{MNC}\left(1.72 + \frac{0.0084W}{D_{MNC}}\right)$$

$$= 1.14(1\text{ft})\left(1.72 + \frac{0.0084(3\text{ft})}{(1\text{ft})}\right) = 1.4\text{ft}.$$

Scour Depth = $D_{MXB} - D_{MNC} = 1.4 - 0.5 = 0.9$ft.

Total Rock Weir Height

* Assume height above bed = 0.5ft } use 3' thickness
* Minimum embedment = 0.9 ft
* Use 18" base stone to prevent piping & increase stability

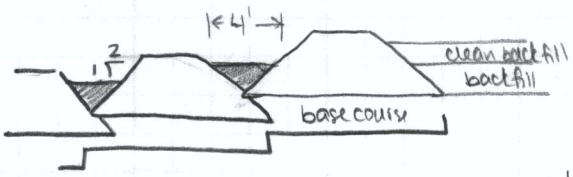
Rock Weir Geometry

Crest Width = $2D_{50} \sim 1.5$ft

R = rock weir radius = $\frac{L^2}{8M} + \frac{M}{2} = \frac{(3\text{ft})^2}{8(2.25)} + \frac{2.25}{2} = 1.625$ft

where L = chord length = 3ft

$M = 3D_{50} = 2.25$ft



* Provide bank stabilization up ravine slope 3' either side
* Last weir tapers at 1:10

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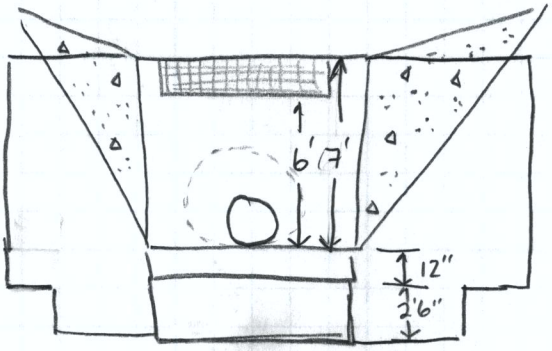
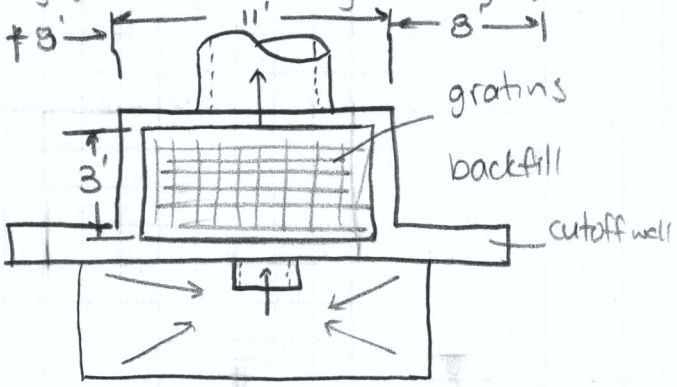
Ravine Head - Option #3

In-Ravine Pipe...
→ Use similar design as to what Greenguard had proposed for Scotts
↳ add plunge pool at head & armor first 100 feet
↳ use junction box at meeting (12" inlet leading to larger)





Detention Feature
→ Intercept outlet pipe of feature & install new discharge pipe down slope
- Cut back a couple feet & run parallel w/ ravine wall intercept at bottom w/ second structure
- Directionally drill new line from basin structure directly down.

Riprap at Head of Ravine

*Size based on velocity

$V_{max} = 12 ft/s \quad V_d = 1.2(12 ft/s) = 14.4 \frac{ft}{s} \sim 15 \frac{ft}{s}$

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left[\frac{(15 ft/s)^2}{2(32.2 ft/s^2)}\right] = 1.46 ft$$

use $D_{50} = 1.5 ft$

*Size based on flow

$Q = 4.2 cfs \quad Q_d = 1.2Q = 5 cfs$

$$D_{30} = 0.02D\left[\frac{Q}{D^{2.5}}\right]^{4/3}\left(\frac{D}{0.4D}\right)$$

$$= 0.02(1.5ft)\left[\frac{5 cfs}{(1.5ft)^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right) = 0.2 ft$$

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Stone Apron at Outfall

* Size Based on velocity

$V = 12.5 ft/s \quad V_d = 1.2V = 15 ft/s$

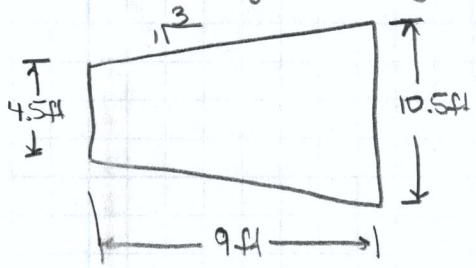
From HEC-14, Eqn. D.3

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{15^2}{2 \cdot 32.2}\right) = 1.5 ft$$

→ Use $D_{50L} = 1.5 ft$

$W_{50L} = 560^{\#}$

* Thickness = D_{100} or $1.5D_{50} \approx 3.75 ft$
* Calculate length using HEC-14 standard



Note: This option discharges directly to the beach! The stone weirs would still be necessary to protect the slope from eroding & exposing the pipe.

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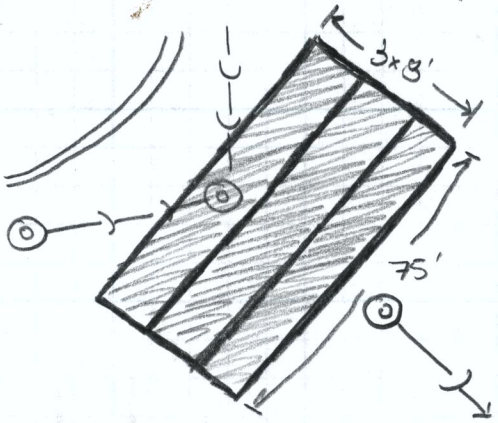
Sewer at Mouth of Ravine

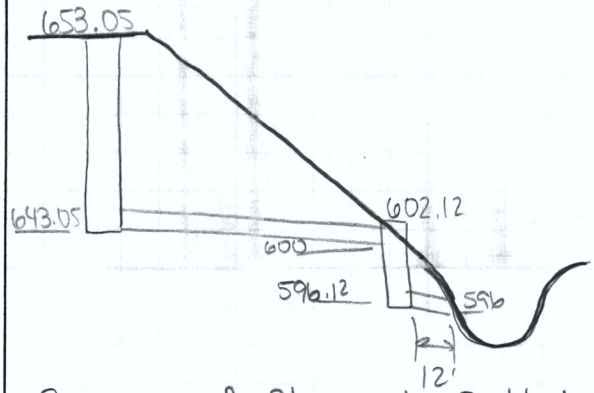
* Size for 100-year, 24-hour event

$Q_{max} = 3.3$ cfs

From EPA SWMM model,



* Provide (3) 3'×10' box culverts, 75' long
* 60" primary outlet
* 12" overflow 2.5' from top
  ↳ highest reached was +7.3' while weir placing at +7.5'
* Provide manhole immediately outside
* $Q_{max-d} = 2.60$ cfs



* Locate structure at bottom to dissipate energy
* Locate at or near start of stone weir.

Sizing of Stone at Outlet

$V_d = 1.6V = 1.6(5\,ft/s) = 8\,ft/s$

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{8\,ft^2/s^2}{2 \cdot 32.2\,ft/s^2}\right) = 0.5\,ft$$

* Position stone apron up opposite side two feet (slope protection)
  Apron width at least 3' if at stone weir.

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Sewer at Mouth of Ravine

From SWMM model, $Q_{max} = 5.5$ cfs

* Assume that existing manhole removed & replaced w/ new one
* New INV = 628.05
  New RIM = 653.05
* Pipe slope needed $\approx \frac{628 - 594}{265} = 26.7\%$

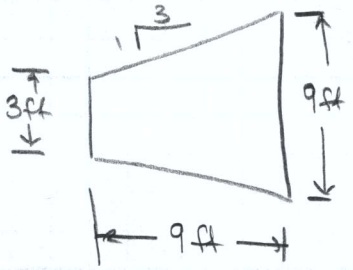
Riprap after outlet

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{15\,ft^2/s^2}{2 \cdot 32.2\,ft/s^2}\right) = 1.5\,ft$$

$$D_{50} = 0.02D\left(\frac{Q}{D^{2.5}}\right)^{4/3}\left(\frac{1}{0.4}\right)$$

↳ adjust for subcritical flow $D_{adj} = \frac{D+y_n}{2} = \frac{1+0.3747}{2} = 0.69$ ft.

$$= 0.02(0.69)\left[\frac{5.5\,cfs}{(0.69)^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right) = 1.14\,ft.$$



Thickness = 1.5D = 2.25ft.

Note: This design discharges to beach. Per local sponsors, the option is not desirable if others are available.

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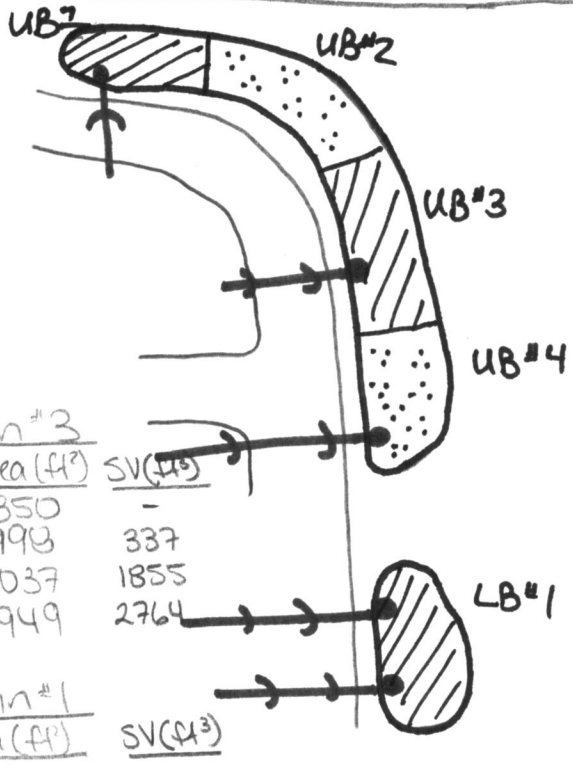
Inner Basin - Eroded Slope : Swale Compartmentalization

* Estimate Storage Volumes



Upper Basin #1

Elev	Area (ft²)	SV (ft³)
654	579	–
655	1,606	1,093
656	2,985	3,388
656.5	3,573	3,935
657	4,255	3,597

Upper Basin #2

Elev	Area (ft²)	SV (ft³)
649.5	266	–
650	1004	318
651	2225	1932
651.5	2851	2884

Upper Basin #3

Elev	Area (ft²)	SV (ft³)
649.5	350	–
650	998	337
651	2037	1855
651.5	2949	2764

Upper Basin #4

Elev	Area (ft²)	SV (ft³)
649.5	221	–
650	854	269
650.5	1454	846
651	2104	1467

Lower Basin #1

Elev	Area (ft²)	SV (ft³)
650	518	–
650.5	1030	387
651	1560	10355
651.5	1975	1531
652	2426	1984
652.5	2947	2444

From "Rock Ramp Design Guidelines", US Dept. of Interior Technical Service Center 9/2007

$V_i = 0.232\sqrt{g \cdot D_{10} \cdot S_o}$ where V_i = interstitial velocity

→ refine later w/ $V_i = n_p \left(S_o g \frac{D_{50}}{K'}\right)^{1/2}$

For now assume D_{50} = 3" stone → 49#
W_{15L} = 15 lb. → 6" stone

Estimate Interstitial Velocity

$D_{10} \sim 6"$
$S_o \sim 0.009$

$V_i = 0.232\sqrt{(32.2\ ft/s^2)(6\ in)(0.009)}$

$= 0.31\ ft/s$

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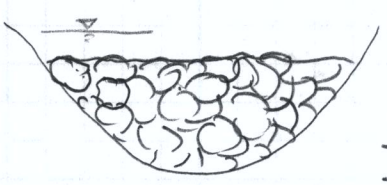
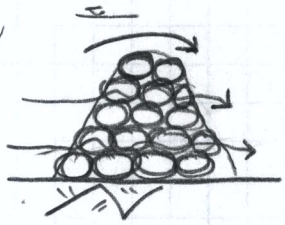
* Estimate flow over weir.

$$Q_{weir} = 0.433\sqrt{2g}\left(\frac{d_1}{d_1+h}\right)^{1/2} L H^{3/2}$$

$$= 0.433\sqrt{2(32.2\,ft/s^2)}\left(\frac{1.25\,ft}{2.25\,ft}\right)^{1/2}(15.5\,ft)(0.25)^{3/2}$$ ← for example

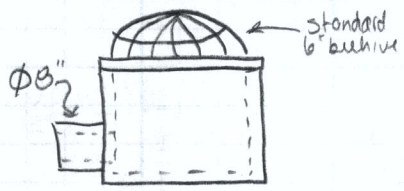
$$= 6.36\ cfs.$$





H	A	Q_{int}	Q_{weir}	Q_{tot}
0	-	0	-	0
0.5	3.6	1.1	-	1.1
1.0	9.32	2.9	-	2.9
1.25	12.99	3.95	5.02	8.97
1.5	16.46	5.04	14.75	19.8

New Outlet Structure



H	Q_{out}	Q_{weir}	Q_{tot}
0	0	-	0
0.75	1.121	-	1.121
1.0	1.418	-	1.418
1.5	1.876	-	1.876
1.75	2.067	2.96	5.027
2.0	2.242	4.50	6.742
2.5	2.556	5.38	7.935

Provide Stone Apron at the Outlet

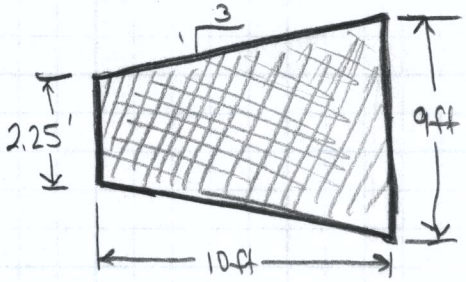
$Q_{max} = 5\ cfs \rightarrow Q_d = 1.2 \cdot 5\ cfs = 6\ cfs$

$V_{max} = 16\ ft/s \rightarrow V_d = 1.2 \cdot 16\ ft/s = 19.2\ ft/s$

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{19.2\,ft^2/s^2}{2 \cdot 32.2\,ft/s^2}\right) = 2.4\,ft$$

$$D_{50} = 0.02D\left(\frac{Q}{D^{2.5}}\right)^{.43}\left(\frac{1}{0.4}\right) = 0.02(0.75ft)\left[\frac{6\,cfs}{(0.75)^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right)$$

$$= 1\,ft.$$



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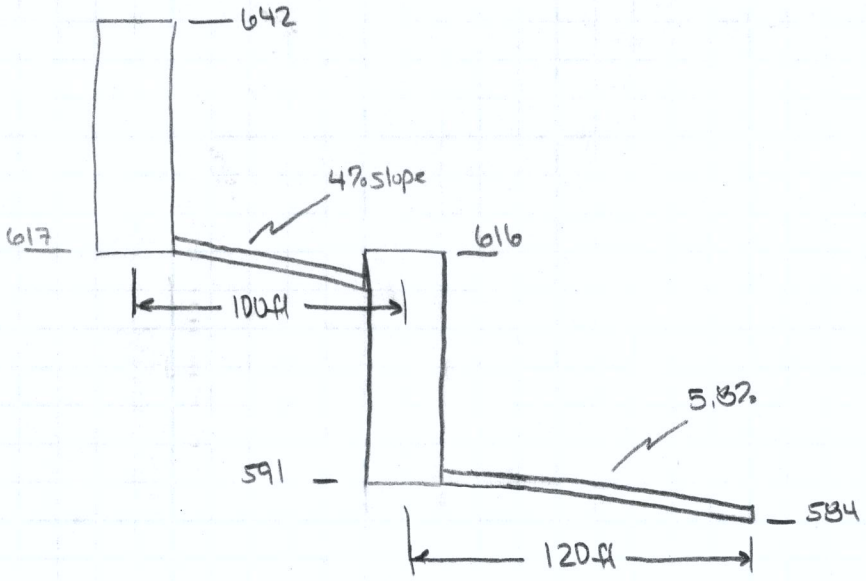
Inner Basin - Eroded Slope: Multiple Drop Structure

* Add two shallow basins to intersect existing pipes leading to outfall.
* Place new structure at point above existing outfalls (Elev ≈ 642)
* Run PVC sewer pipe to new structure.
* Drop first structure 25ft (to 617)
* Run 12" pipe @ 4% slope for 100 ft.
* Install second structure at end of pipe.
* Run 12" pipe @ 5.8% slope to outfall



Riprap Apron

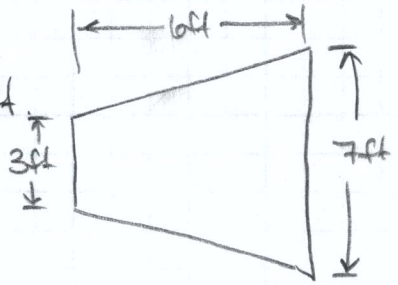
$Q_d = 1.2Q = 1.2(5cfs) = 6cfs \qquad V_d = 1.2V = 1.2(10.7 ft/s) = 12.8 ft/s$

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left[\frac{(12.8 ft/s)^2}{2 \cdot 32.2 ft/s^2}\right] = 1.07 ft$$

$$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{1}{0.4}\right) = 0.02(1ft)\left[\frac{6 cfs}{1^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right) = 0.55 ft.$$

Use $D_{50} = 1.07ft$

Thickness $= 1.5D_{50} = 1.6ft$



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Innes Basin - Eroded Slope: Baffled Chute Design

* Add two small structures to intersect existing pipes leading to outfall
* Discharge to shallow slope chute before entering baffles

Calculate Normal Flow Conditions in Box/Chute

For $Q_d = 2Q = 2(5 cfs) = 10 cfs$, $B = 2ft$ $S_o = 0.3$

$y_n = 0.13 ft$ $V_n = 39.5 ft/s$

* Baffles Necessary!

Verify Approach Conditions

For $Q_d = 10 cfs$, $B = 2ft$, $S_o = 0.005$

$y_{n,box} = 0.98$ $V_{n,box} = 5.1 \frac{ft}{s}$

Verify that $V_c > V_n$...

$q = \frac{Q}{B} = \frac{10 cfs}{2ft} = 5 \frac{cfs}{ft} \rightarrow V_c = (gq)^{1/3} = 5.44 \frac{ft}{s}$

$V_c > V_n$ & $Q << 60 cfs$... baffled chute acceptable.

Estimate Discharge Velocity

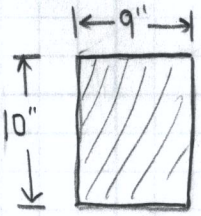
+ Will be no more than 1/3 critical velocity.

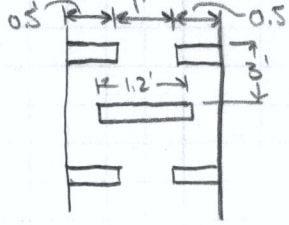
$\frac{1}{3} V_c = 1.8 \frac{ft}{s} << 10 \frac{ft}{s}$

Size Baffle Structure

$y_c = \frac{q}{V_c} = \frac{5 \frac{cfs}{ft}}{5.44 \frac{ft}{s}} = 0.92ft \sim 1ft.$

Baffle Height: $H = 0.8 y_c = 0.8(1ft) = 0.8ft$
Baffle Width: $W = 1.5H = 1.5(0.8ft) = 1.2ft.$
Baffle Spacing = $W = 1.5H = 1.2ft.$
Row Spacing = $2H = 2(0.8ft) = 1.6ft$
↑ can increase, but to no more than 6ft (use 3ft)





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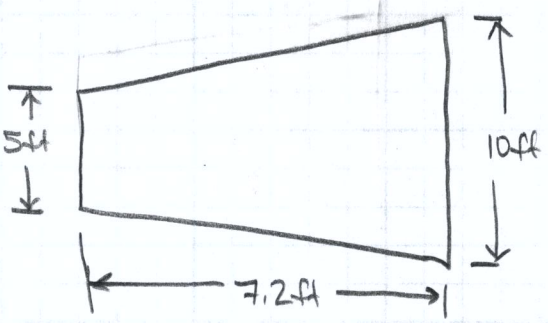
Size Riprap Apron

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) = \frac{0.692}{2.65-1}\left(\frac{2\,\frac{ft}{s}^2}{2 \cdot 32.2\,\frac{ft}{s^2}}\right) = 0.03\,ft.$$

$$D_{50} = 0.02\,D\left(\frac{Q}{D^{2.5}}\right)^{4/3}\left(\frac{1}{0.4}\right) = 0.02(1)\left[\frac{10}{1^{2.5}}\right]^{4/3}\left(\frac{1}{0.4}\right)$$

$$= 1\,ft.$$

Thickness = 1.75 feet ← D_{100} controls