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25 October 2024

TECHNICAL MEMORANDUM

TO: Delia Kaye, Natural Resources Director

FROM: Amy Hunt, PE, Senior Engineer
Erika Towne, PE, Project Engineer

SUBJECT: Warner's Pond Dam Removal Preliminary Design
Impoundment Drawdown Analysis
EA Project No. 64040-01-00-LS

EA Engineering, Science, and Technology, Inc., PBC (EA) was contracted by the Town of Concord (the Town) in September 2023 to prepare preliminary engineering designs for the removal of the Warner's Pond dam (see Figure 1 for the site location). EA's scope of services under this contract also includes a general evaluation of up to five alternative/supplemental projects related to the management and/or restoration of Warner's Pond. Based on our discussions with the Town, one such project is implementing a drawdown program at Warner's Pond. The purpose of a drawdown at Warner's Pond would be to manage nuisance aquatic plant growth by lowering water levels in the impoundment and exposing a portion of the pond bottom during winter months. As part of the evaluation of this project idea, the Town requested that EA calculate the potential depth that the impoundment could be lowered via drawdown. This technical memorandum summarizes the methodology and results of the drawdown analysis.

Additional discussion related to considerations for implementing a drawdown program at Warner's Pond is provided in Section 4.4.1.5 of the Warner's Pond Alterantives Analysis Report (EA 2023) and in Section 5.1.5 of the Warner's Pond Watershed Management Plan (ESS Group, Inc. 2012).

DATA SOURCES

The data required for this analysis consists of (1) the flow rate exiting the impoundment at the Warner's Pond dam, (2) the dimensions and elevations of the low-level outlet structures through which flow would exit the impoundment, and (3) bathymetry of the impoundment to visually depict the extent of a drawdown. The following sections describe these data sources.

Flow Rate

Drawdown for aquatic plant management is typically conducted during the winter. A USGS stream gage (01097380) was installed on Nashoba Brook at Commonwealth Ave (approximately 150 feet downstream of the dam) from January 2006 to October 2009; the average discharge in the months of December, January, and February was 101 cfs, 102, cfs, and 161 cfs, respectively (see Attachment A). The average of those values is 121 cfs; this value was used in the analysis as the flow rate exiting the impoundment.

Outlet Structures

Drawdown of an impoundment is achieved by allowing flow to exit the impoundment at an elevation that is lower than the elevation of the structure that controls the normal pool elevation. Therefore, the depth of drawdown is limited by the elevation and capacity of the low-level outlet structure(s). Two low-level outlet structures exist at the Warner's Pond dam: a 5-foot wide sluiceway located on the western edge of the main spillway and a 24" reinforced concrete pipe (RCP) that runs through the earthen berm immediately adjacent to the sluiceway on river left. For the purposes of this analysis, both structures are assumed to be in full working order.

The dimensions of the sluiceway were taken from sheet S1 "Spillway Structural Sections and Details" of the Warners Pond Dam Rehabilitation plan set developed by GZA Environmental, Inc. in November 2006 (see Attachment B). Currently there are stoplogs in place in the sluiceway, and the plans show that removal of all stoplogs results in a sluiceway crest elevation of 115 feet NAVD88. The size, material, location, and invert elevations of the 24" RCP were taken from a survey performed by SGC Engineering, LLC in October 2023. The upstream invert elevation is 113.5 feet NAVD88 and the downstream invert elevation is 113.0 feet NAVD88. See Figure 2 for a schematic of the low-level outlet structures.

Bathymetry

EA collected bathymetry data in Warner's Pond on December 20, 2022 and January 14, 2023 (data collection occurred on non-consecutive days because a portion of the pond was covered in ice during the first day of data collection). Water depth data was collected at a total of 156 stations throughout the pond. At each station, a 16-foot graduated steel rod was lowered from a small boat to the water-sediment interface to assess water depth. The data were then processed using geospatial software to generate one-foot contours for water depth.

DRAWDOWN ANALYSIS

The drawdown analysis for Warner's Pond was completed in Microsoft Excel using engineering principles and guidance in the PE Civil Reference Manual, 16th Edition for calculating flow capacity through pipes and the height of water over a broad-crested weir (the sluiceway) (Lindeburg 2018). The following sections describe the methodology and results of the drawdown analysis for Warner's Pond.

Methodology

The Warner's Pond dam main spillway elevation is 118.77 feet NAVD88; for the purposes of this analysis, EA used this value as the normal impoundment elevation in Warner's Pond. EA also assumed that all flow exits Warner's Pond at the dam; this assumption is supported by the lack of other known outlets from the pond. As discussed above, drawdown of Warner's Pond would be initiated by activating the pond's two low-level outlets (the 24" RCP and the sluiceway at the main spillway). Immediately following implementation of a drawdown at Warner's Pond, the outflow at the dam would exceed the inflow for a period of time until the water surface elevation of the impoundment reaches a new equilibrium. Once the outflow at the dam has stabilized to its new controlling elevation, the rate of flow exiting the pond would again be equal to the flow rate into the pond.

To complete this analysis, EA assumed the 24" RCP would pass flow at its full capacity. The maximum flow capacity of the pipe was calculated using energy and head loss equations as 21.9 cfs (see Attachment C for detailed calculations). This value was rounded down to 21 cfs to include a factor of safety.

EA assumed that all stoplogs would be removed from the sluiceway. The flow through the sluiceway was calculated by subtracting the capacity of the 24" RCP (21 cfs) from the measured flow rate stated previously (121 cfs), which results in a flow of 100 cfs through the sluiceway.

The height of water above the sluiceway invert was calculated using the *flow over the sluiceway in the broad-crested weir* equation. This height was calculated to be 3.3 feet (see Attachment D for detailed calculations). Adding this height to the sluiceway invert elevation of 115 feet NAVD88 results in a water surface elevation under drawdown conditions of 118.3 feet NAVD88. It was assumed that this water surface elevation is consistent throughout the impoundment and represents the impoundment elevation during a drawdown of Warner's Pond, given the assumptions discussed previously.

Results

Based on the spillway elevation of 118.77 feet NAVD88 and the calculated impoundment elevation during drawdown of 118.3 feet NAVD88, the depth by which the impoundment could be lowered when the flow rate exiting the pond is 121 cfs is approximately 0.5 feet. As discussed throughout this technical memorandum, this value is dependent upon several factors; the most variable of these is the flow rate into the pond during drawdown. If the flow rate during drawdown is less than 121 cfs, the depth that the impoundment can be lowered during drawdown increases; conversely, if the flow is greater than 121 cfs, the drawdown depth decreases. Table 1 and Attachment D display calculated drawdown depths for Warner's Pond under a variety of flow conditions.

Table 1. Impoundment Drawdown Depths

Total Flow (cfs)	Flow through Sluiceway (cfs)[2]	Impoundment Elevation (ft, NAVD88)	Depth below Normal Impoundment Elevation (ft)
121[1]	100	118.3	0.5
111	90	118.1	0.7
101	80	117.8	0.9
91	70	117.6	1.2
81	60	117.4	1.4
71	50	117.1	1.7
61	40	116.8	2.0
51	30	116.5	2.3
41	20	116.1	2.6
31	10	115.7	3.1

[1] Average winter flow from stream gage data 2006-2009
[2] Includes 21 CFS through the low level outlet

The potential drawdown depths associated with flows under 31 cfs (with 10 cfs flowing through the sluice way) was not evaluated due to the unlikelihood that flow would be this low and remain this low through the winter months.

Figure 3 displays the approximate portion of the pond bottom that would be exposed during a 3-foot drawdown of Warner's Pond. As displayed in Table 1, a 3-foot drawdown at Warner's Pond could be achieved if flow rates were approximately 31 cfs (e.g., approximately 25% of the average observed winter flow rate from 2006-2009). As displayed in Figure 3, the largest area of exposed pond bottom is located in the southwestern cove of the pond, where water depths are shallow. A relatively narrow band along the shoreline of the pond and its islands would also be exposed under a 3-foot drawdown scenario. Areas not exposed during drawdown would see no benefit relative to management of nuisance aquatic plants.

Assumptions and Limitations

The following assumptions apply to this drawdown analysis for Warner's Pond:

- the spillway elevation is 118.77 feet;
- all flow exits Warner's Pond at the dam;
- the water surface elevation at the sluiceway is equal to the water surface elevation throughout the impoundment.

The overall feasibility of drawdown at Warner's Pond is also predicated on the assumption that the 24" RCP low-level outlet is functional and that all stoplogs can be removed from the sluiceway.

The potential drawdown depth of Warner's Pond as calculated through this analysis is not necessarily equal to the drawdown depth that could realistically be achieved. The drawdown depth is driven by the amount of flow exiting the impoundment, which is constantly changing and difficult to predict to an exact value due to meteorologic factors including precipitation rates and frequency of storm events.

References

Lindeburg, Michael R. 2018. PE Civil Reference Manual. 16th Edition. Professional Publications, Inc., Belmont, CA.

Figures

1	Site Location
2	Outlet Structures
3	Water Depth with 3-foot Drawdown Impact Area

Attachments

A	USGS Stream Gage Data
B	Spillway Structural Sections and Dimensions
C	Pipe Capacity Calculations
D	Drawdown Depth Calculations

Figures

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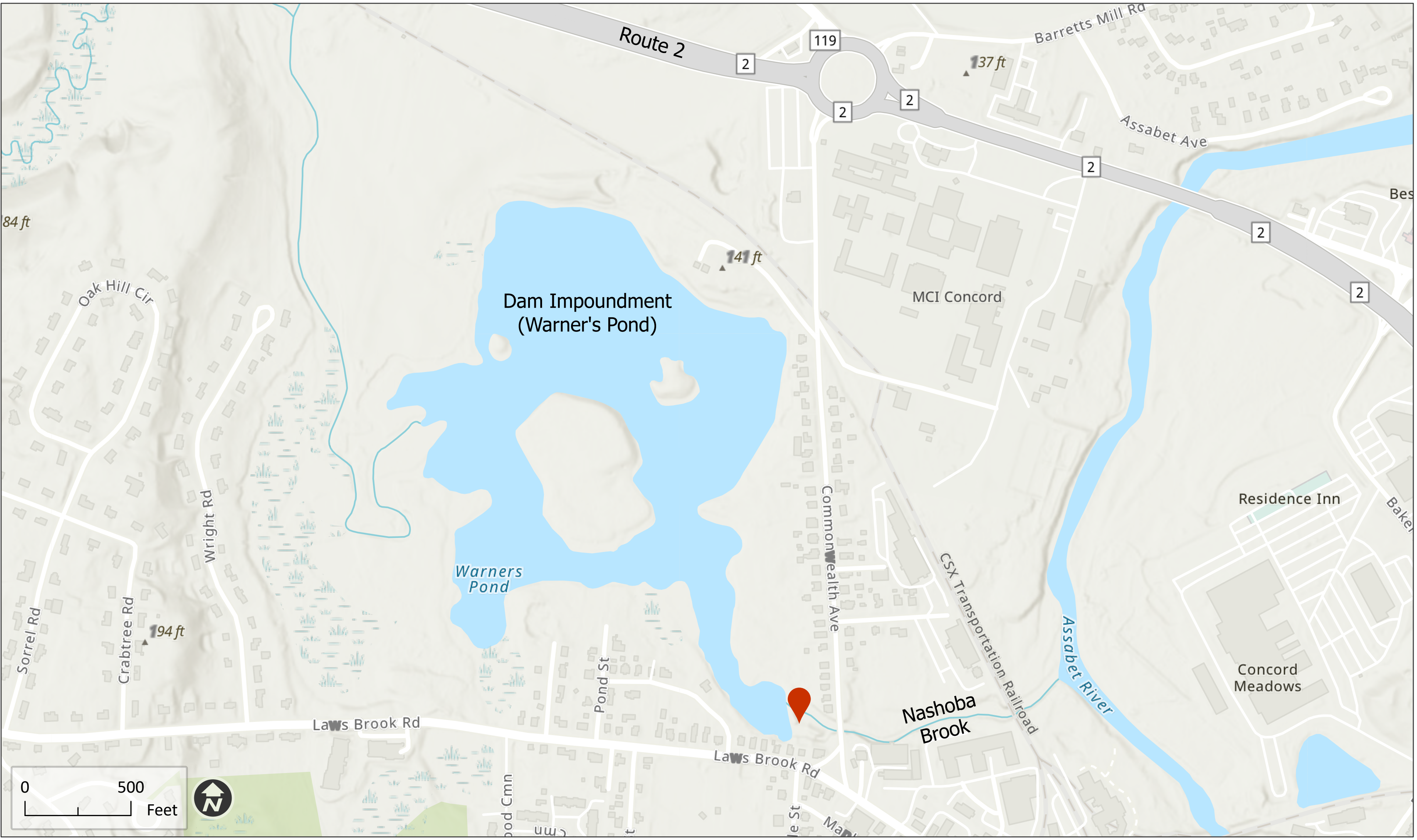

● Dam Location

Figure 1
Site Location

Warner's Pond
Drawdown Analysis
Concord, Massachusetts

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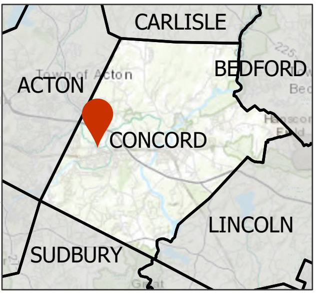

▬	24" RCP
🟨	Sluiceway
🟦	Main Spillway
⬜	Concrete Wall

Data Source: Survey by SCG Engineering, LLC,
Spillway Details Drawing by GZA Environmental, Inc.

Figure 2
Outlet Structures

Warner's Pond
Drawdown Analysis
Concord, Massachusetts

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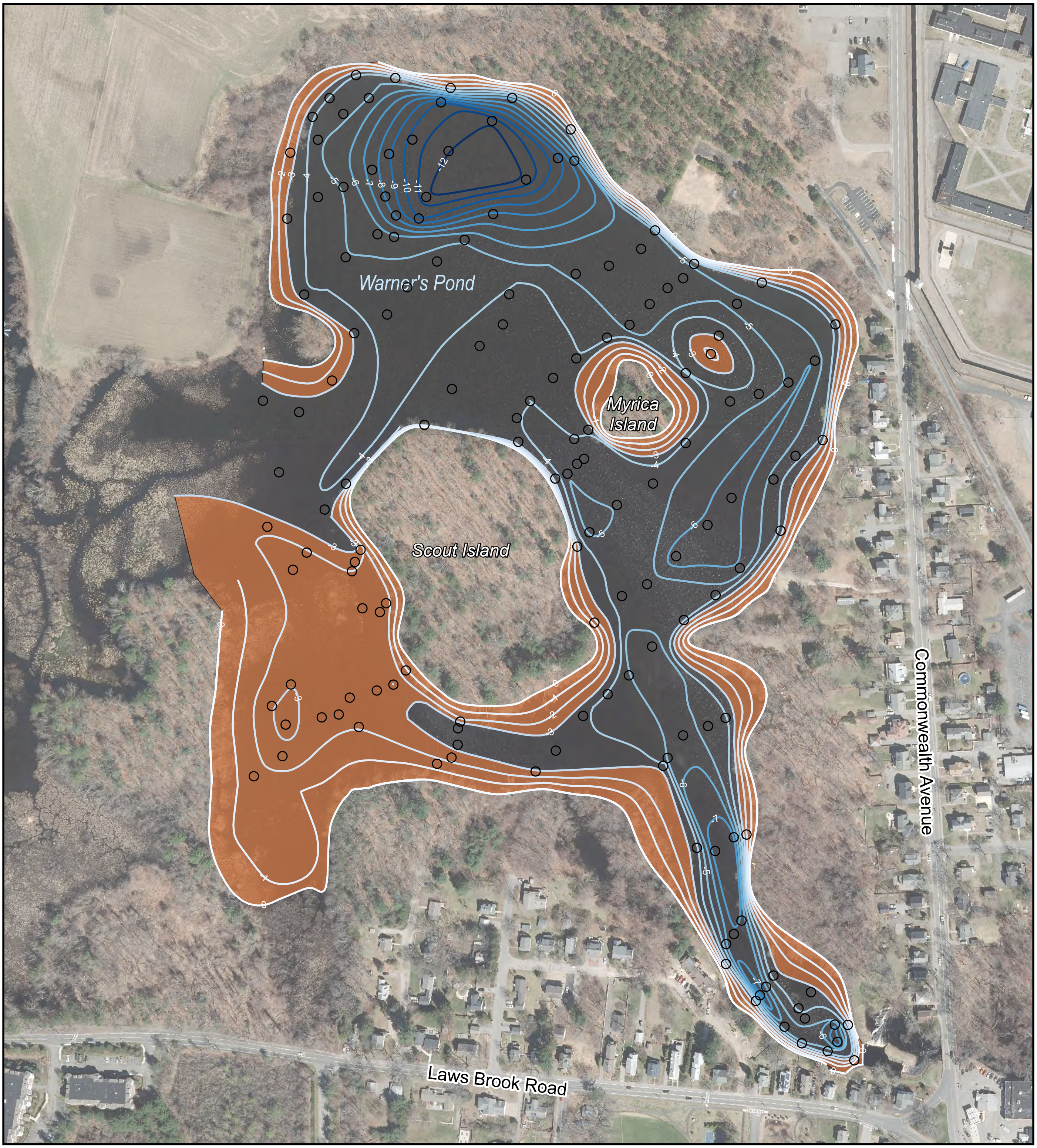
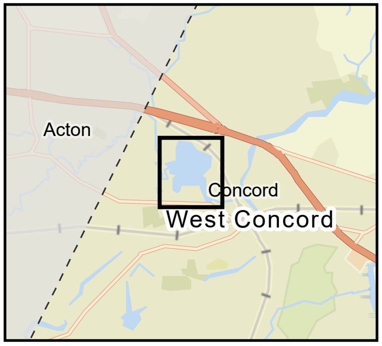


VICINITY MAP



Legend

○ Bathymetric / Water Depth Sample Point

▨ Draw Down Impact Area 3 feet

Water Depth Contour (feet)

— -12
— -11
— -10
— -9
— -8
— -7
— -6
— -5
— -4
— -3
— -2
— -1
— 0

Figure 3
Water Depth with 3-foot Draw Down Impact Area

Concord, MA

Map Date: 10/21/2024
Source: MassGIS 2022
Projection: NAD 1983 UTM Zone 19N

N

0 200 400
Feet

Attachment A

USGS Stream Gage Data

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01097380 NASHOBA BROOK AT COMMONWEALTH AVE AT WEST CONCORD, MA

MERRIMACK RIVER BASIN
CONCORD RIVER SUBBASIN; ASSABET RIVER SUBBASIN

LOCATION.--Lat 42°27′32″, long 71°23′50″ referenced to North American Datum of 1983, Middlesex County, MA, Hydrologic Unit 01070005, stage sensor on right upstream side of Commonwealth Ave. bridge, 100 ft downstream from Warners Pond Dam, 0.2 mi upstream from mouth at Assabet River, at West Concord.

DRAINAGE AREA.--48.0 mi^2.

SURFACE-WATER RECORDS

PERIOD OF RECORD.--June 2006 to October 2009 (Discontinued).

GAGE.--Water-stage recorder with satellite telemeter. Datum of gage is 120 ft above National Geodetic Vertical Datum of 1929, from topographic map.

COOPERATION.--Massachusetts Department of Conservation and Recreation, Water Resources Commission; Massachusetts Department of Environmental Protection, Office of Watershed Management; and Massachusetts Executive Office of Energy and Environmental Affairs.

REMARKS.--Records fair except those for discharges greater than 100 ft^3/s, which are poor. Possible backwater conditions at high flows from Assabet River 0.2 downstream. Flow affected at times during 2007 and 2008 from Warner Pond Dam reconstruction project.

01097380 NASHOBA BROOK AT COMMONWEALTH AVE AT WEST CONCORD, MA—Continued

DISCHARGE, CUBIC FEET PER SECOND
WATER YEAR OCTOBER 2008 TO SEPTEMBER 2009
DAILY MEAN VALUES

Day	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1	182	56	164	198	105	230	156	61	35	59	198	69
2	158	50	193	195	98	175	137	58	29	103	203	47
3	134	44	182	176	93	155	134	64	24	158	154	32
4	127	41	160	158	91	150	177	61	21	169	108	25
5	109	37	143	147	85	135	185	56	18	125	77	19
6	84	38	118	144	81	129	161	91	17	85	60	15
7	66	56	97	155	76	143	201	137	15	70	48	13
8	53	69	75	194	79	187	211	178	14	97	37	12
9	43	71	65	203	92	248	180	182	15	134	32	11
10	41	60	81	179	93	252	151	155	16	185	27	10
11	41	51	143	162	92	250	143	130	16	166	25	9.5
12	38	45	366	145	119	269	157	103	49	140	32	19
13	36	40	463	133	160	252	151	85	69	105	30	39
14	34	39	408	124	159	208	131	69	79	73	29	40
15	33	41	332	114	142	183	113	61	76	47	28	26
16	36	50	269	114	121	172	102	55	66	34	24	18
17	34	64	232	103	110	163	92	64	48	29	22	14
18	40	84	199	101	100	154	83	64	36	42	17	12
19	39	54	161	123	111	146	79	63	68	40	15	11
20	35	44	117	112	132	141	75	54	90	33	15	9.5
21	54	42	120	102	128	131	113	44	78	30	16	9.1
22	41	38	138	98	124	119	197	37	68	39	18	8.6
23	35	33	135	94	152	109	222	33	76	44	24	8.2
24	32	31	125	90	145	101	176	28	76	144	28	7.6
25	31	81	144	84	132	94	142	26	72	269	27	7.0
26	72	196	183	79	117	89	117	23	78	290	23	6.4
27	126	219	189	76	116	97	99	25	80	213	18	8.0
28	139	179	219	78	187	100	88	30	67	149	17	9.5
29	114	144	285	102	---	108	75	34	67	99	46	16
30	87	121	297	110	---	141	68	44	58	70	106	16
31	68	---	253	110	---	167	---	45	---	87	104	---
Total	2,162	2,118	6,056	4,003	3,240	4,998	4,116	2,160	1,521	3,328	1,608	547.4
Mean	69.7	70.6	195	129	116	161	137	69.7	50.7	107	51.9	18.2
Max	182	219	463	203	187	269	222	182	90	290	203	69
Min	31	31	65	76	76	89	68	23	14	29	15	6.4
Cfsm	1.45	1.47	4.07	2.69	2.41	3.36	2.86	1.45	1.06	2.24	1.08	0.38
In.	1.68	1.64	4.69	3.10	2.51	3.87	3.19	1.67	1.18	2.58	1.25	0.42

STATISTICS OF MONTHLY MEAN DATA FOR WATER YEARS 2006 - 2009, BY WATER YEAR (WY)

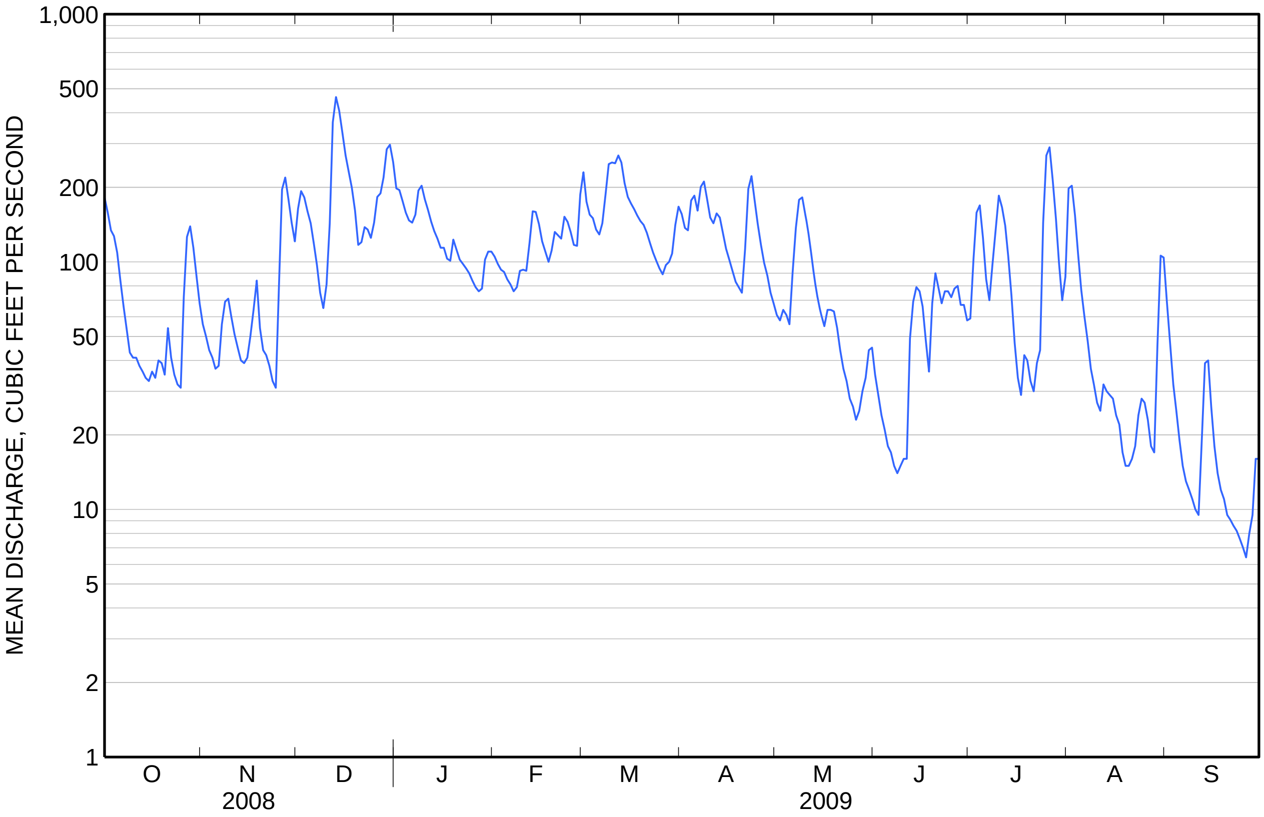
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Mean	40.7	80.8	101	102	161	206	199	96.2	42.3	64.3	41.0	40.3
Max	69.7	152	195	129	329	282	342	148	52.5	107	84.5	120
(WY)	(2009)	(2007)	(2009)	(2009)	(2008)	(2008)	(2007)	(2007)	(2007)	(2009)	(2008)	(2008)
Min	3.68	19.5	23.6	79.5	31.4	161	117	69.7	23.6	11.2	5.98	3.05
(WY)	(2008)	(2008)	(2008)	(2008)	(2007)	(2009)	(2008)	(2009)	(2008)	(2007)	(2007)	(2007)

01097380 NASHOBA BROOK AT COMMONWEALTH AVE AT WEST CONCORD, MA—Continued

SUMMARY STATISTICS

	Calendar Year 2008		Water Year 2009		Water Years 2006 - 2009	
Annual total	46,251.5		35,857.4			
Annual mean	126		98.2		98.7	
Highest annual mean					102	2008
Lowest annual mean					95.9	2007
Highest daily mean	626	Mar 9	463	Dec 13	789	Apr 17, 2007
Lowest daily mean	9.5	Jun 22	6.4	Sep 26	1.1	Oct 9, 2007
Annual seven-day minimum	12	Aug 31	7.8	Sep 21	1.2	Oct 4, 2007
Maximum peak flow			477	Dec 13	819	Apr 17, 2007
Maximum peak stage			6.37	Dec 13	7.59	Apr 17, 2007
Instantaneous low flow			6.2	Sep 26[a]	0.63	Sep 5, 2007
Annual runoff (cfsm)	2.63		2.05		2.06	
Annual runoff (inches)	35.84		27.79		27.95	
10 percent exceeds	285		187		227	
50 percent exceeds	80		85		64	
90 percent exceeds	23		19		9.5	

[a] Also occurred on Sept. 27, 2009.



Attachment B

Spillway Structural Sections and Dimensions

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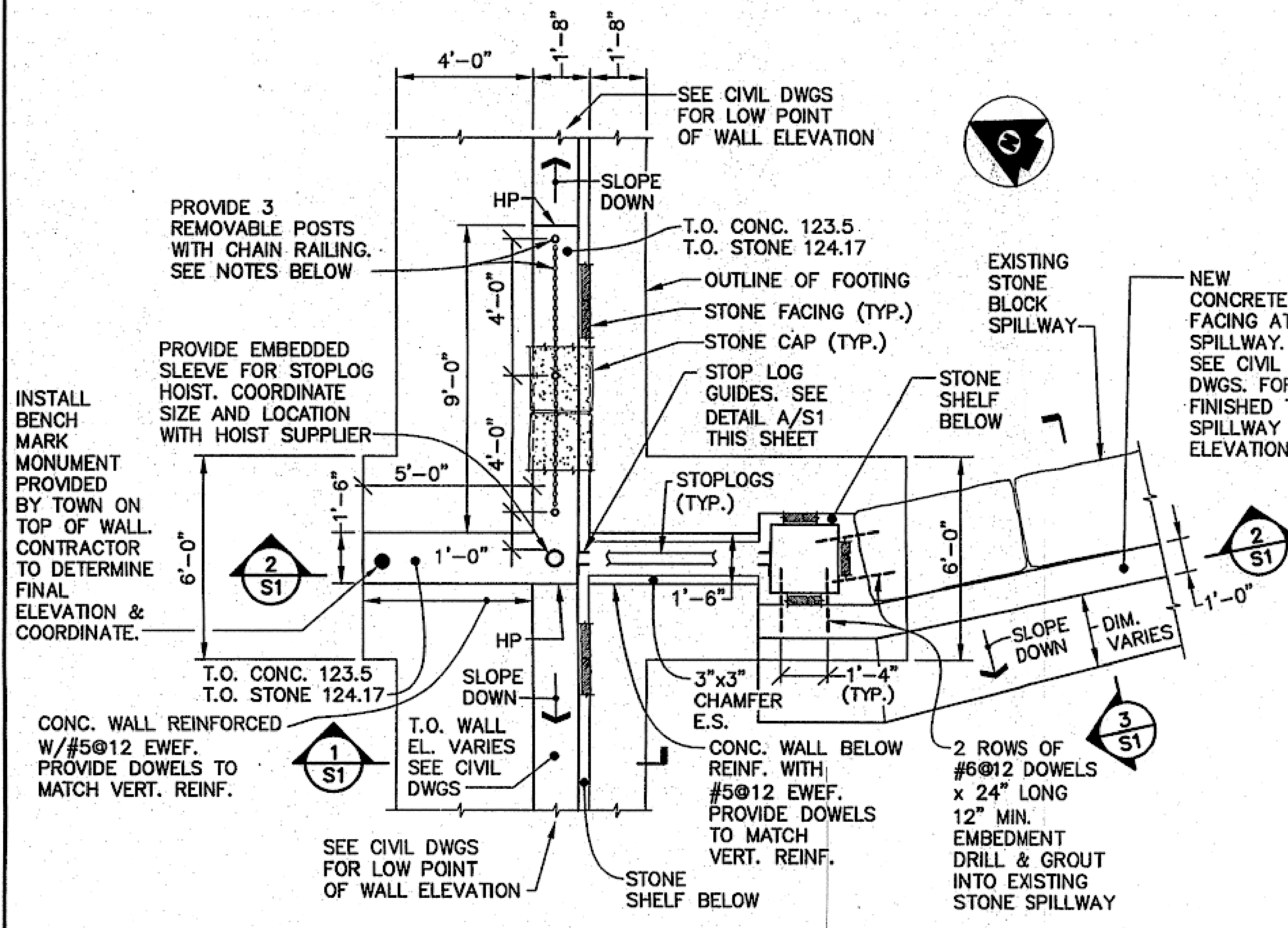
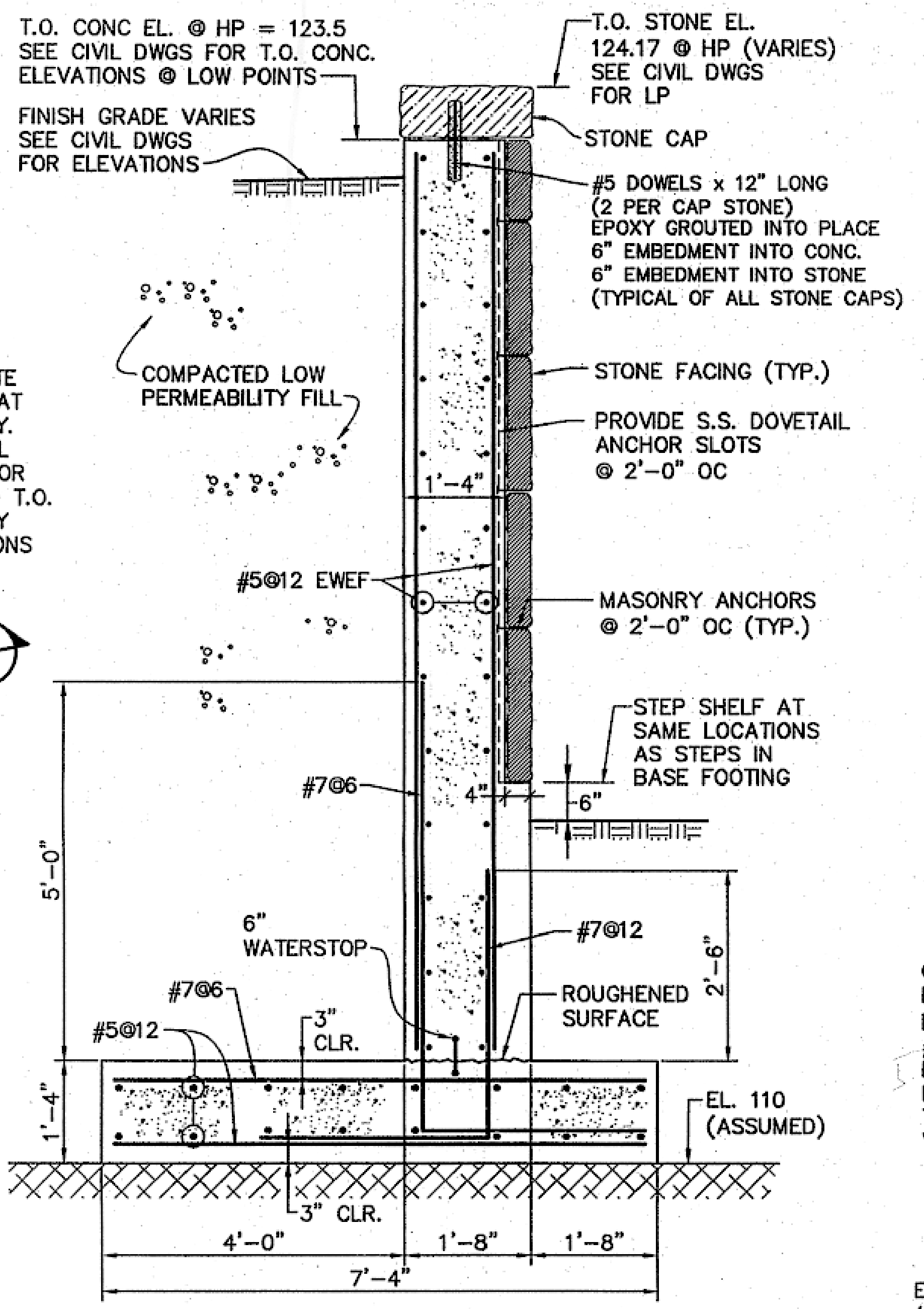


PARTIAL PLAN VIEW @ SLUICEWAY

1/4" = 1'-0"

REMOVABLE RAILING NOTES:

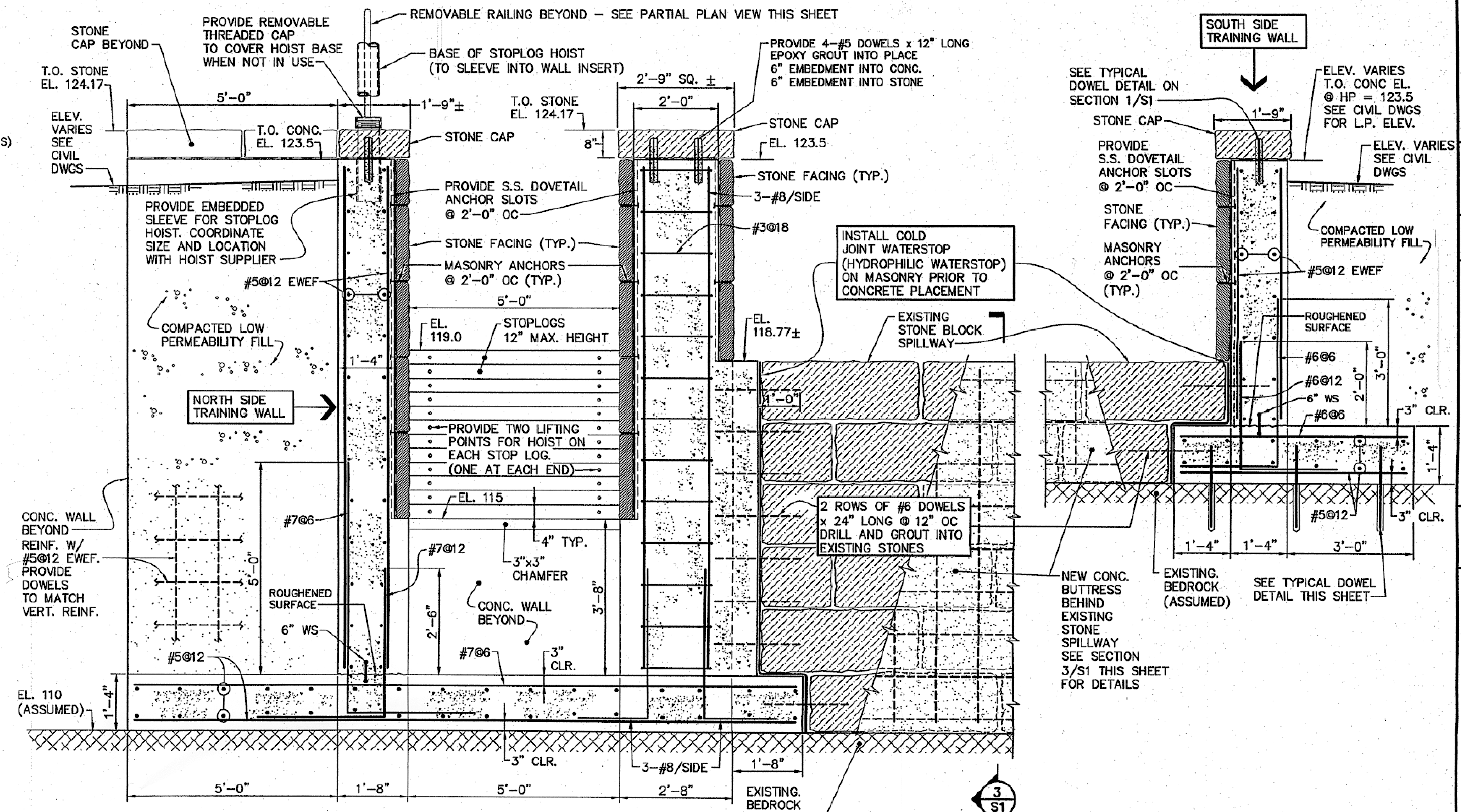
1. POSTS TO BE 2"ø ALUMINUM AND 3'-6" IN HEIGHT ABOVE THE TOP OF WALL.
2. LOCATE CHAIN RAILS AT 1ft, 2ft, AND 3ft ABOVE FINISH GRADE.
3. PROVIDE SLEEVED CONNECTION AT BASE OF RAILING POSTS. PROVIDE 8" MINIMUM EMBEDMENT INTO CONCRETE.
4. EMBED SLEEVED PIPE RECEIVER WITHIN STONE CAP.
5. PROVIDE THREADED CAP TO COVER SLEEVES AT HOIST AND RAILING WHEN NOT IN USE.



SECTION 1/S1

1/2" = 1'-0"

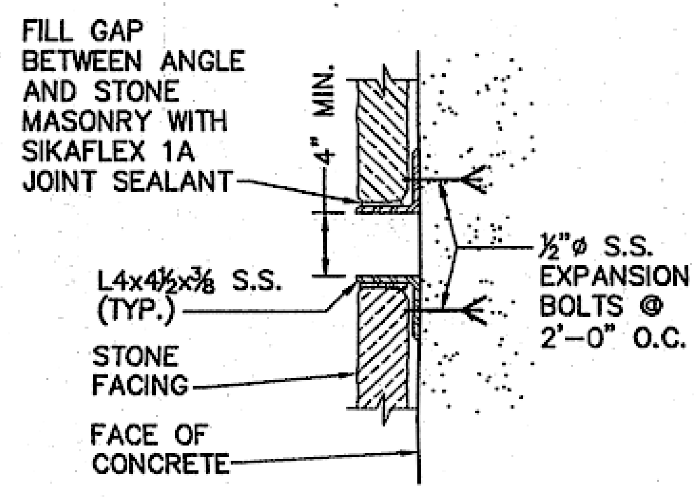
TYPICAL SPILLWAY TRAINING WALL
@ MAIN SPILLWAY AND AUXILIARY SPILLWAY



SECTION 2/S1

1/2" = 1'-0"

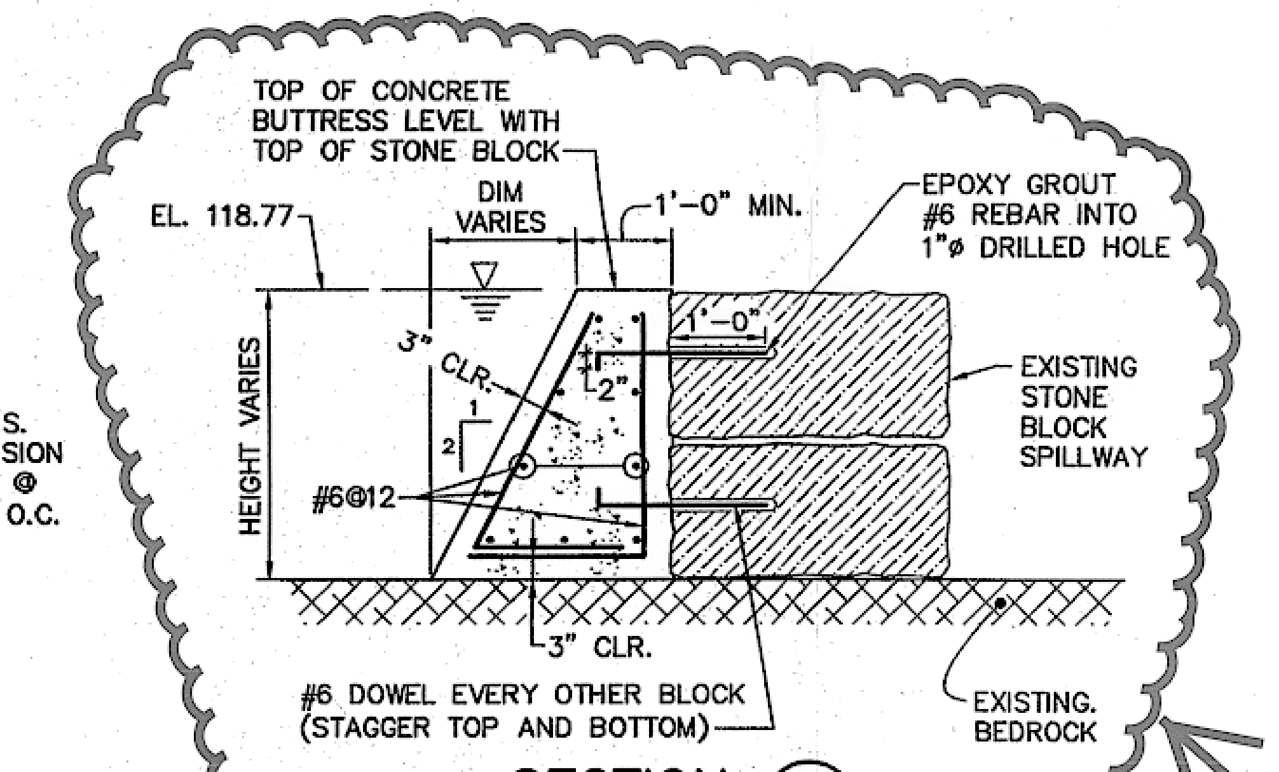
@ MAIN SPILLWAY



DETAIL A/S1

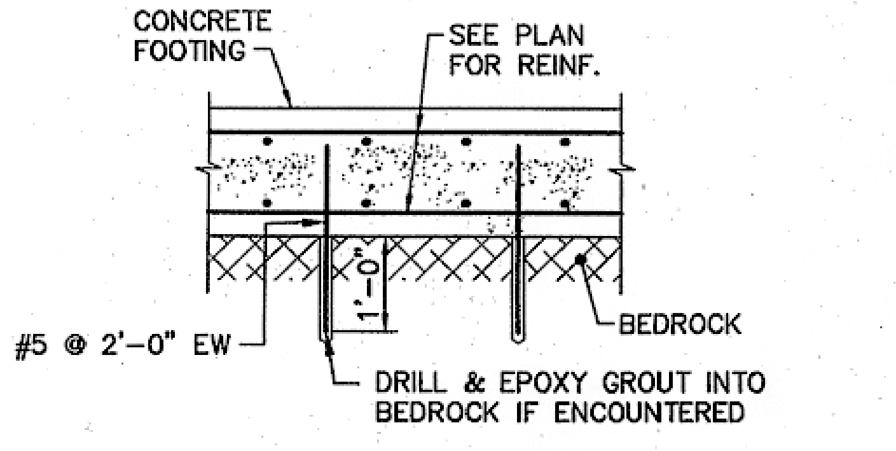
1" = 1'-0"

STOP LOG GUIDE MODIFY PER MANUFACTURER REUQIREMENTS.



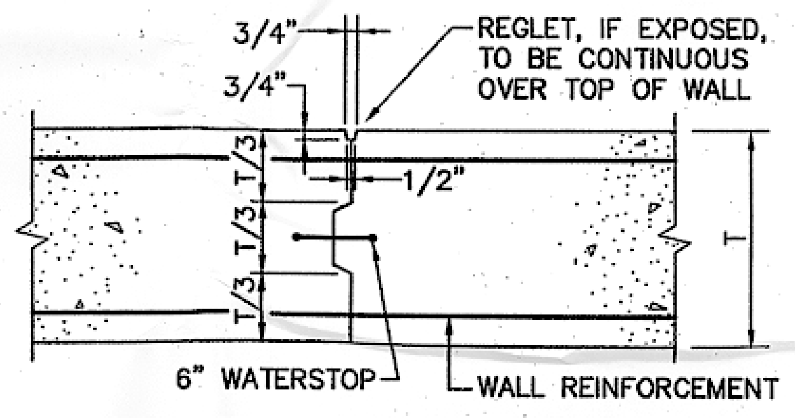
SECTION 3/S1

1/2" = 1'-0"



TYPICAL DOWEL DETAIL

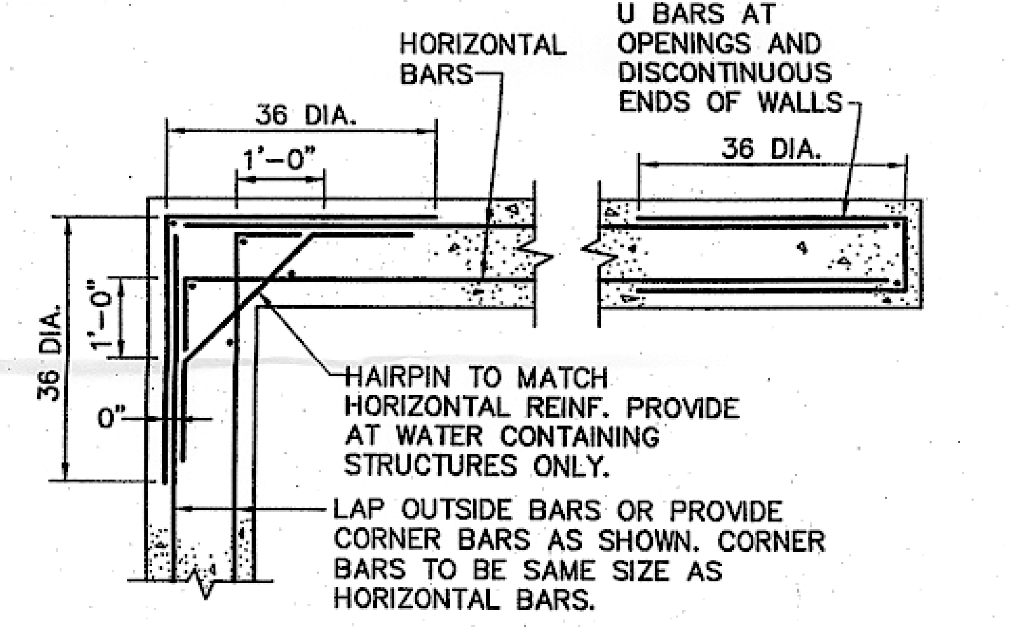
1/2" = 1'-0" (WHERE HIGH BEDROCK IS ENCOUNTERED)



NOTE: MAXIMUM JOINT SPACING = 30 FT.

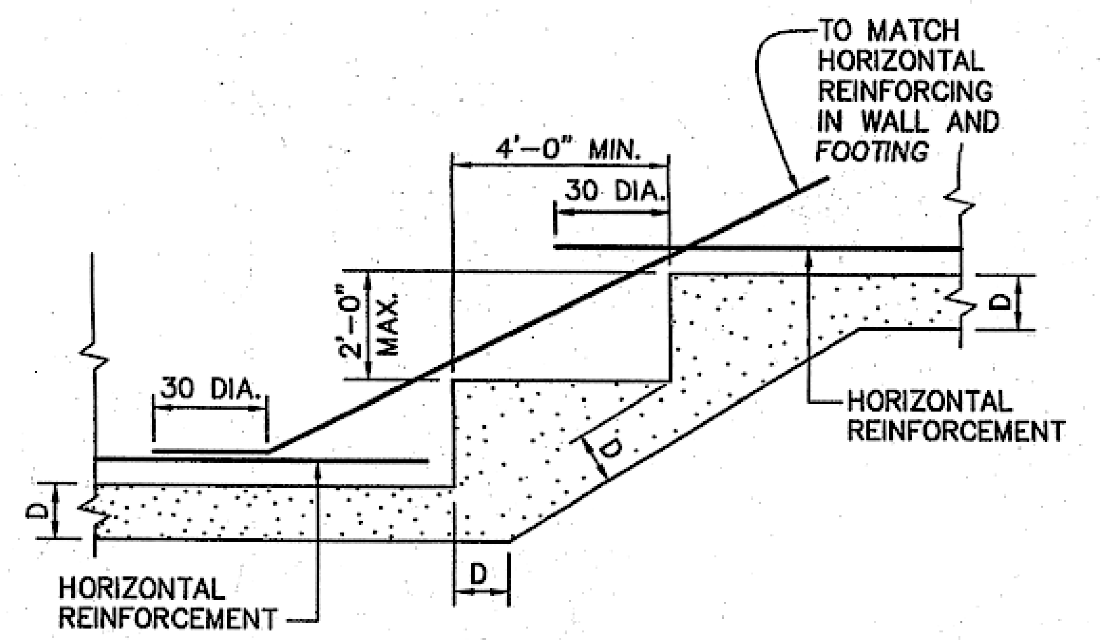
TYPICAL WALL CONSTRUCTION JOINT

NOT TO SCALE



PLAN OF HORIZONTAL REINF. AT CORNERS

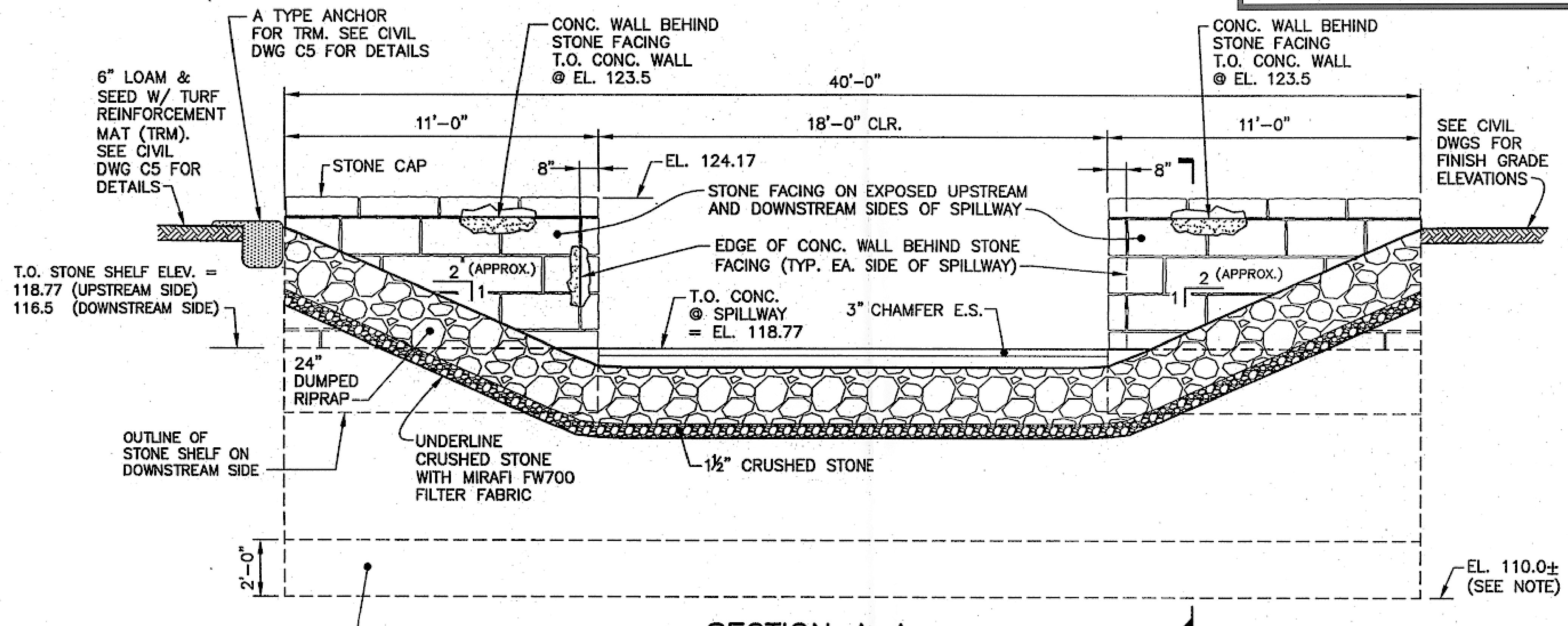
NOT TO SCALE



TYPICAL ELEVATION OF CONTINUOUS STEPPED WALL FOOTING

NOT TO SCALE

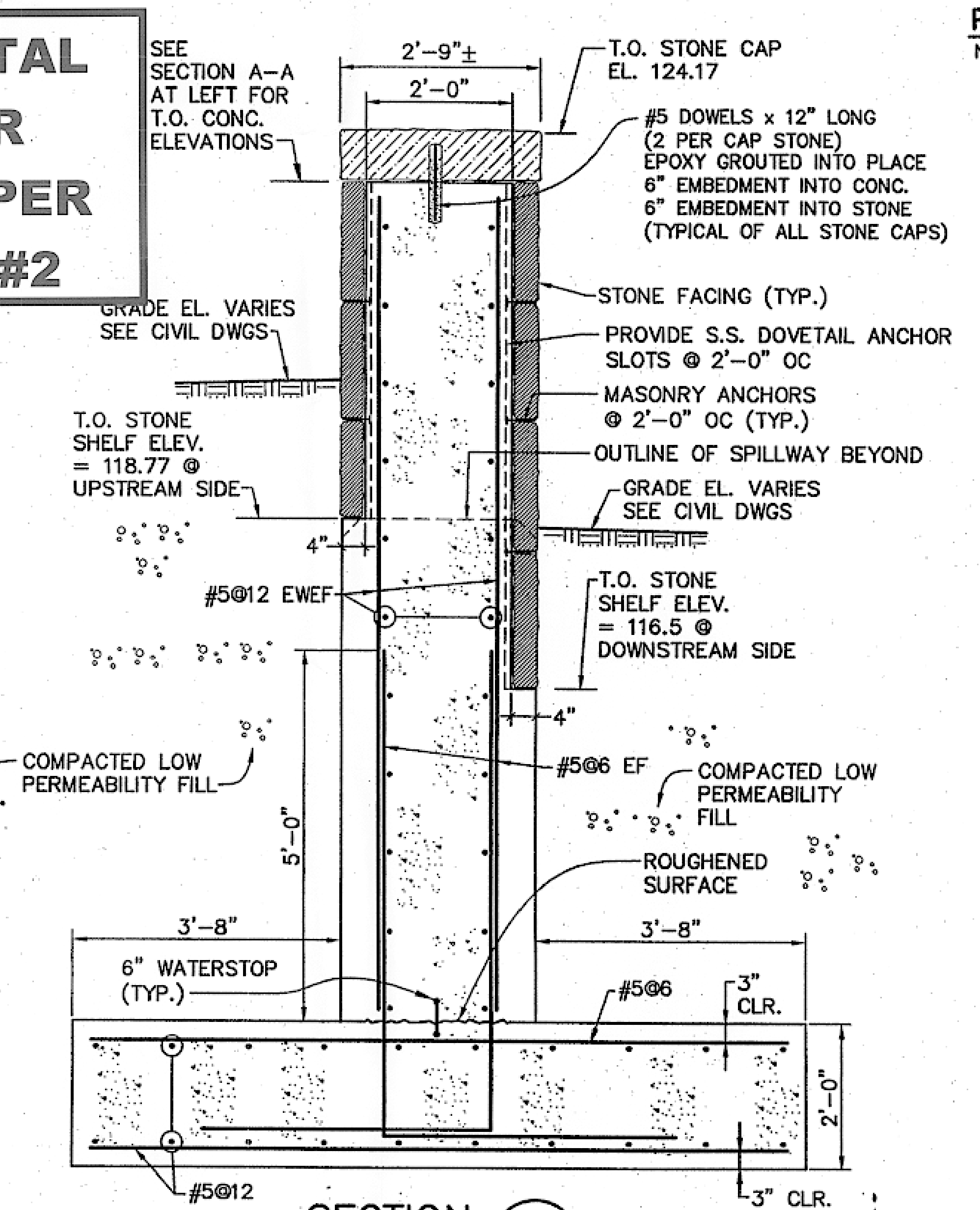
SEE SUPPLEMENTAL SHEET C5B FOR MODIFICATIONS PER CHANGE ORDER #2



SECTION A-A

1/4" = 1'-0" (AUXILIARY SPILLWAY)

NOTE: IF BEDROCK IS ENCOUNTERED HIGHER THAN EL. 110 FT, BEAR FOOTING ON CLEANED, INTACT BEDROCK.



SECTION 4/S1

1/2" = 1'-0"

SECTION AT AUXILIARY SPILLWAY

GENERAL NOTES

FOUNDATIONS

1. NO CONCRETE SHALL BE PLACED IN WATER OR ON FROZEN GROUND.
2. IN GENERAL, EXTERIOR CONSTRUCTION SHALL BE CARRIED DOWN A MINIMUM OF 4'-0" BELOW FINISHED EXTERIOR GRADE.
3. ALL FOUNDATIONS SHALL BEAR ON UNDISTURBED SOIL WITH A BEARING CAPACITY OF NOT LESS THAN 2 TONS PER SQUARE FOOT.
4. ALL FINISHED EXCAVATIONS SHALL BE INSPECTED AND APPROVED BY THE ENGINEER OR HIS DESIGNEE BEFORE ANY CONCRETE IS PLACED.
5. ALL BACKFILL UNDER OR ADJACENT TO ANY PORTION OF THE STRUCTURES SHALL BE COMPACTED IN 12" LIFTS. SEE SPECIFICATIONS.
6. IF BEDROCK IS ENCOUNTERED HIGHER THAN EL. 110 FT, BEAR FOOTING ON CLEANED, INTACT BEDROCK.
7. PROVIDE AT LEAST 4 FEET OF FROST PROTECTION / COVER FROM THE BOTTOM OF FOOTING WHEN BEARING ON SOIL OR 2.5 FEET WHEN BEARING ON BEDROCK, WHERE BEDROCK IS LESS THAN 2.5 FEET FROM FINAL GRADE. DOWEL WALL FOOTINGS INTO BEDROCK.

CONCRETE

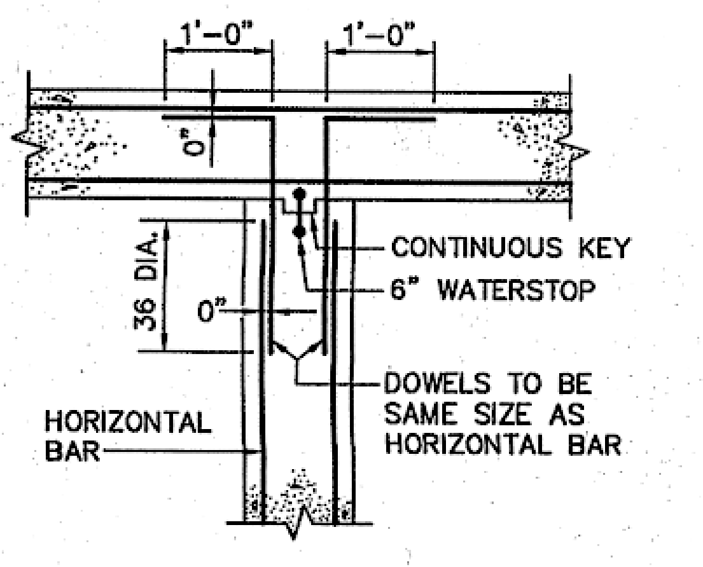
1. ALL CONCRETE WORK SHALL CONFORM TO THE REQUIREMENTS OF THE SPECIFICATIONS, THE LATEST EDITION OF THE ACI BUILDING CODE (ACI 318) AND ACI 301, SPECIFICATIONS FOR STRUCTURAL CONCRETE FOR BUILDINGS, AND TO THE MASSACHUSETTS BUILDING CODE. IN CASE OF CONFLICT, THE MORE STRINGENT REQUIREMENTS SHALL GOVERN.
2. ALL CONCRETE SHALL BE READY-MIXED IN ACCORDANCE WITH ASTM C94.
3. AT LEAST 48 HOURS SHALL ELAPSE BEFORE DEPOSITING OF NEW CONCRETE AGAINST PREVIOUSLY PLACED CONCRETE.
4. ALL CONCRETE SHALL ATTAIN 4,000 PSI MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS.

REINFORCING

1. ALL REINFORCING BAR DETAILS SHALL CONFORM TO THE LATEST ACI CODE AND DETAILING MANUAL, EXCEPT AS OTHERWISE SPECIFIED.
2. ALL REINFORCING BARS SHALL CONFORM TO ASTM A615 GRADE 60 EXCEPT TIES AND STIRRUPS GRADE 40. ALL WELDED WIRE FABRIC SHALL CONFORM TO ASTM A185 AND SHALL BE PROVIDED IN FLAT SHEETS. LAPS SHALL BE STAGGERED AND SHALL BE 1-1/2 FULL MESH MINIMUM.
3. SCHEDULE WITH THE SHOP DRAWINGS ALL NECESSARY ACCESSORIES TO HOLD REINFORCING SECURELY IN POSITION. MINIMUM REQUIREMENTS SHALL BE: HIGH CHAIRS – 4 FEET ON CENTERS; SLAB BOLSTERS – 3'-6" ON CENTERS; SUPPORT BARS FOR HIGH CHAIRS – #5.
4. ALL BARS, EXCEPT AS OTHERWISE NOTED, SHALL BE CONTINUOUS AND SHALL BE RUN CONTINUOUSLY AROUND CORNERS, LAPPED AT NECESSARY SPLICES, AND HOOKED AT DISCONTINUOUS ENDS. LAPS SHALL BE 36-BAR DIAMETER MINIMUM, UNLESS OTHERWISE NOTED.
5. THE CONCRETE PROTECTIVE COVERING FOR MAIN REINFORCEMENT SHALL BE, UNLESS SHOWN OTHERWISE:
   A. FOOTING BOTTOMS...3 INCHES.
   B. FORMED SURFACES IN DIRECT CONTACT WITH SOIL OR EXPOSED TO THE WEATHER... 2 INCHES
6. ALL CONCRETE, UNLESS SPECIFICALLY NOTED TO BE PLAIN CONCRETE, SHALL BE REINFORCED.
7. ALL REINFORCING SHALL BE INSPECTED AND APPROVED BY THE ENGINEER OR HIS DESIGNEE BEFORE CONCRETE IS PLACED.

MISCELLANEOUS

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS AT THE JOB SITE.
2. DO NOT SCALE FROM DRAWINGS.



PLAN OF HORIZONTAL REINFORCING AT INTERSECTIONS

NOT TO SCALE

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PROJ MGR: C. HAKER
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REVIEWED BY: G.R.V./P.B.
OPERATOR: M.D. BECKER
DATE: JULY 2006

WARNERS POND DAM
REHABILITATION
TOWN OF CONCORD, MA

SPILLWAY STRUCTURAL
SECTIONS AND DETAILS

PROJECT NO.
18834.00

DRAWING
S1

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Attachment C

Pipe Capacity Calculations

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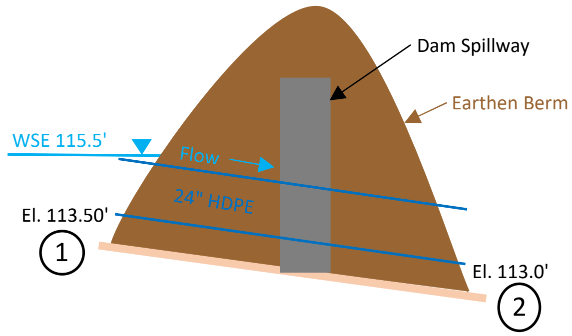
EA Engineering, Science, and Technology, Inc., PBC

CLIENT:	Town of Concord	**Prepared By:**	EJT	**Date:**	9/30/24
PROJECT:	Warner's Pond Dam Removal Preliminary Design	**Reviewed By:**	AEH	**Date:**	10/24/24
NUMBER:	64040-01-00-LS				
SUBJECT:	Pipe Flow Capacity				

Purpose: The flow capacity of the 24" RCP pipe was calculated using the energy and head loss equations. The pipe carries flow starting upstream of the dam, through an earthen berm, and discharging downstream of the dam. The pipe discharges openly at the downstream end. Velocity was determined iteratively, and then used in the continuity equation to determine flow.

Reference: PE Civil Reference Manual, 16th Edition, Michael R. Lindeburg, PE

Diagram:



Solution:

Input Value | Calculation | Conversion/Constant

Step 1. Energy Equation

$$\frac{V_1{}^2}{2g} + \frac{P_1}{\rho} + h_1 = \frac{V_2{}^2}{2g} + \frac{P_2}{\rho} + h_2 + h_L$$

Assume: Pressure at point 2 equals zero because it is taken at open discharge.

Velocity at point 1 equals zero because the velocity of water behind the cofferdam is treated as negligable

$$h_1 = \frac{V_2{}^2}{2g} - \frac{P_1}{\gamma} + h_2 + h_L$$

v_x	Velocity at point x
P_x	Pressure at point x
h_x	Height at point x
h_L	Head loss

$$h_1 - h_2 = \frac{{V_2}^2}{2g} - \frac{P_1}{\rho} + h_L$$

h_1	113.5	ft	Height at point 1	(Elevation is assumed)
h_2	113	ft	Height at point 2	
h_1-h_2	0.5	ft		
D	24	in	Pipe inner diameter	
n	0.011		Mannings n	
g	32.2	ft/s^2	Gravitational acceleration	
L	50.4	ft	Length of pipe	

Step 1a. Pressure head $$h_p = \frac{P_1}{\rho}$$

h_p The pressure head h_p is equivalent to P_1/ρ, the height of the column of water with density ρ (62.4 lb/cf) required to give a pressure difference P_1.

h_{wc}	115.5	ft	Water surface elevation at max pool
h_1	113.5	ft	Height at point 1
h_p	2	ft	pressure head, h_{wc} - h_1

Step 2. Head Loss $$h_L = h_e + h_{ex} + h_{f_{1-2}} + h_b$$

h_e	Entrance head loss
h_{ex}	Exit head loss
h_f	Friction head loss
h_b	Losses due to bends

Step 2a. Entrance head loss $$h_e = K_e\left(\frac{{V_2}^2}{2g}\right)$$

K_e Entrance loss coefficient from page 17-14 "PE Civil Reference Manual"

K_e	0.78	unitless

Step 2b. Exit head loss

$$h_{ex} = K_{ex}\left(\frac{{V_2}^2}{2g}\right)$$

K_{ex}	When a fluid exits a pipe into a much larger body of water the velocity is reduced to 0 and all of the kinetic energy is dissipated; exit loss coefficient from page 17-14 "PE Civil Reference Manual"

K_{ex}	1.0

Step 2c. Friction head loss

$$h_f = f\left(\frac{L_1}{D_1}\right)\left(\frac{{V_2}^2}{2g}\right)$$

f	Friction factor for pipe
L	Length of pipe
D	Diameter of pipe

L_1/D_1	25.2

Step 2d. Losses due to bends

$$h_b = K_b\left(\frac{{V_2}^2}{2g}\right)$$

K_b	No bends in 24" RCP pipe configuration

K_{b1}	N/A	Loss coefficient for each bend
n	0	number of bends
K_b	0	Total loss coeffiecient

Step 2e. Velocity

$$h_1 - h_2 = \frac{{V_2}^2}{2g} - h_p + K_e\left(\frac{{V_2}^2}{2g}\right) + K_{ex}\left(\frac{{V_2}^2}{2g}\right) + f\left(\frac{L_1}{D_1}\right)\left(\frac{{V_2}^2}{2g}\right) + K_b\left(\frac{{V_2}^2}{2g}\right)$$

$$h_1 - h_2 + h_p = \left(\frac{{V_2}^2}{2g}\right)\left(1 + K_e + K_{ex} + f\left(\frac{L_1}{D_1}\right) + K_b\right)$$

$$V_2 = \sqrt{\frac{\left(h_1 - h_2 + h_p\right)(2g)}{\left(1 + K_e + K_{ex} + f\left(\frac{L_1}{D_1}\right) + K_b\right)}}$$

Step 3 Friction Factor

Velocity is dependent on friction factor. Moody's Diagram, figure 17.4 "PE Civil Reference Manual" is used to iterate through friction factor values to find velocity. To use Moody's Diagram, relative roughness (e/D) and Reynolds number must be used.

e	Roughness height for RCP from Appendix 17.A	
e	0.004	
e/D	0.002	

$$Re = \frac{DV}{\nu} \qquad \frac{Re}{V_2} = \frac{D}{\nu}$$

v	Kinematic viscosity (ft^2/s) found online in "The Engineering Toolbox"		
v	1.41E-05	ft^2/s	at 50 deg F
Re/V_2	141844	s/ft	

Step 4 Iteration

f	0.023		Assumed f value
V_2	6.9	ft/s	Velocity at the end of the pipe
Re	981928.9		
fRe	0.023		f from Moody diagram

f-fRe 0 Check for correct friction factor. If not equal to zero, redo step 4 with new f

V_2 6.9 ft/s Velocity at the end of the pipe

Step 5 Flow

$$Q_2 = V_2 A$$

A	3.1	ft^2	Cross sectional area of the pipe
Q	21.7	cfs	Flow through the pipe

Table 17.4 *Typical Loss Coefficients, K*[a]

device	K
angle valve	5
bend, close return	2.2
butterfly valve,[b] 2–8 in	$45f_t$
butterfly valve, 10–14 in	$35f_t$
butterfly valve, 16–24 in	$25f_t$
check valve, swing, fully open	2.3
corrugated bends	1.3–1.6 times value for smooth bend
standard 90° elbow	0.9
long radius 90° elbow	0.6
45° elbow	0.42
gate valve, fully open	0.19
gate valve, 1/4 closed	1.15
gate valve, 1/2 closed	5.6
gate valve, 3/4 closed	24
globe valve	10
meter disk or wobble	3.4–10
meter, rotary (star or cog-wheel piston)	10
meter, reciprocating piston	15
meter, turbine wheel (double flow)	5–7.5
tee, standard	1.8

[a]The actual loss coefficient will usually depend on the size of the valve. Average values are given.
[b]Loss coefficients for butterfly valves are calculated from the friction factors for the pipes with complete turbulent flow.

Loss coefficients for specific fittings and valves must be known in order to be used. They cannot be derived theoretically. However, the loss coefficients for certain changes in flow area can be calculated from the following equations.[16]

- *sudden enlargements* (D_1 is the smaller of the two diameters)

$$K = \left(1 - \left(\frac{D_1}{D_2}\right)^2\right)^2 \qquad 17.41$$

- *sudden contractions* (D_1 is the smaller of the two diameters)

$$K = \tfrac{1}{2}\left(1 - \left(\frac{D_1}{D_2}\right)^2\right) \qquad 17.42$$

- *pipe exit* (projecting exit, sharp-edged, or rounded)

$$K = 1.0 \qquad 17.43$$

- *pipe entrance*

reentrant: $K = 0.78$
sharp-edged: $K = 0.50$
rounded:

bend radius D	K
0.02	0.28
0.04	0.24
0.06	0.15
0.10	0.09
0.15	0.04

- *tapered diameter changes*

$$\beta = \frac{\text{small diameter}}{\text{large diameter}} = \frac{D_1}{D_2}$$

$$\phi = \text{wall-to-horizontal angle}$$

For enlargement, $\phi \leq 22°$:

$$K = 2.6\sin\phi(1 - \beta^2)^2 \qquad 17.44$$

For enlargement, $\phi > 22°$:

$$K = (1 - \beta^2)^2 \qquad 17.45$$

For contraction, $\phi \leq 22°$:

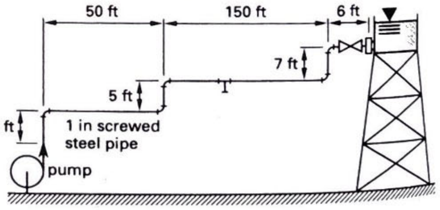
$$K = 0.8\sin\phi(1 - \beta^2) \qquad 17.46$$

For contraction, $\phi > 22°$:

$$K = 0.5\sqrt{\sin\phi}\,(1 - \beta^2) \qquad 17.47$$

Example 17.7

A pipeline contains one gate valve, five regular 90° elbows, one tee (flow through the run), and 228 ft of straight pipe. All fittings are 1 in screwed steel pipe. Disregard entrance and exit losses. Determine the total equivalent length of the piping system shown.



(not to scale)

Solution

From App. 17.D, the individual and total equivalent lengths are

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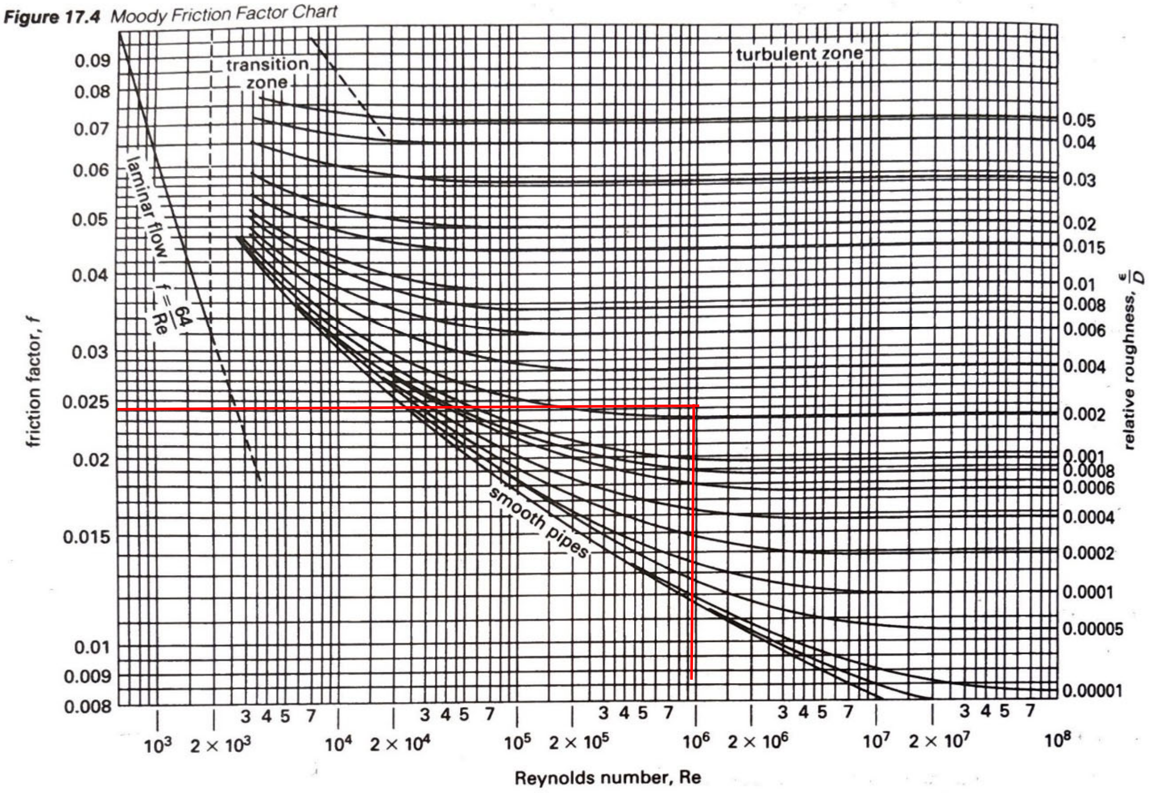
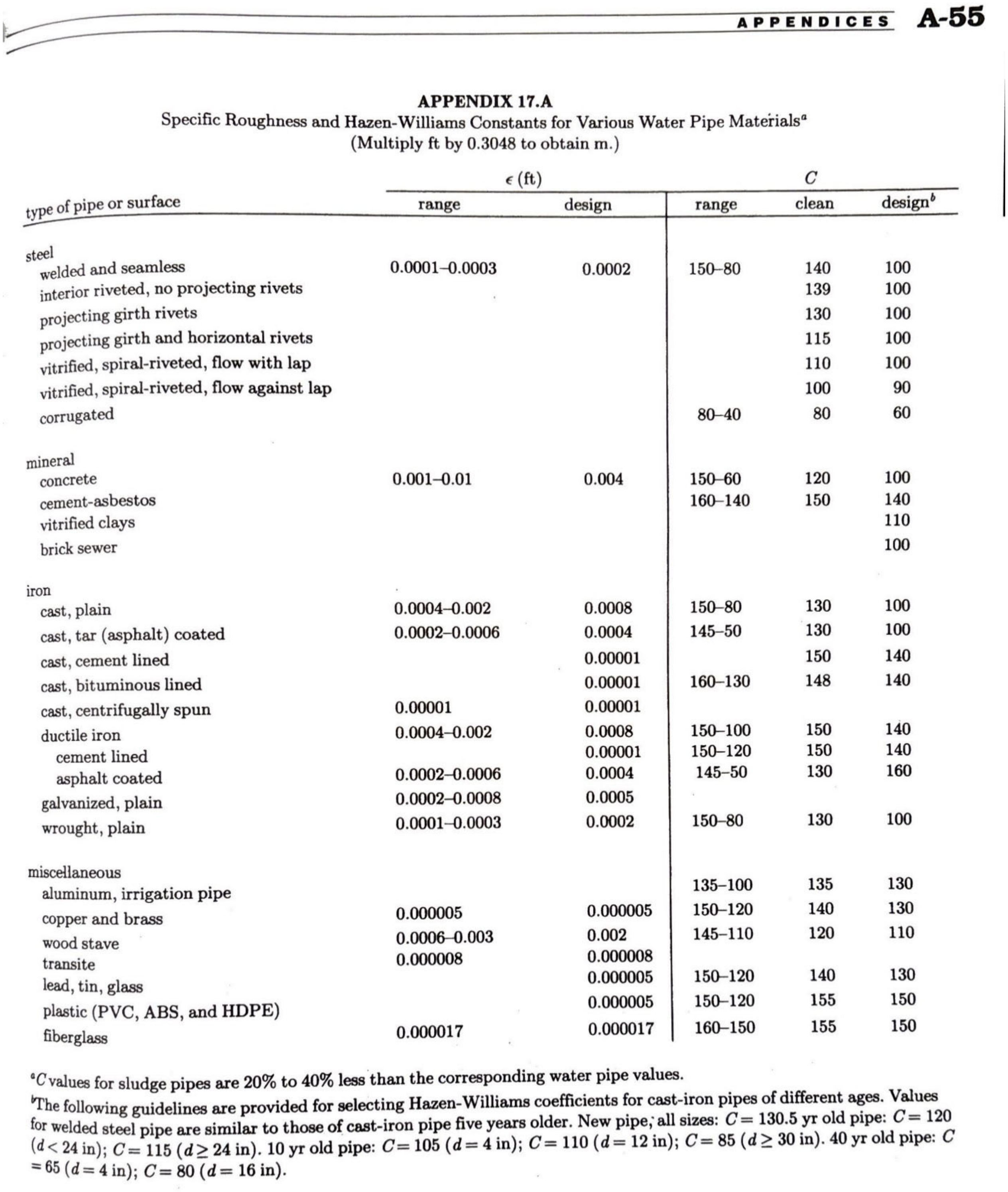
[16]No attempt is made to imply great accuracy with these equations. Correlation between actual and theoretical losses is fair.

Figure 17.4 *Moody Friction Factor Chart*





APPENDIX 17.A

Specific Roughness and Hazen-Williams Constants for Various Water Pipe Materials[a]
(Multiply ft by 0.3048 to obtain m.)

type of pipe or surface	ε (ft)		C		
	range	design	range	clean	design[b]
steel					
welded and seamless	0.0001–0.0003	0.0002	150–80	140	100
interior riveted, no projecting rivets				139	100
projecting girth rivets				130	100
projecting girth and horizontal rivets				115	100
vitrified, spiral-riveted, flow with lap				110	100
vitrified, spiral-riveted, flow against lap				100	90
corrugated			80–40	80	60
mineral					
concrete	0.001–0.01	0.004	150–60	120	100
cement-asbestos			160–140	150	140
vitrified clays					110
brick sewer					100
iron					
cast, plain	0.0004–0.002	0.0008	150–80	130	100
cast, tar (asphalt) coated	0.0002–0.0006	0.0004	145–50	130	100
cast, cement lined		0.00001		150	140
cast, bituminous lined		0.00001	160–130	148	140
cast, centrifugally spun	0.00001	0.00001			
ductile iron	0.0004–0.002	0.0008	150–100	150	140
cement lined		0.00001	150–120	150	140
asphalt coated	0.0002–0.0006	0.0004	145–50	130	160
galvanized, plain	0.0002–0.0008	0.0005			
wrought, plain	0.0001–0.0003	0.0002	150–80	130	100
miscellaneous					
aluminum, irrigation pipe			135–100	135	130
copper and brass	0.000005	0.000005	150–120	140	130
wood stave	0.0006–0.003	0.002	145–110	120	110
transite	0.000008	0.000008			
lead, tin, glass		0.000005	150–120	140	130
plastic (PVC, ABS, and HDPE)		0.000005	150–120	155	150
fiberglass	0.000017	0.000017	160–150	155	150

[a] C values for sludge pipes are 20% to 40% less than the corresponding water pipe values.

[b] The following guidelines are provided for selecting Hazen-Williams coefficients for cast-iron pipes of different ages. Values for welded steel pipe are similar to those of cast-iron pipe five years older. New pipe, all sizes: $C = 130$.5 yr old pipe: $C = 120$ ($d < 24$ in); $C = 115$ ($d \geq 24$ in). 10 yr old pipe: $C = 105$ ($d = 4$ in); $C = 110$ ($d = 12$ in); $C = 85$ ($d \geq 30$ in). 40 yr old pipe: $C = 65$ ($d = 4$ in); $C = 80$ ($d = 16$ in).

Attachment D

Drawdown Depth Calculations

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CLIENT:	Town of Concord	**Prepared By:**	EJT	**Date:**	10/2/24
PROJECT:	Warner's Pond Dam Removal Preliminary Design	**Reviewed By:**	AEH	**Date:**	10/24/24
NUMBER:	64040-01-00-LS				
SUBJECT:	Impoundment Drawdown Potential				

Purpose: Determine the height of water above the sluiceway for various flows. This was used to determine the potential drawdown depth of the Warner's Pond impoundment.

Data Sources

A	PE Civil Reference Manual, 16th Edition, Michael R. Lindenburg
B	S-1 Spillway Details.pdf
C	Warners Pond StreamStats.pdf
D	Warner's Pond - Pipe Flow Capacity Calulations.xlsx

Broad-Crested Weir Equation

Equation	Source	Assumptions
$Q = C_s b H^{3/2}$	A	Assumes velocity of approach is insignificant

$$H = \left(\frac{Q}{C_s b}\right)^{2/3}$$

Value	Unit	Variable	Source	Description
3.33	$ft^{1/2}/s$	C_S	A	Spillway coefficient
5	ft	b	B	Width of weir
121	cfs	Q_{WAvg}	C	Average winter flow
21	cfs	Q_{pipe}	D	24" RCP pipe capacity

Determine flow over weir during average winter flow

$$Q_{Weir\ WAvg} = Q_{W\ Avg} - Q_{pipe}$$

100 cfs

Determine impoundment elevation with drawdown

Value	Unit	Variable	Source	Description
115	ft	Z_{weir}	B	Elevation of the bottom of the weir with stoplogs removed
118.77	ft		B	Elevation of the spillway

$$Z_{imp} = Z_{Weir} + H$$

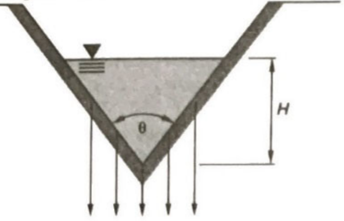
Total flow, Q	Flow over weir, Q_{weir}	Height, H		Impoundment elevation, Z_{imp}	Depth below normal pool elevation	
cfs	cfs	ft	in	ft	ft	
121	100	3.3	39.7	118.3	0.5	$Q_{W\ Avg}$
111	90	3.1	37.0	118.1	0.7	
101	80	2.8	34.2	117.8	0.9	
91	70	2.6	31.3	117.6	1.2	
81	60	2.4	28.2	117.4	1.4	
71	50	2.1	25.0	117.1	1.7	
61	40	1.8	21.5	116.8	2.0	
51	30	1.5	17.8	116.5	2.3	
41	20	1.1	13.6	116.1	2.6	
31	10	0.7	8.5	115.7	3.1	

Broad-Crested Weir check

Value	Unit	Variable	Source	Description
1.5	ft	T	B	Weir thickness

1/2 H ft	T > 1/2 H?
1.66	No
1.54	No
1.42	Yes
1.30	Yes
1.18	Yes
1.04	Yes
0.90	Yes
0.74	Yes
0.56	Yes
0.36	Yes

Figure 19.8 *Triangular Weir*



16. TRAPEZOIDAL WEIRS

A *trapezoidal weir* is essentially a rectangular weir with a triangular weir on either side. (See Fig. 19.9.) If the angle of the sides from the vertical is approximately 14° (i.e., 4 vertical and 1 horizontal), the weir is known as a *Cipoletti weir*. The discharge from the triangular ends of a Cipoletti weir approximately make up for the contractions that would reduce the flow over a rectangular weir. Therefore, no correction is theoretically necessary. This is not completely accurate, and for this reason, Cipoletti weirs are not used where great accuracy is required. The discharge is

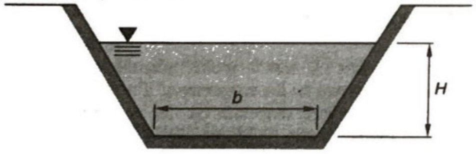
$$Q = \tfrac{2}{3} C_d b \sqrt{2g} H^{3/2} \qquad 19.57$$

The average value of the discharge coefficient is 0.63. The discharge from a Cipoletti weir is found by using Eq. 19.58.

$$Q = 1.86 bH^{3/2} \quad \text{[SI]} \qquad 19.58(a)$$

$$Q = 3.367 bH^{3/2} \quad \text{[U.S.]} \qquad 19.58(b)$$

Figure 19.9 *Trapezoidal Weir*



17. BROAD-CRESTED WEIRS AND SPILLWAYS

Most weirs used for flow measurement are sharp-crested. However, the flow over spillways, broad-crested weirs, and similar features can be calculated using Eq. 19.49 even though flow measurement is not the primary function of the feature. (A weir is broad-crested if the weir thickness is greater than half of the head, H.)

A *dam's spillway (overflow spillway)* is designed for a capacity based on the dam's inflow hydrograph, turbine capacity, and storage capacity. Spillways frequently have a cross section known as an *ogee*, which closely approximates the underside of a nappe from a sharp-crested weir. This cross section minimizes the cavitation that is likely to occur if the water surface breaks contact with the spillway due to upstream heads that are higher than designed for.[8]

Discharge from an overflow spillway is derived in the same manner as for a weir. Equation 19.59 can be used for broad-crested weirs ($C_1 = 0.5$–0.57) and ogee spillways ($C_1 = 0.60$–0.75).

$$Q = \tfrac{2}{3} C_1 b \sqrt{2g} H^{3/2} \qquad 19.59$$

The *Horton equation* for broad-crested weirs combines all of the coefficients into a spillway (weir) coefficient and adds the velocity of approach to the upstream head. (See Eq. 19.60.) The *Horton coefficient*, C_s, is specific to the Horton equation. (C_s and C_1 differ by a factor of about 5 and cannot easily be mistaken for each other.)

$$Q = C_s b \left(H + \frac{\mathrm{v}^2}{2g} \right)^{3/2} \qquad 19.60$$

If the velocity of approach is insignificant, the discharge is found using Eq. 19.61.

$$Q = C_s b H^{3/2} \qquad 19.61$$

C_s is a *spillway coefficient*, which varies from about 3.3 $\text{ft}^{1/2}/\text{sec}$ to 3.98 $\text{ft}^{1/2}/\text{sec}$ (1.8 $\text{m}^{1/2}/\text{s}$ to 2.2 $\text{m}^{1/2}/\text{s}$) for ogee spillways. 3.97 $\text{ft}^{1/2}/\text{sec}$ (2.2 $\text{m}^{1/2}/\text{s}$) is frequently used for first approximations. For broad-crested weirs, C_s varies between 2.63 $\text{ft}^{1/2}/\text{sec}$ and 3.33 $\text{ft}^{1/2}/\text{sec}$ (1.45 $\text{m}^{1/2}/\text{s}$ and 1.84 $\text{m}^{1/2}/\text{s}$). (Use 3.33 $\text{ft}^{1/2}/\text{sec}$ (1.84 $\text{m}^{1/2}/\text{sec}$) for initial estimates.) C_s increases as the upstream design head above the spillway top, H, increases, and the larger values apply to the higher heads.

Broad-crested weirs and spillways can be calibrated to obtain greater accuracy in predicting flow rates.

Scour protection is usually needed at the toe of a spillway to protect the area exposed to a hydraulic jump. This protection usually takes the form of an extended horizontal or sloping apron. Other measures, however, are needed if the tailwater exhibits large variations in depth.

---

[8] Cavitation and separation will not normally occur as long as the actual head, H, is less than twice the design value. The shape of the ogee spillway will be a function of the design head.