HYDROLOGY REPORT

Bar Mills Bridge (#3333) over the Saco River and Canal Bridge (#1525) over a Saco River canal carry Maine state route 4A between the towns of Hollis and Buxton, Maine. The Saco River basin begins in Carroll, New Hampshire at Saco Lake and flows southeast for approximately 136 miles; The river outlets into the Atlantic Ocean at Saco Bay between the towns of Saco and Biddeford, Maine. Bar Mills and Canal Bridges are located approximately 18.8 miles upstream of the mouth of the river. The river is approximately 260 feet wide at the Bar Mills Bridge and the canal is approximately 105 feet wide at Canal Bridge.

The Saco River typically floods in the spring due to the combination of heavy rainfall and snowmelt. The flood of record at the route 4A crossing was the March 1936 flood. The peak discharge during this event was 58,269 cubic feet per second at a gage station in close proximity upstream. This event exceeded the 500 year recurrence interval and washed out the bridge at this location. According to the plans for the replacement bridge, which is the existing bridge, the peak water elevation was 152.4 (NAVD 88) during the 1936 event and the previous recorded high water elevation at this location was elevation 145.3 (NAVD 88) during the March 1896 flood.

USGS WRI Report 99-4008 (Hodgkins, 1999) was used to develop the hydrologic data used for this project. The USGS gage station at West Buxton (01067000) is located approx. 5 miles upstream from the bridge site. This gage is just upstream of the West Buxton dam which was operated by the USGS from 1908 until 1948. The gage values for this location included in Hodgkins (1999) are based on gage records and additional data from the dam that enabled the period of record to be extended until 1996. The drainage area of the bridge is only about 1.4% larger than that reported for the West Buxton gage. A second gage is located at Cornish (01066000), but is further away (approx. 25 miles upstream) and has a drainage area that is about 23% less than the bridge. Where the West Buxton gage is considerably closer to the bridge site, has nearly the same drainage area, and is also located in proximity to a dam, it was used as the basis for determining peak flow estimates for the bridge. Plus or minus 3 percent of the gage station drainage area is considered the limit of the weighted average technique; therefore, the weighted peak flow estimates for the West Buxton gage taken from Table 1 of Hodgkins (1999) are the recommended peak flows for the bridge site. However, Table 1 of Hodgkins (1999) does not include $Q_{1.1}$ so this value was determined by projecting $Q_{1.1}$ on a log-normal probability plot based on the other data points.

HYDROLOGY REPORT

Drainage Area	mi^2	1,594
$Q_{1.1}$	ft^3 / s	8,230
Q_{10}	ft^3 / s	23,340
Q_{25}	ft^3 / s	28,389
Q_{50}	ft^3 / s	32,344
Q_{100}	ft^3 / s	36,369
Q_{500}	ft^3 / s	46,609

Figure 1: Hydrology Summary

Reported By: Garrett Gustafson
Date: September 20, 2013

Methodology

This site presents a number of hydraulic modeling challenges including splitting flow, four interconnected weirs (two upstream spillway weirs that feed the bypass reach under Bar Mills Bridge and two weirs downstream of Canal Bridge immediately before the power facility) and a primarily regulated headwater elevation. Additionally, application of historic data is complicated by the reconstruction of the dam facility in the 1950's (See Figure 2 below) and the presence of ice during the flood of record in 1936. Conventional practice of development of a comprehensive model in Hydrologic Engineering Centers River Analysis System (HEC-RAS) version 4.0, was pursued, but proved to be a futile effort requiring too many "educated guesses" to warrant such a rigorous model. The complexity of the site and unavailability of reliable data for benchmarking made development of an all-inclusive model both exceedingly convoluted and unreliable. A number of approximate methods along with historic information sources were composited in lieu of a HEC-RAS model of the entire site to develop an understanding of the hydraulics at this location.

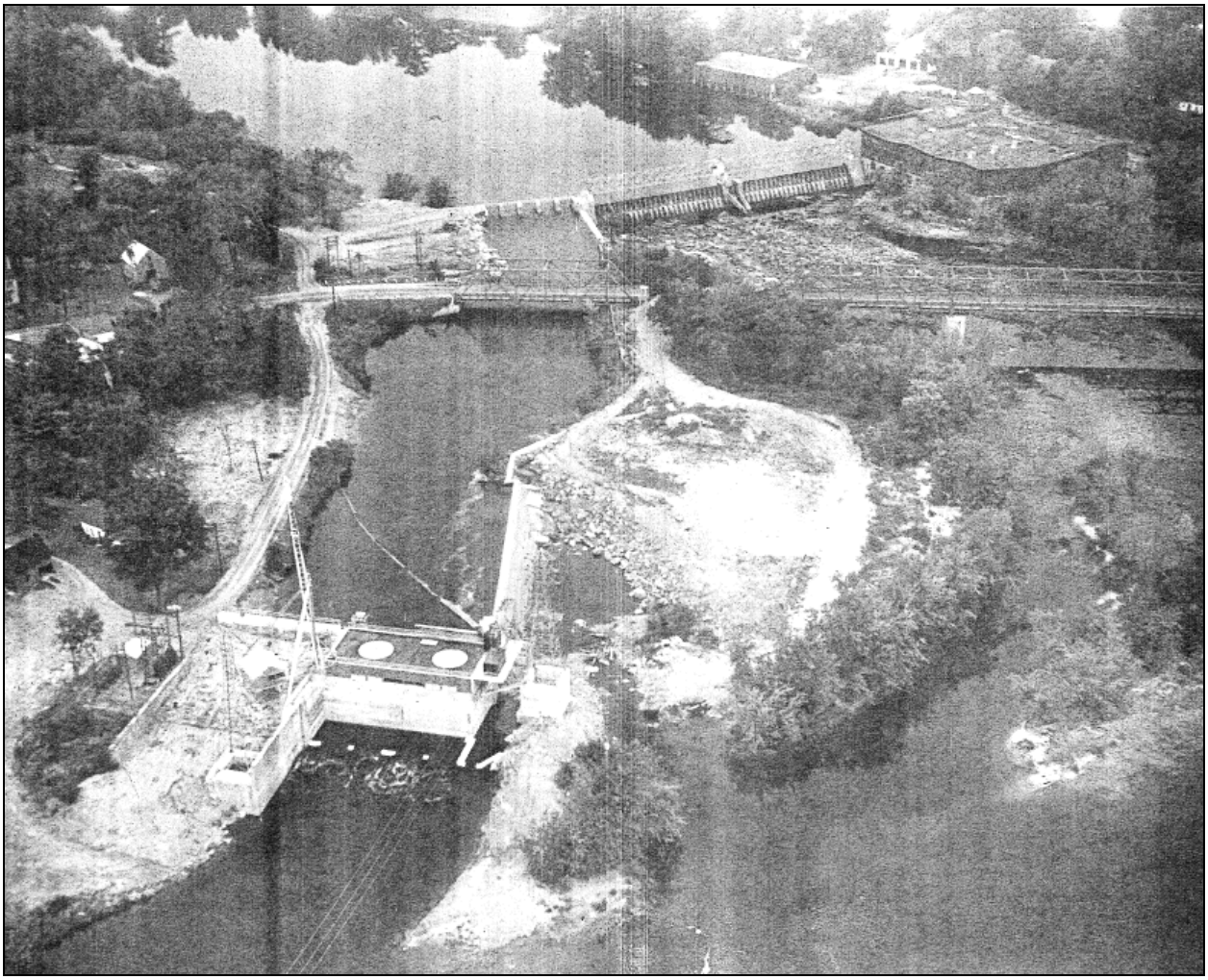


Figure 2: Aerial of Site Following Major Reconstruction

Reference historical information was significant in the analysis of this site. One of the best sources of information available was the Hollis Flood Insurance Study from 1982 (FIS). Figure 3 contains data from the stream profile presented in the FIS report;

this data is also shown graphically in Figures 4 and 6 for Canal and Bar Mills Bridges respectively.

	Flow (cfs)	Bar Mills (ft)	Canal (ft)
Headwater elevation @ Q_{10}	25,600	138.5	149.1
Headwater elevation @ Q_{50}	38,200	141.7	150.8
Headwater elevation @ Q_{100}	44,200	143.0	151.7
Headwater elevation @ Q_{500}	61,500	147.0	154.2

Figure 3: Hollis FIS Stream Profile Data

Vertical datum adjustments were conducted with a datum shift calculated with the VERTCON program on the noaa website, http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl. The datum shift at this site is -0.68 feet from NGVD 29 to NAVD 88.

Canal Bridge

The hydraulics at the Canal Bridge location are highly regulated and required review of data from a number of sources as outlined below.

A Central Maine Power (CMP) two sheet plan set dated September 9, 1955 and titled “Bar Mills Redevelopment” was obtained from Roland Cote, a former CMP engineer. These sheets contain a number of useful pieces of information including weir geometries and a hydraulic rating curve for the facility. The datum for the plan set is not labeled, however, as the sheets are from 1955 it is assumed to be NGVD 29. This assumption is circumstantiated in the following paragraph.

The design team was able to access five years of daily headwater elevations and corresponding flow data from Brookfield, the owner and operator of the dam. At an August 12, 2013 meeting with Brookfield, they confirmed that the flow data is for the entire river, Bar Mills plus Canal. Additionally, the flow data is calculated using a long standing formula incorporating headwater elevation, power production and number of flash boards down. Brookfield also verified that their elevations are “referenced to sea level” and have not changed over the life of the dam. Based on the information above, the elevations referenced in the hydraulic data from Brookfield and the CMP plan sheets are assumed to be referenced to the NGVD 29 datum.

A spreadsheet was developed specifically for this site as a simplistic approach to a hydraulic model which utilized available information and required little approximation outside of the available data. This spreadsheet mathematically modeled the four unique weir geometries, per the CMP plan set, that make up the site and solved the corresponding weir equations simultaneously to resolve headwater elevation. Conducting hydraulic calculations by hand helped establish an understanding of the behavior of the site and provided reference data for comparison with the other data sets. This spreadsheet used sharp crested weir equations since the width of the weirs at the top is relatively small; this assumption also cut the numbers of variables to be solved for in half. By assuming the headwater elevations are identical at the four weirs,

a system of four equations with a shared unknown, headwater elevation, were developed. This four equation system was solved with a routine and the excel add-on "Goal Seek" to calculate headwater elevations at the recurrence interval flows. A summary table at half foot headwater elevation intervals was also developed and is available for reference with a sample output of the spreadsheet in the appendices of this Preliminary Design Report.

The Brookfield data, the CMP rating curve, the FIS stream profile data and the output of the spreadsheet were overlaid and compared for the purpose of compositing all the data and is shown in Figure 4. The CMP rating curve uses the NGVD 29 datum so the other sets of data were adjusted as necessary to make the whole graph NGVD 29. The final reported elevations were adjusted by the datum shift to convert to NAVD 88.

Figure 4 was developed by initially graphing the Brookfield data and the spreadsheet output. The FIS data was added by taking points off the stream profile by hand. Points for Bar Mills Bridge were taken at the bridge location indicated on the profile and the water surface elevation at Canal Bridge was assumed to be close to the water surface elevation just upstream of the weir shown in the profile. While the FIS stream profile had precise enough grid lines to extract points to reproduce in Figure 4, the grid lines on the CMP rating curve were not at a small enough interval to accurately reconstruct the curve with hand methods; therefore, the image of this graph was manipulated to remove the grid lines and other superfluous data leaving only reference marks at the intersections of Q = 0 and 70,000 cfs and headwater elevations of 142.0 and 154.0 NGVD 29. These reference marks were aligned with the extents of Figure 4 to complete the graph.

The graph shows good correlation between the CMP rating curve, the FIS data and the spreadsheet output. However, all three of these data sources are based on theory and do not account for the regulation of the headwater elevation by Brookfield. To account for the water surface regulation, the final recommendation is to use the maximum water surface elevation from the five years of Brookfield data as a minimum elevation for each recurrence interval. The final recommended water surface elevations are the maximum of the CMP rating curve, the FIS data and the spreadsheet output data at each of the recurrence intervals outlined in the Hydrology Report, Figure 1, with a minimum of 150.52 NAVD 88. Water surface elevations for each recurrence interval are shown in the summary table at the end of this report. The proposed wall and bank geometries match the existing geometries and therefore the proposed water surface elevations are identical to the existing.

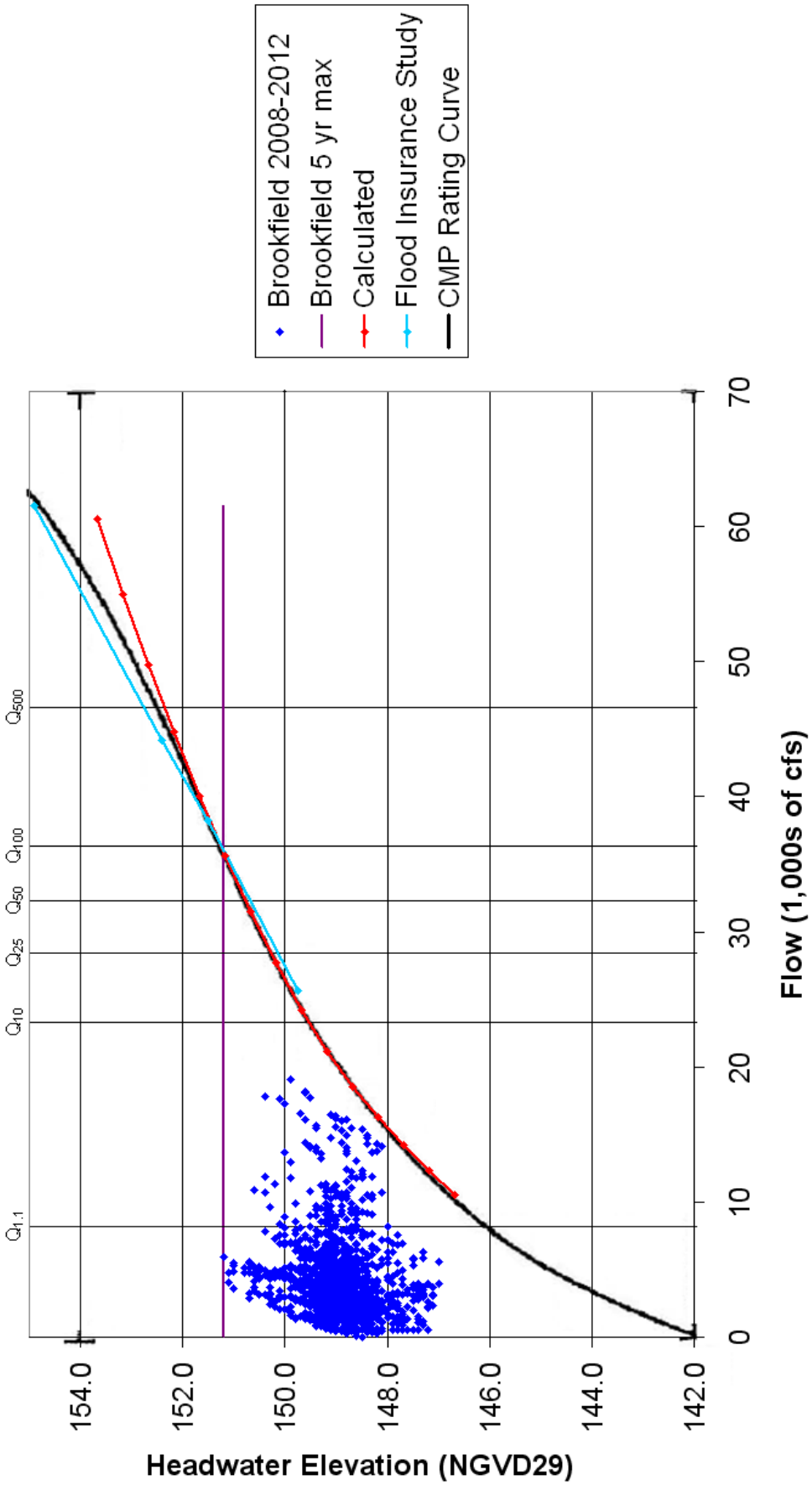


Figure 4: Canal Bridge Composite Rating Curve (NGVD 29)

Bar Mills Bridge

The previously referenced weir calculation spreadsheet was also used to calculate the portion of flow in the Bar Mills reach. This calculation was done with all the flashboards assumed to be down to ensure a maximum flow for each recurrence interval. The flow down the Bar Mills reach, under Bar Mills Bridge, bypassing the power facility, is shown in Figure 5.

$Q_{1.1}$	ft^3 / s	8,230
Q_{10}	ft^3 / s	22,729
Q_{25}	ft^3 / s	26,764
Q_{50}	ft^3 / s	29,779
Q_{100}	ft^3 / s	32,775
Q_{500}	ft^3 / s	40,200

Figure 5: Bar Mills Reach Flow

The above calculation of the flow split enabled modeling the Bar Mills reach in HEC-RAS. This model represents a 390 foot long segment of the Saco River, including the Bar Mills Bridge carrying route 4A. Limited survey allowed for one up and downstream cross section each and up and downstream bridge cross sections. These cross sections were based on digital topography created by the Department's survey team.

A number of assumptions were necessary for the development of the Bar Mills model. First, the model assumes steady flow. This is considered reasonable because this location is not tidal and the river is highly regulated with a number of hydropower facilities up and downstream. Instantaneous increases in flow rate are likely as flashboards are lowered to regulate the pond headwater elevation. However, Brookfield has indicated that they never lower more than one flashboard at a time and the increase in flow from lowering one flashboard would constitute 1% or less of the total flow at recurrence intervals Q_{10} and greater and are therefore considered insignificant. Secondly, this model assumes a Manning's n value of 0.055 in the channel and 0.11 in the overbank areas. These values were selected because they are representative of the conditions at the site: 0.055 is described as rocks and angular boulders and 0.11 is described as thin wooded areas with little brush. The FIS reported a range of Manning's n values for the Saco River of 0.03-0.055 for the channel and 0.04-0.11 for the overbank areas. The selected values are the upper bounds of these ranges and since water surface elevation rises with increasing Manning's n, these assumptions are considered sufficiently conservative with respect to water surface elevation.

Due to a lack of available reliable data at the bridge location, normal depth was used as the up and downstream boundary conditions. The final slope used in the HEC-RAS model was 0.0008 which was the smallest slope between any two model cross sections. The minimum slope within the extents of the available survey was used since water surface elevation is maximized by minimization of the channel slope boundary conditions and a representative slope of the river was not available.

The existing geometry was modeled in HEC-RAS; due to the limited encroachment of the proposed structures, the proposed geometry was not modeled. It was decided that the change in contraction dynamics due to the proposed abutment locations was unnecessary to model. The percent change in flow area at all of the proposed substructure location cross sections is small due to the 48 degree skew of the proposed alignment in conjunction with the width of the waterway. Refining the model for such differences would be misleading as the accuracy is intrinsically imperfect due to the lack of sufficient information to model contraction and expansion or benchmark the model properly. It should be acknowledged that this HEC-RAS model is rough with inherent imperfections but represents a reasonable approximation of the water surface elevations of both the existing and proposed conditions under Bar Mills Bridge.

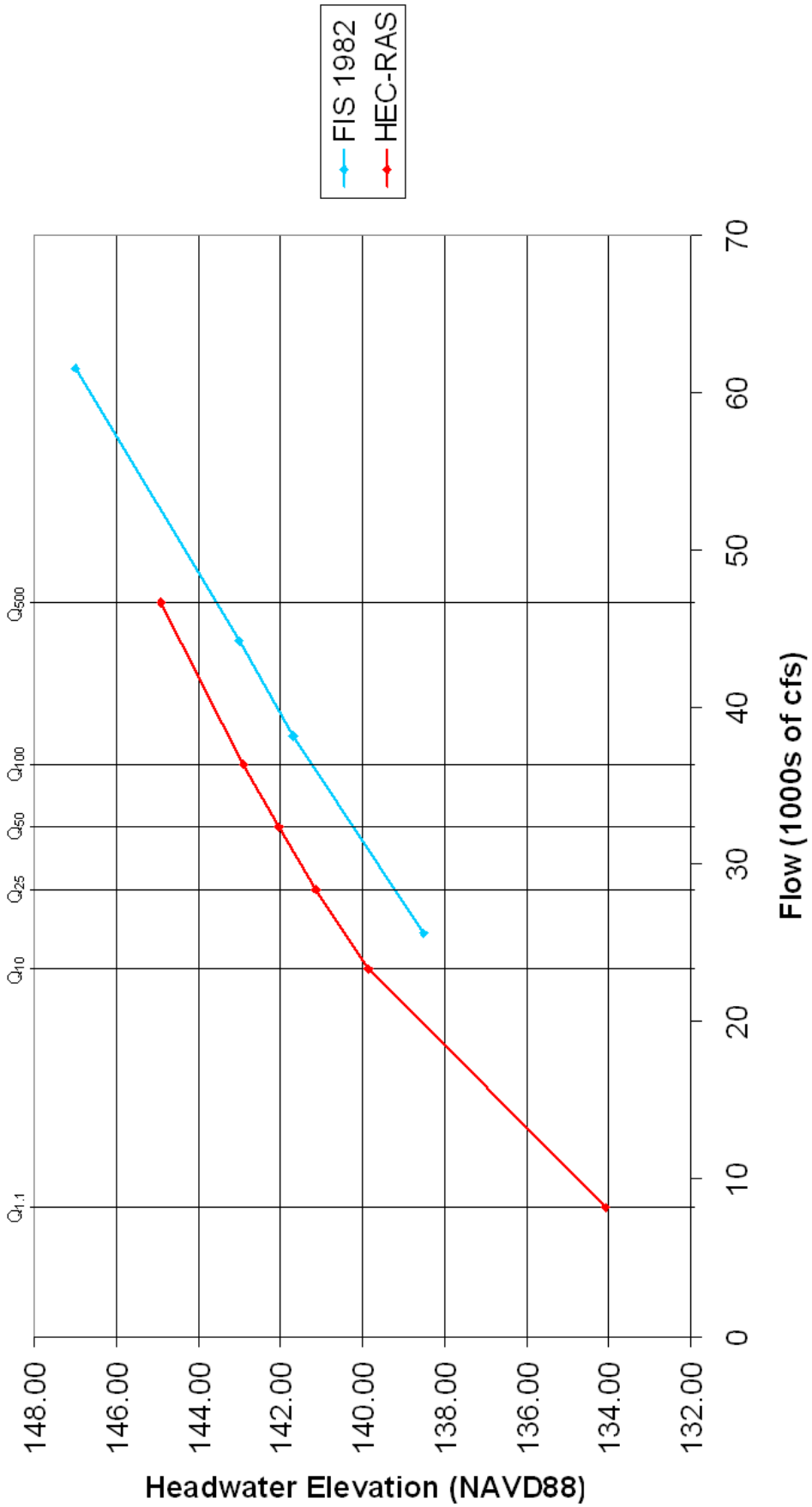


Figure 6: Bar Mills Bridge Composite Rating Curve (NAVD 88)

This report assumes free-flow conditions on the Saco. Ice jam effects were not considered or incorporated into any models or calculations. The 1936 flood elevation at the existing bridge location is the recommended worst case water surface elevation for the proposed Bar Mills Bridge under an ice jam condition, as reported below. This event occurred prior to the 1950's reconstruction of the hydropower facility and spillway weirs and the effect of this reconstruction on water surface elevations during high flow events is uncertain; however, the elevation reported is the best available data.

Summary

		Bar Mills	**Canal**
Total Area of Waterway Opening	ft^2	10485	1978
Headwater elevation @ $Q_{1.1}$	ft	134.08	150.52
Headwater elevation @ Q_{10}	ft	139.87	150.52
Headwater elevation @ Q_{25}	ft	141.14	150.52
Headwater elevation @ Q_{50}	ft	142.05	150.52
Headwater elevation @ Q_{100}	ft	142.91	150.58
Headwater elevation @ Q_{500}	ft	144.92	152.07
Freeboard @ Q_{50}	ft	9.90	3.42
Freeboard @ Q_{100}	ft	9.04	3.36
Flood Of Record (May 1936) Elevation 152.4 ft			
Outlet Velocity @ $Q_{1.1}$	ft/s	3.63	N/A
Outlet Velocity @ Q_{10}	ft/s	4.57	N/A
Outlet Velocity @ Q_{25}	ft/s	4.79	N/A
Outlet Velocity @ Q_{50}	ft/s	4.94	N/A
Outlet Velocity @ Q_{100}	ft/s	5.09	N/A
Outlet Velocity @ Q_{500}	ft/s	5.41	N/A

Reported By: Garrett Gustafson
Date: September 20, 2013

Note: All elevations based on North American Vertical Datum (NAVD) of 1988 unless otherwise noted.

Scour

All four foundation elements for Bar Mills Bridge will be founded on bedrock and therefore a scour analysis for this bridge was not conducted. The western abutment for Canal Bridge will also be founded on bedrock; however, the foundation type for the eastern abutment is yet to be determined and is pending boring results. Should a pile supported foundation alternative be selected, a scour analysis will be completed during final design. It should be noted that this report aimed to find maximum water surface elevations and due to the inverse relation between water surface elevation and water velocity, modeling assumptions should be revisited and all water velocities recalculated. The velocities reported in the summary table represent approximate minimum velocities.

Reported By: Garrett Gustafson
Date: September 20, 2013

Note: Supporting documentation is provided in the appendix of this Preliminary Design Report.

Table 1. Estimated peak flows and maximum recorded flows for selected U.S. Geological Survey streamflow-gaging stations—Continued

[m^3/s, cubic meters per second; km^2, square kilometers; u, unregulated; r, regulated; nr, near]

USGS gaging station number	Gaging station name	Gaging station (G), Regression (R), Weighted (W)[a]	Peak flow (m^3/s) for given recurrence interval							Highest peak flow known			Regulation[c]	Drainage area (km^2)
			2 years	5 years	10 years	25 years	50 years	100 years	500 years	Date	Flow (m^3/s)	Period of known peak flows[b]		
01060000	Royal River at Yarmouth, Maine	G	107	158	194	242	280	318	414	3-13-1977	326	1949-96	u	365
		R	88.7	131	160	199	229	259	335					
		W	106	157	192	238	275	312	404					
01062700	Patte Brook nr Bethel, Maine	G	6.35	11.4	15.6	21.7	26.8	32.5	48.1	7-1-1973	18.8	1964-74	u	14.6
		R	7.31	12.0	15.5	20.5	24.5	28.8	39.7					
		W	6.50	11.5	15.6	21.4	26.1	31.2	44.8					
01064000	Presumpscot River at Outlet of Sebago Lake, Maine	G	36.2[h]	59.2[h]	78.9[h]	110[h]	138[h]	172[h]	273[h]	4-7-1902	198[i]	1886-1996	r	1,140
01064118	Presumpscot River at Westbrook, Maine	G	150	222	280	368	446	534	792	10-22-1996	660	1895-1996	r	1,500
01064200	Mill Brook nr Old Orchard Beach, Maine	G	2.57	4.00	5.03	6.41	7.48	8.60	11.4	4-2-1973	5.89	1964-74	u	5.57
		R	3.65	6.21	8.21	11.1	13.4	15.9	22.5					
		W	2.74	4.44	5.80	7.77	9.39	11.2	15.7					
01064500	Saco River nr Conway, New Hampshire	G	462	723	913	1,170	1,370	1,580	2,110	3-27-1953	1,340	1903-09, 1929-96	u	997
		R	329	490	603	754	870	989	1,280					
		W	459	715	900	1,150	1,340	1,540	2,040					
01065000	Ossipee River at Effingham Falls, New Hampshire	G	100	135	160	192	216	242	304	3-28-1953	331	1937-96	u	855
		R	175	251	303	370	422	474	602					
		W	103	141	169	206	235	265	339					
01065500	Ossipee River at Cornish, Maine	G	127	180	218	268	306	347	446	3-21-1936	487	1916-96	u	1,170
		R	235	335	403	492	559	627	793					
		W	129	185	225	279	320	365	472					
01066000	Saco River at Cornish, Maine	G	378	519	610	721	801	880	1,060	3-21-1936	1,320	1786-1996	u	3,350
		R	618	860	1,020	1,230	1,390	1,550	1,930					
		W	383	529	626	746	834	922	1,120					
01066100	Pease Brook nr Cornish, Maine	G	4.51	7.49	9.83	13.2	16.0	19.1	27.4	4-23-1969	13.8	1964-74, 1996	u	12.4
		R	6.52	10.8	14.0	18.6	22.2	26.2	36.3					
		W	4.82	8.14	10.8	14.8	18.0	21.6	30.9					

Table 1. Estimated peak flows and maximum recorded flows for selected U.S. Geological Survey streamflow-gaging stations—Continued

[m^3/s, cubic meters per second; km^2, square kilometers; u, unregulated; r, regulated; nr, near]

USGS gaging station number	Gaging station name	Gaging station (G), Regression (R), Weighted (W)[a]	Peak flow (m^3/s) for given recurrence interval							Highest peak flow known			Regulation[c]	Drainage area (km^2)
			2 years	5 years	10 years	25 years	50 years	100 years	500 years	Date	Flow (m^3/s)	Period of known peak flows[b]		
01066500	Little Ossipee River nr South Limington, Maine	G	57.5	88.9	112	145	172	200	275	3-19-1936	242[d]	1936-96	u	435
		R	87.8	127	154	189	217	244	312					
		W	58.7	91.0	115	149	176	205	280					
01067000	Saco River at West Buxton, Maine	G	398	529	621	744	840	940	1,190	3-22-1936	1,650	1786-1996	u	4,070
		R	694	957	1,130	1,360	1,520	1,690	2,090					
		W	412	555	661	804	916	1,030	1,320					
01069500	Mousam River nr West Kennebunk, Maine	G	39.8	57.3	70.0	87.5	102	116	155	3-20-1983	114	1939-84, 1996	r	256
01069700	Branch Brook nr Kennebunk, Maine	G	7.18	12.6	16.9	23.2	28.4	34.2	50.0	10-22-1996	28.9	1964-74, 1996	u	26.7
		R	8.71	13.6	17.3	22.3	26.2	30.3	40.8					
		W	7.42	12.8	17.0	22.9	27.7	32.9	46.4					
01072100	Salmon Falls River at Milton, New Hampshire	G	37.4	57.0	71.6	92.0	109	126	172	4-6-1984	113	1955-96	r	280
01072500	Salmon Falls River nr South Lebanon, Maine	G	48.8	73.3	93.3	124	150	180	269	3-19-1936	155	1929-76	r	363

[a]Gaging station (G) refers to gaging-station peak flow. Regression (R) refers to regression-equation peak flow. Weighted (W) refers to weighted-average peak flow (the weighted average of the gaging-station peak flow and the regression-equation peak flow).

[b]Period of known peak flows includes relevant historical information (information outside of the period of systematic data collection at or near a streamflow-gaging station).

[c]Regulated (r) indicates that the drainage basin upstream of a streamflow-gaging station has more than 49,200 cubic meters of usable storage per square kilometer (Benson, 1962). Usable storage is the volume of water normally available for release from a reservoir, between the minimum and maximum controllable elevations. Unregulated (u) indicates that the drainage basin upstream of a gaging station has less than 49,200 cubic meters of usable storage per square kilometer.

[d]Peak flow is an estimate.

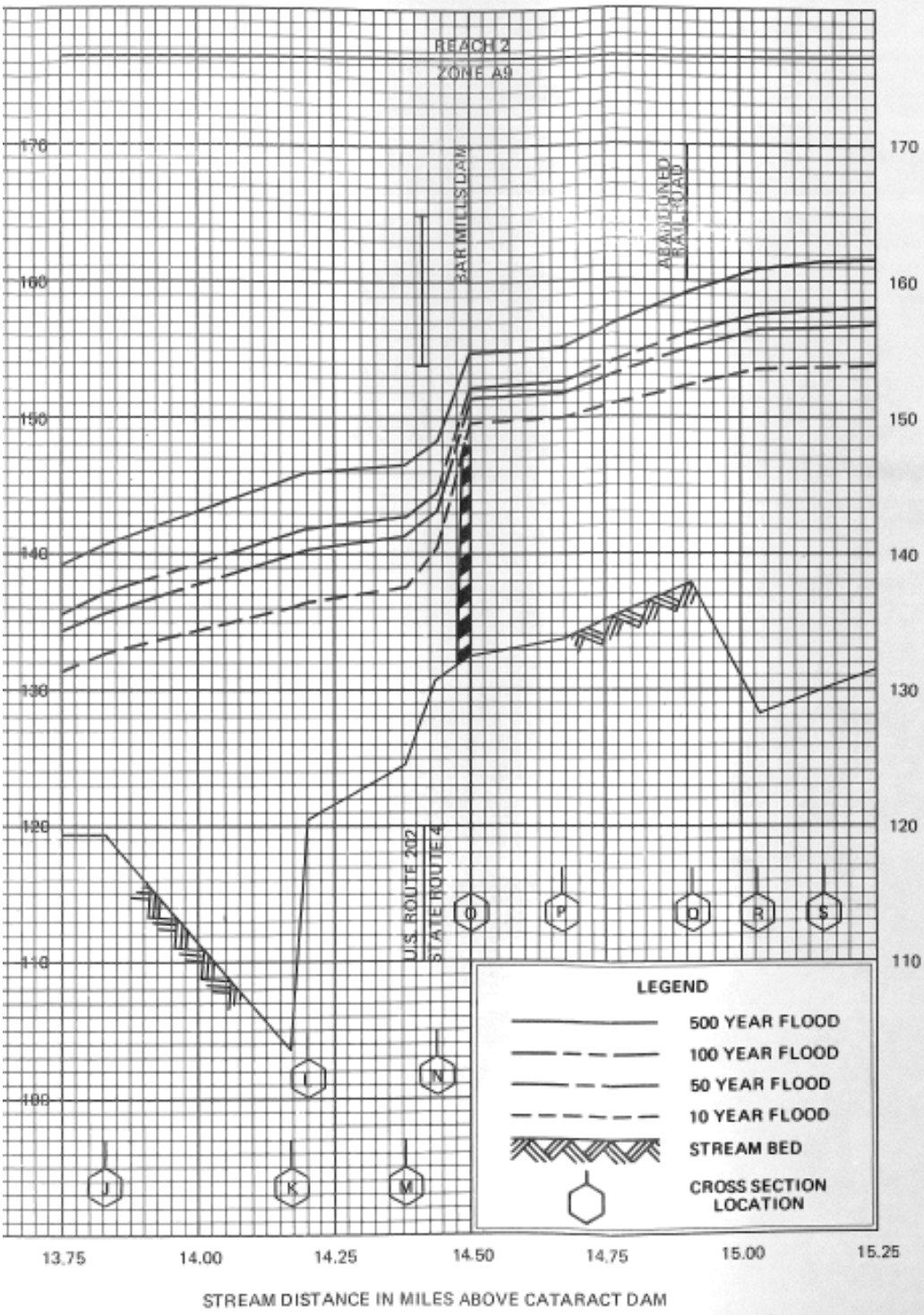
[e]Peak flow was affected by a dam break. The peak flow, removing the effects of the dam break, is estimated to be 326 m^3/s.

[f]Day of occurrence is not exact.

[g]Gaging station formerly published under the name Cold Brook near Northern Maine Junction, Maine.

[h]Peak flows for given recurrence intervals at this site were computed using daily-mean peak flows rather than (instantaneous) peak flows. Peak flows are equal to or greater than daily-mean peak flows.

[i]Peak flow is a daily-mean peak flow.



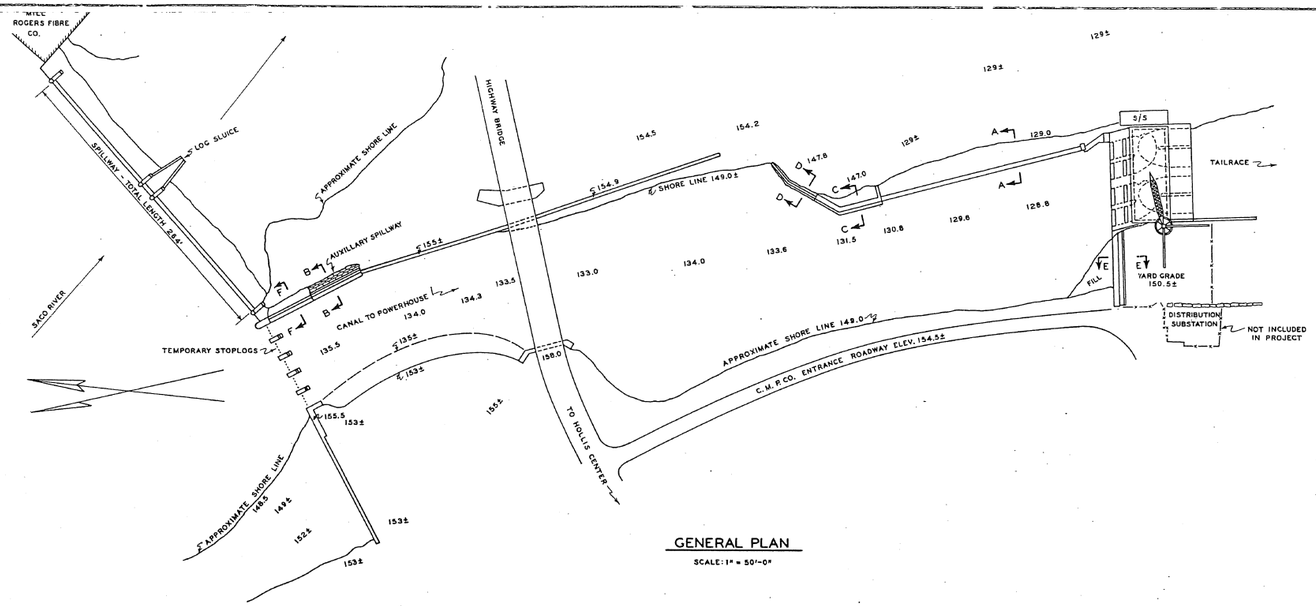
FLOOD PROFILES

SACO RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF HOLLIS, ME
(YORK CO.)

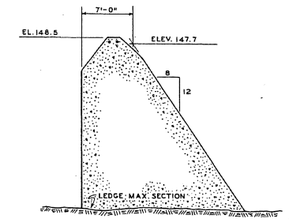
01P



GENERAL PLAN

SCALE: 1" = 50'-0"

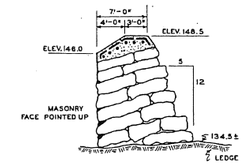
SCALE: 1" = 50'-0"



SECTION A-A

(CANAL)

SCALE: 1/8" = 1'-0"

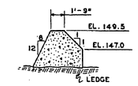


SECTION B-B

TYPICAL

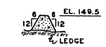
(CANAL)

SCALE: 1/8" = 1'-0"



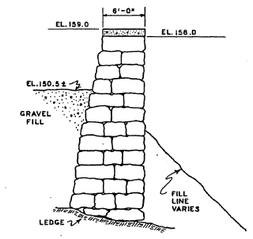
SECTION C-C

SCALE: 1/8" = 1'-0"



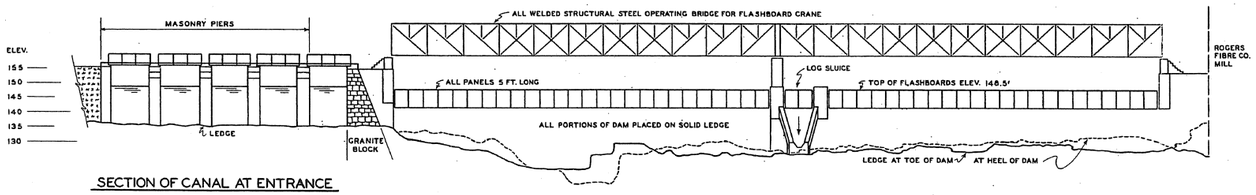
SECTION D-D

SCALE: 1/8" = 1'-0"



SECTION E-E

SCALE: 1/8" = 1'-0"

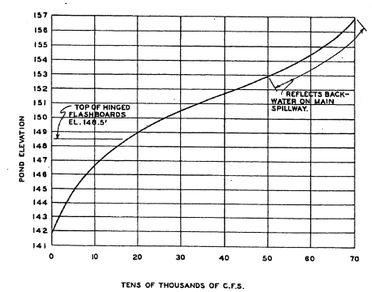


SECTION OF CANAL AT ENTRANCE

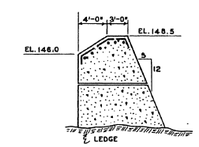
ELEVATION OF SPILLWAY LOOKING UPSTREAM

SCALE: 1" = 20'-0"

SCALE: 1" = 20'-0"

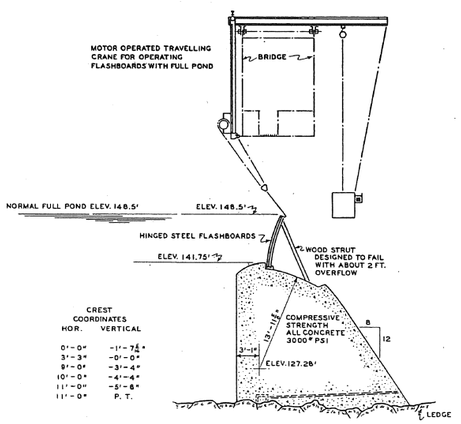


SPILLWAY DISCHARGE CURVE



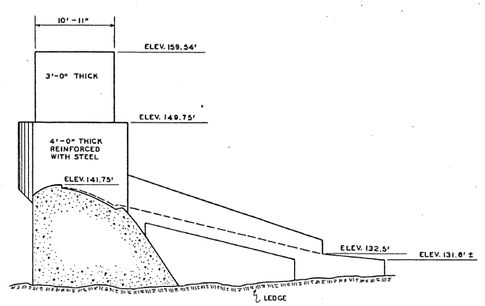
SECTION F-F

SCALE: 1/8" = 1'-0"



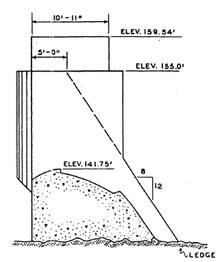
TYPICAL SECTION SPILLWAY SHOWING BRIDGE AND CRANE FOR FLASHBOARD OPERATION

SCALE: 1/8" = 1'-0"



SECTION THRU MIDDLE PIER

SCALE: 1/8" = 1'-0"



TYPICAL SECTION BRIDGE ABUTMENT

SCALE: 1/8" = 1'-0"

SCALE: 1/8" = 1'-0"

THIS MAP IS A REVISION OF EXHIBIT L SHEET 1 ORIGINALLY FILED.

CENTRAL MAINE POWER COMPANY

BY _____

FORD W. HARRIS, CHIEF ENGINEER

REV. 1 1-30-57

EXHIBIT L SHEET 1, REV. 1

CONCRETE DAM AND CANAL PLAN AND TYPICAL SECTIONS

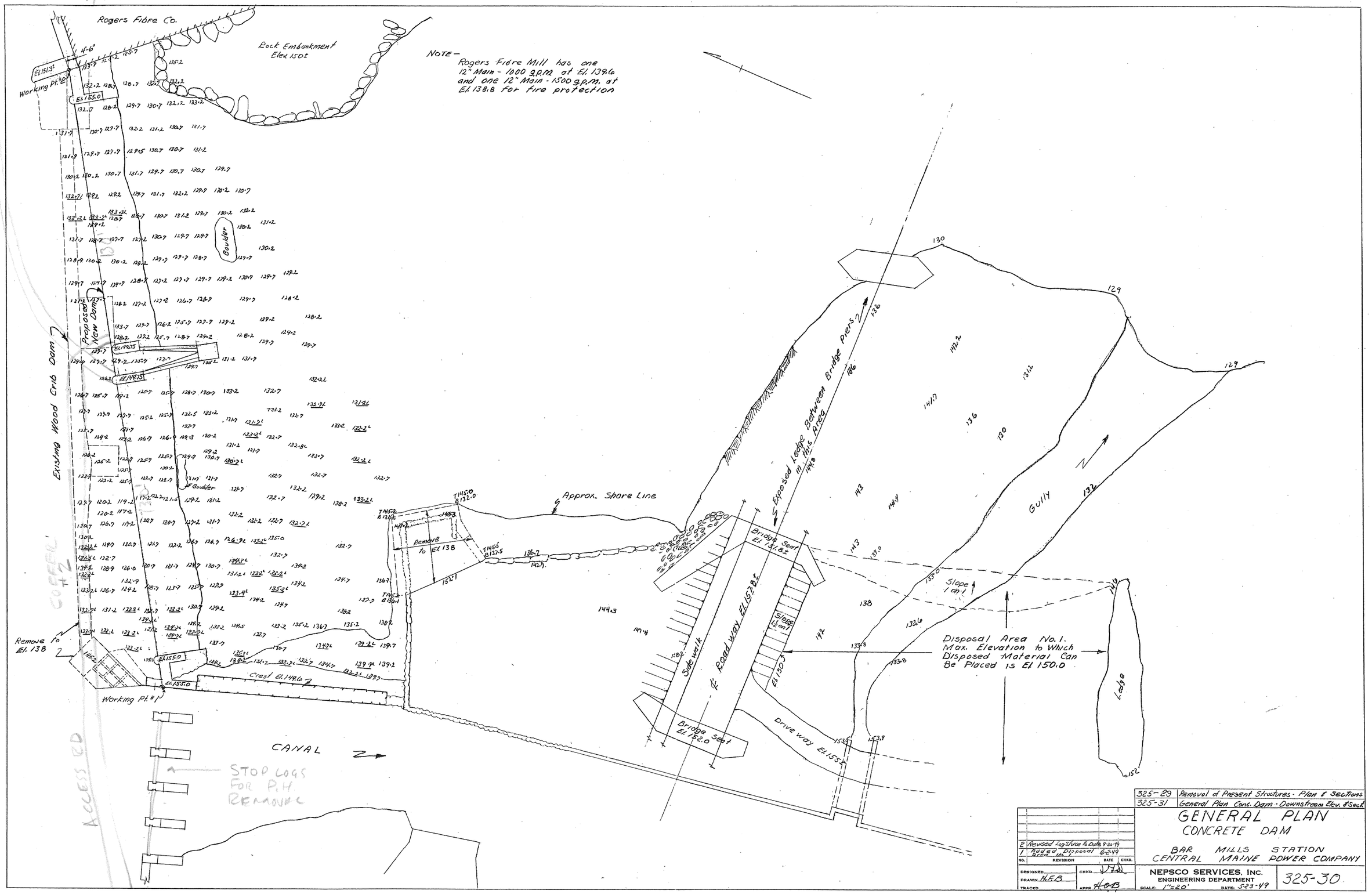
BAR MILLS REDEVELOPMENT

CENTRAL MAINE POWER COMPANY

DATE	SCALE	APPROVED
9-23-55	AS SHOWN	

325-9-6A

5/10/02


Rogers Fibre Co.
Rock Embankment Elev. 150±
NOTE – Rogers Fibre Mill has one 12" Main - 1000 g.p.m. at El. 139.6 and one 12" Main - 1500 g.p.m. at El. 138.8 for fire protection
Working Pt. #2
Existing Wood Crib Dam
Proposed New Dam
Boulder
Remove to El. 138
Crest El. 149.6
Working Pt. #1
CANAL
STOP LOGS FOR P.H. REMOVAL
ACCESS RD
Approx. Shore Line
Exposed Ledge Between Bridge Piers in this Area
Bridge Seat El. 141.8±
Bridge Seat El. 152.0
Side walk
Road way El. 157.8±
Drive way El. 155±
Slope 1 on 1
Disposal Area No. 1. Max. Elevation to Which Disposed Material Can Be Placed is El. 150.0
GULLY
Ledge
325-29 Removal of Present Structures - Plan & Sections
325-31 General Plan Conc. Dam - Downstream Elev. & Sect.
GENERAL PLAN
CONCRETE DAM
BAR MILLS STATION
CENTRAL MAINE POWER COMPANY
2 Revised Log Sluice to Date 9-20-49
1 Added Area Disposal 6-2-49
NO. REVISION DATE CHKD.
DESIGNED
DRAWN H.E.B.
TRACED
CHKD
APPR. H.E.B.
NEPSCO SERVICES, INC.
ENGINEERING DEPARTMENT
SCALE: 1"=20'
DATE: 5-23-49
325-30

Garrett Gustafson

Hollis-Buxton, ME
Bar Mills Canal Bridges #3333 #1525
WIN 19280-1.00

Sharp Crested Weir Hand Calcs

$$Q = \frac{2}{3} C_d \sqrt{2g} \cdot bH^{3/2}$$

$$C_d = 0.602 + 0.083 \frac{H}{P}$$

Weir Lengths

b(BP)	264 ft
b(BS)	90 ft
b(CP)	175 ft
b(CS)	120 ft

Weir Heights

P(BP)	5.07 ft
P(BS)	11.82 ft
P(CP)	11.82 ft
P(CS)	12.82 ft

Binary Controls

Flashboard down?	1

Discharge Coefficients

Cd(BP)	0.7577
Cd(BS)	0.6214
Cd(CP)	0.6214
Cd(CS)	0.6134

Water Surface Elevation

h	2.76 ft
EL(DATUM)	147.82 ft
EL	150.58 ft

Top of weir elevations

EL(BP)	141.07 ft
EL(BS)	147.82 ft
EL(CP)	147.82 ft
EL(CS)	148.82 ft

Streambed Elevations

ELsb(BP)	136 ft
ELsb(BS)	136 ft
ELsb(CP)	136 ft
ELsb(CS)	136 ft

Constants

g	32.2 ft/s^2

Flow Rates

q(BP)	31,401 cfs
q(BS)	1,374 cfs
q(Bar Mills)	32,775 cfs
q(CP)	2,672 cfs
q(CS)	922 cfs
q(POWER)	0 cfs
q(Canal)	3,594 cfs
q(TOTAL)	36,369 cfs

Calculation Summary

NAVD88	NGVD29	Q (cfs)
146.00	146.68	10555
146.50	147.18	12347
147.00	147.68	14258
147.50	148.18	16287
148.00	148.68	18,500
148.50	149.18	21,182
149.00	149.68	24,220
149.50	150.18	27,695
150.00	150.68	31,518
150.50	151.18	35,654
151.00	151.68	40,083
151.50	152.18	44,792
152.00	152.68	49,773
152.50	153.18	55,018
153.00	153.68	60,524

NOTE: Discharge coefficients were calculated with the Rehbock equation
NOTE: H equals h plus EL(DATUM) minus EL
NOTE: P equals EL minus ELsb

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Bar Mills	40	Q1.1	8230.00	128.30	134.59		135.13	0.007984	5.91	1392.97	361.67	0.53
Bar Mills	40	Q10	22729.00	128.30	140.04		140.68	0.003371	6.39	3558.60	429.39	0.39
Bar Mills	40	Q25	26764.00	128.30	141.29		141.96	0.002986	6.54	4099.92	437.48	0.37
Bar Mills	40	Q50	29779.00	128.30	142.18		142.87	0.002774	6.64	4492.00	443.25	0.37
Bar Mills	40	Q100	32775.00	128.30	143.03		143.74	0.002610	6.75	4869.86	448.74	0.36
Bar Mills	40	Q500	40200.00	128.30	145.01		145.77	0.002315	7.00	5772.62	461.58	0.35
Bar Mills	30	Q1.1	8230.00	125.70	134.08		134.29	0.002117	3.63	2269.88	451.09	0.28
Bar Mills	30	Q10	22729.00	125.70	139.87		140.19	0.001289	4.57	4974.20	478.67	0.25
Bar Mills	30	Q25	26764.00	125.70	141.14		141.50	0.001228	4.79	5587.42	483.01	0.25
Bar Mills	30	Q50	29779.00	125.70	142.05		142.43	0.001190	4.94	6028.04	488.92	0.25
Bar Mills	30	Q100	32775.00	125.70	142.91		143.31	0.001159	5.09	6451.21	494.27	0.25
Bar Mills	30	Q500	40200.00	125.70	144.92		145.37	0.001094	5.41	7452.67	502.52	0.24
Bar Mills	20	Q1.1	8230.00	125.60	133.79		134.03	0.001743	3.88	2121.27	330.76	0.27
Bar Mills	20	Q10	22729.00	125.60	139.51		139.98	0.001600	5.52	4245.10	421.11	0.28
Bar Mills	20	Q25	26764.00	125.60	140.77		141.30	0.001573	5.85	4789.38	438.01	0.29
Bar Mills	20	Q50	29779.00	125.60	141.66		142.23	0.001552	6.06	5183.48	442.50	0.29
Bar Mills	20	Q100	32775.00	125.60	142.51		143.11	0.001536	6.26	5561.03	446.76	0.29
Bar Mills	20	Q500	40200.00	125.60	144.50		145.18	0.001501	6.71	6456.95	456.71	0.29
Bar Mills	10	Q1.1	8230.00	123.30	133.75	128.84	133.88	0.000800	2.87	3042.74	484.77	0.19
Bar Mills	10	Q10	22729.00	123.30	139.55	131.22	139.81	0.000801	4.23	6092.99	550.00	0.21
Bar Mills	10	Q25	26764.00	123.30	140.82	131.74	141.12	0.000801	4.50	6793.54	550.00	0.21
Bar Mills	10	Q50	29779.00	123.30	141.73	132.16	142.05	0.000800	4.69	7291.63	550.00	0.21
Bar Mills	10	Q100	32775.00	123.30	142.59	132.54	142.93	0.000800	4.86	7763.94	550.00	0.21
Bar Mills	10	Q500	40200.00	123.30	144.59	133.41	144.99	0.000800	5.25	8866.42	550.00	0.22