HYDROLOGY

There are two sets of storm event discharge rates for this site. Below are the values used in the Plan of Action developed by Clough Harbor & Associates.

HYDROLOGY DATA IN P.O.A. (NOT USED)
Q2 = 1090 c.f.s.
Q50 = 2560 c.f.s.
Q100 = 2890 c.f.s.
Q500 = 3640 c.f.s.

Much more complete information was developed by Alex Mann, which included Drainage Basin Characteristics and storm event discharge rates calculated by the U.S.G.S. Regression Equations. These values are reported immediately below and used for the hydraulic analysis of this scour prone site.

SUMMARY OF HYDROLOGY DATA
Drainage Area = 131.9 sq. mi.
Check Discharge (Q100) = 3590 c.f.s.
Design Discharge (Q500) = 4575 c.f.s.
Ordinary High Water (Q1.1) = 726 c.f.s.

Reported By: Robert S. Bulger
Date: August 21, 2013

HYDRAULICS

Before this hydraulics study was begun, a Plan of Action for scour abatement had been completed based on an earlier hydraulics study by Clough Harbor Associates. The principal reasons for the second hydraulics model are: 1) there was no input data and other missing information restricting the use of the hydraulic model from the P.O.A.; 2) a 0.5 foot error on the reported low chord elevation made datum conversion from the P.O.A. untrustworthy; 3) all of the P.O.A.'s storm event discharge rates are low compared to results for the USGS Regression Equation as calculated by the MaineDOT; 4) the plan did not locate/identify the streambed section that was used to design their single countermeasure, sized riprap; and 5) the P.O.A. did not provide the channel shear stress design value, which is necessary for designing most of the scour countermeasures.

Though the above reasons made creation of an independent hydraulic model necessary and initial understanding of the P.O.A. data difficult, information provided by the P.O.A. was the key to establishing the new model's boundary conditions. Another important reason for the new hydraulic model was to determine which reach section is subjected to the greatest scour forces. All model sections were considered for maximum hydraulic shear, by review of the detailed output. Particular attention was given to the following sections: 1) at the ends of the upstream wingwall; 2) at the upstream fascia; and 3) at the downstream fascia. The downstream fascia section proved critical.

The P.O.A. relied on another report, the USGS Scientific Investigation Report 2008-5099, to calibrate their hydraulics model on observed floods. See the seven pages excerpted from this USGS Report as the P.O.A.'s Attachment 'L'. The P.O.A. also included three sheets of HEC-RAS output. The P.O.A. output included: 1) Water Surface elevations for their Q10 through Q500 discharge values at Reach Stations 1, 2, 2.5, 3 and 4; 2) a bridge section graph at Station 2.5 with hydraulic data for Q100; and 3) the hydraulic depth, velocity, and left and right flooded bank width for a "critical design flow". Report 2008-5099 concludes that the observable site scour is due to the highest discharge rate during the life of the bridge, Q73.

From past experience, HEC-RAS models are often set up with integer stations for the streambed representing 100 feet. In the P.O.A.'s hydraulic output, the most downstream station is '1', the most upstream is '4', and the bridge is located at '2.5'. The assumption that these station integers represent hundreds of feet, provides essential data, the location of the P.O.A. upstream and downstream streambed sections. This assumption appears well justified by the extremely close match between the streambed sections of the P.O.A.'s upstream section '4', and the survey models section at Station 4+50. P.O.A. Station '4' is 150 feet upstream of the bridge centerline; Streambed Station 4+50 cut from the survey is 132.2 feet upstream of the bridge centerline, only 17.8 feet apart. On waterways the size of Umcolcus Stream, it is rare to have information as dependable as the P.O.A. data. Still, this analysis should be considered only comparative, not rigorously quantitative.

The boundary conditions in the new hydraulic analysis were determined by matching flow conditions for the paired upstream sections described above. The aim was to match the P.O.A.'s discharge rates, flow velocities, and water surface elevations as closely as possible. The stream slope was taken from the P.O.A., as 0.002 feet/feet. At first, the P.O.A.'s water surface elevations at Station 4 were used as the second, required boundary condition for each discharge rate. It was quickly determined that more consistent and logical hydraulic results were obtained by setting tailwater elevations as boundary conditions, not the headwater elevations. A normally-shaped profile of tailwater elevations was developed at the extreme downstream section by using step-backwater runs to match headwater elevations at the P.O.A.'s Station 4.

For this report, all output station numbers are based on the new model, i.e. the stations cut from the survey. The centerline of the bridge roadway intersects the reach at Station 3+17.8. To clarify which discharge rate is considered, this report prefaces P.O.A. discharge values with the letter 'U' for 'Unused for Design', and the MaineDOT discharge values with the letter 'M'.

A local resident has placed boulders and other fill inside the channel at the westerly side of the downstream fascia. The restricted downstream opening is the effective existing opening, which measures 514 square feet. This material will be removed with the installation of the channel scour protection. The area of opening will be restored to 600 square feet, matching the opening at the upstream fascia.

DESIGN SCOUR PARAMETERS

	Upstream End of Wingwall	Upstream Fascia	Downstream Fascia
Q500 Channel Shear	1.03 psf	3.85 psf	4.87 psf
Q500 Channel Velocity	4.66 fps	7.98 fps	9.36 fps
Q500 Flow Depth	9.88 feet	12.79 feet	10.1 feet

HYDRAULICS SUMMARY

	Q1.1	Q10	Q25	Q50	Q100	Q500
Input Tailwater Elevation (feet)	610.03	616.16	617.05	617.70	618.26	618.63
Headwater Elevation (feet)	611.53	616.33	617.25	617.92	618.52	619.08
Outlet Velocity (f.p.s.)	4.01	5.70	6.39	6.85	7.38	9.36
Clearance (feet)	7.66	3.06	2.23	1.63	1.12	0.76

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.
Most storm event profiles exhibit a deep drawdown. See plotted stream profiles in the Hydrology/Hydraulics Data appendix. Headwater is given 17 feet upstream of bridge.

Consider the difference in the design parameters for riprap sizing between the MaineDOT Q500 value of 4575 c.f.s. and Clough Harbor & Associates' Q500 value of 3640 c.f.s., a 26% difference in discharge rates. By the MDOT discharge, the design velocity is 9.36 f.p.s. and the design hydraulic depth is 10.1 feet. By the Clough Harbor & Associates value, the design velocity is 7.5 f.p.s., and the design hydraulic depth is 11.0 feet. Note that the storm event discharge rates used in the P.O.A. seem to correlate better with the USGS Scientific Investigative Report 2008-5099.

SCOUR

EMBANKMENT PROTECTION

At the outset, the team foresaw that embankment protection would require sized rip rap. Other countermeasures, such as a cable mat system, would not be stable on slopes as steep as the embankment side slopes. As required in previous scour Preliminary Design Reports, Heavy Rip Rap - Item 610.16 shall meet the latest specification requirements of 703.28 Heavy Rip Rap. The Heavy Rip Rap shall be used to protect the cone-shaped slopes from the end of each wingwall to the intersection with the new channel protection or the existing ground. Heavy Rip Rap shall also be used, as shown on the Plans, to protect all re-graded sideslopes perpendicular channel. By the P.O.A. the required d_{50} of the Heavy Rip Rap is 1.1 feet. By this analysis the minimum d_{50} calculated is 1.68 feet. Specify a minimum d_{50} of 1.75 feet and a blanket depth of 3.5 feet. Also stipulate that the d_{100} is 3.5 feet.

CHANNEL SCOUR COUNTER-MEASURE OPTION

Please note that Appendix M of the P.O.A. states that the clear-water channel scour should be estimated as zero. It also states that the effective mean diameter of the bed material, D_m, is 0.20 feet. On the same page, the P.O.A. provides a conversion to indicate D_{50} is 0.16 feet. This is assurance that the underlying bed material is relatively clean, i.e. has a low amount of fine soil, and indicates that a gravel filtering layer is not required.

A Cable-Connected Articulating Concrete Block system is recommended for the channel protection. From the file "Scour Protection Design Computations", the critical channel shear, τ_{des}, is 4.87 pounds per square foot. Concrete block systems can be designed for extremely high magnitude of streambed shear. In order to determine the correct block size, two employees of International Erosion Control Systems were contacted and the company's "Block Selection Guide, version 3.0" was used. The table shows that block "CC70", a closed block, can safely resists a channel hydraulic shear of up to 11.6 pounds per square foot, installed at with transverse slopes as steep as 3 horizontal to 1 vertical. This block is 8.5 inches thick. A design example from I.E.C.S under calculated the target safety factor for an installation to protect a bridge substructure, but both the sized rip rap and the concrete block systems in this report satisfy target safety factors consistent with H.E.C. 23.

Respectfully Submitted,

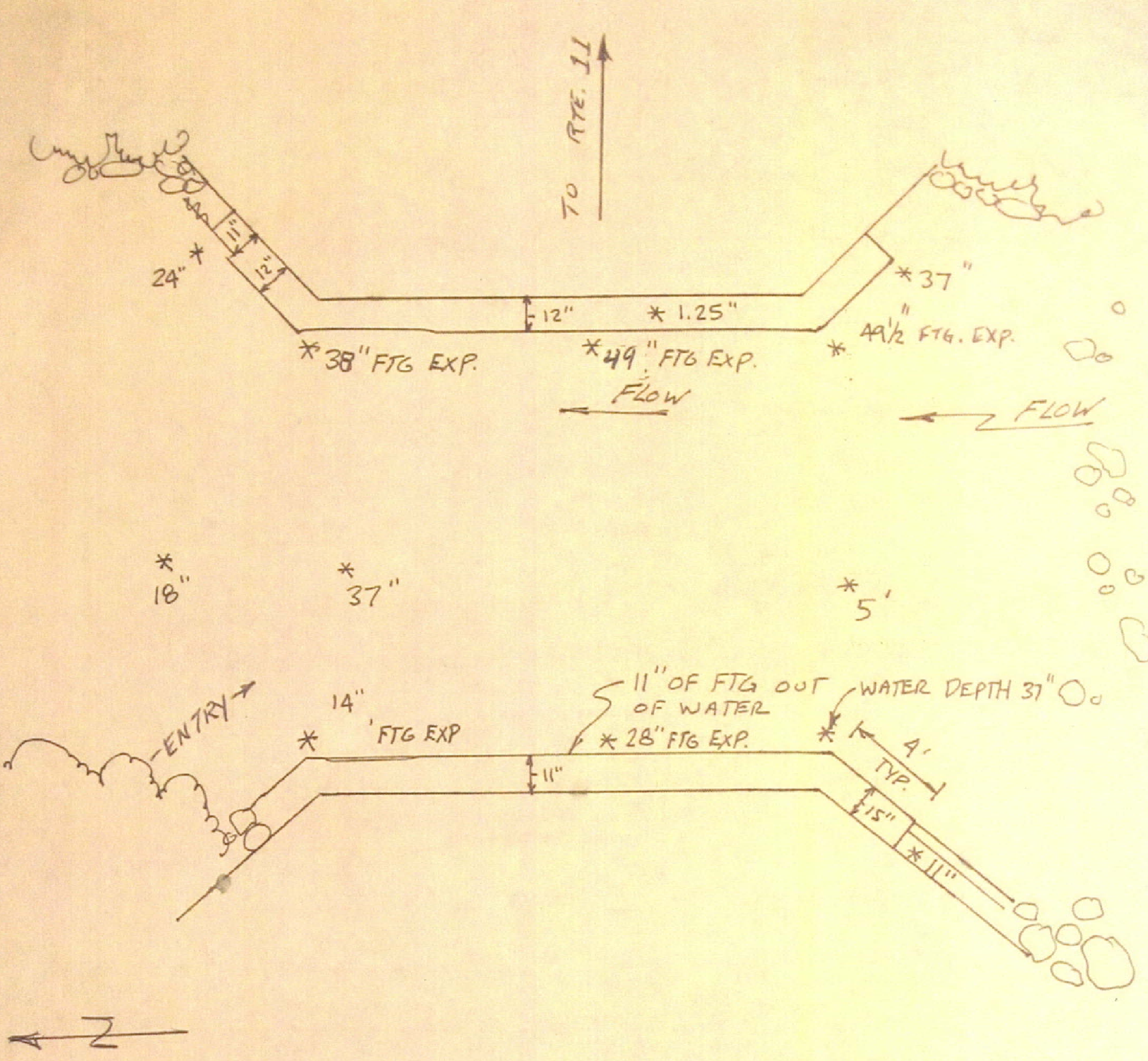
Robert S. Bulger

Robert S. Bulger

March 28, 2013

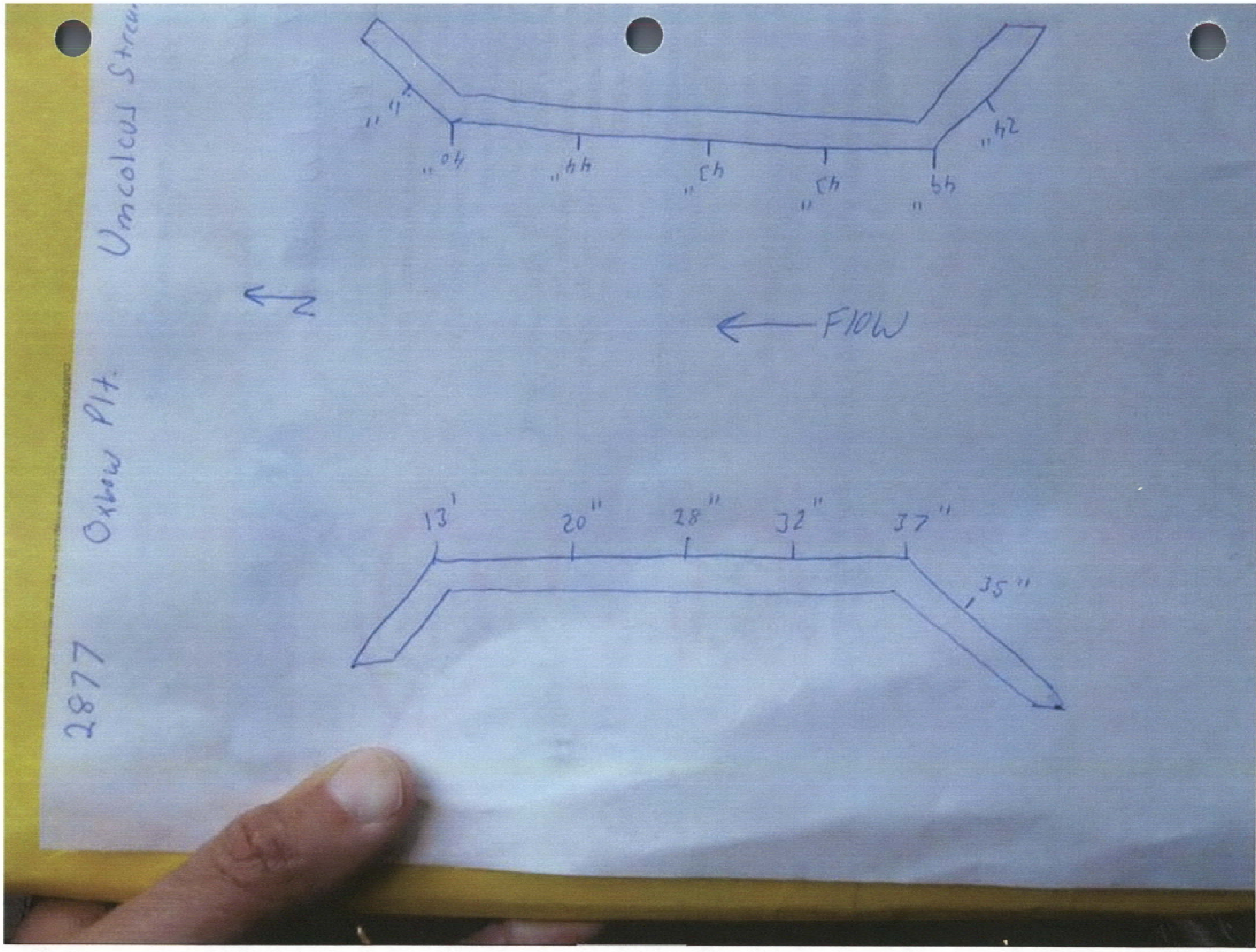
BRIDGE #2877 OXBOW PLT, UMCOLCUS STREAM

2877 OXBOW PLT. UMCOLCUS



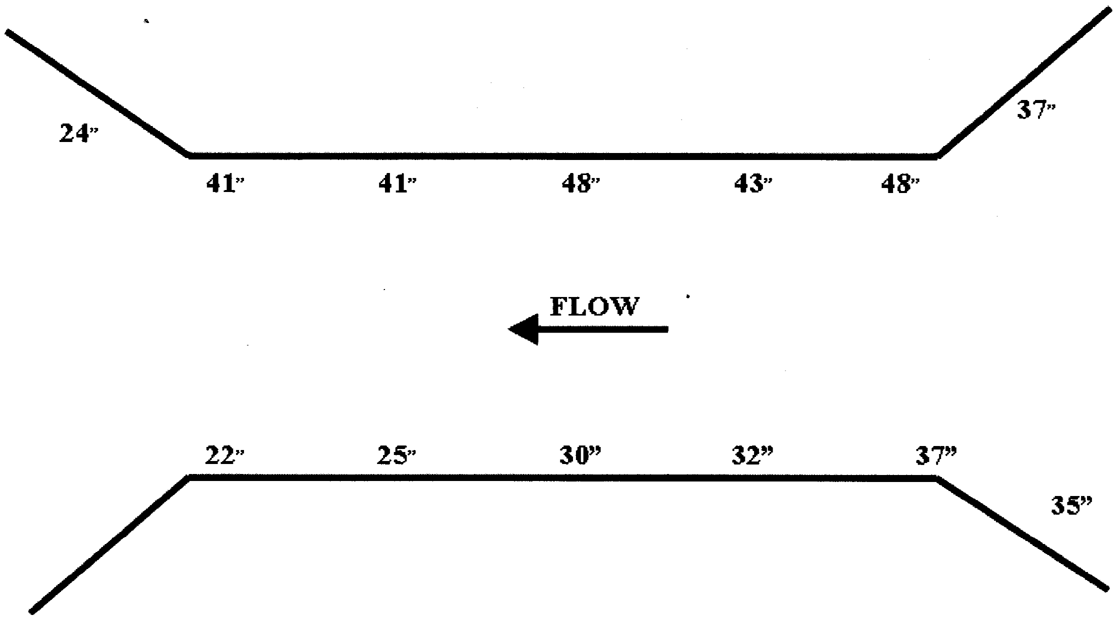
WATER TEMP: 67°
OVERCAST
CURRENT: SLIGHT

07-2406



Scour Chart	No significant changes since last insp.

2877	Oxbow Plt	Umcolcus Stream	6/2/2008



sjh

Exposed footing		Exposure Sketch	
2877	Oxbow Pit	Umcolcus Stream	9/9/2009

APPENDIX D

Hydrology, Hydraulic and Scour Countermeasure Data

Project Name:	Bridge Scour	**PIN:**	
Stream Name:	Umcolcus Stream	**Town:**	Oxbow Plt
Bridge Name:		**Bridge No.**	2877
Route No.	Oxbow Rd	**USGS Quad:**	
Analysis by:	A.W.Mann	**Date:**	Revised 12/2/2008, 6/11/2008

Peak Flow Calculations by USGS Regression Equations (Hodgkins, 1999)

Enter data in blue cells only!

	km^2	mi^2	ac	
A	212.311	82.0		RSB Watershed Area
W	34.986	13.5		Wetlands area (by NWI)

Use just one consistent set of units:

A (km^2)	212.311	**Conf Lvl**	0.67
W (%)	16.48		

Ret Pd T (yr)	Peak Flow Estimate Lower	Q_T (m^3/s)	Upper	Q_T (ft^3/s)
1.1		20.56		725.9
2	26.46	36.84	51.30	1300.9
5	37.97	52.97	73.90	1870.3
10	45.63	64.32	90.67	2271.1
25	55.24	78.89	112.68	2785.8
50	62.28	89.94	129.88	3175.8
100	69.63	101.70	148.55	3591.0
500	86.03	129.55	195.09	4574.5

Worksheet prepared by:
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Chief Hydrologist
Maine Dept. Transportation
Augusta, ME 04333-0016
207-287-1105
Charles.Hebson@maine.gov

Reference:

Hodgkins, G., 1999.
Estimating the magnitude of peak flows for streams in Maine for selected recurrence intervals
Water-Resources Investigations Report 99-4008
US Geological Survey, Augusta, Maine

$Q_T = b \times A^a \times 10^{-wW}$

PRELIM. BY R.S. Bulger DATE 5/1/13 PROJ. NO. 17880.00 FILE NO. ____ OF ____
FINAL CHK. BY ____ DATE ____ LOCATION Oxbow Pltn - Umcolcus SH. NO. 1 OF 2
ITEM NO. ____ SUBJECT Hydrology / Hydraulics Data

DATA FROM CHA's "Scour Critical Bridge P.O.A."

ABSTRACT Some documentation is necessary to clarify the data and assumptions used for design particularly because of contradictions in the Plan of Action document provided by C.H.A.

Data Contradictions, Errors, & Poorly Documented Parameters

Cover sheet

a.) gives closure trigger as El. 46.0ft (perhaps by existing plans)?

Attachment 'A' a) gives low chord as 49.0' & roadway as 51.9' but superstructure depth is ≫ 2.9'. Bottom should be 47.76+.5 ≈ 48.26' (Lt., West)

48.70+.33 = 49.03' (Rt., East)

Attachment 'D' gives "low chord elevations" as 47.76 & 48.70.

Use 48.64' as average low chord and 48.26' near left abut, 49.03 rt.

(Rdwy: 51.95' Lt. & 52.69' Rt. Avg = 52.32'; depth = 3.68': OK)

Reliable Data Utilized for MDOT HEC-RAS Runs

2008 U.S.G.S. Report (Included as Attachment 'L')

a) from page 9: Use 30.1ft. to Left Bank; 128.5ft. to Right, 6.5ft. deep flow and 3.5 f.p.s. velocity for main channel at 2710 c.f.s. "peak" flow.

b) from page 12: Peak flow recurrence ~13 years, slope at bridge ~0.002

DATA CONVERSION CALCULATION

Abut. #1 Finish G. 622.45 − 51.94 = 570.51
Br. Seat 618.28 − 47.76 = 570.52
Br. Curb 622.97 − (47.76 + .5 + 4.23) = 570.48

Abut. #2 F.G. 623.19 − 52.72 = 570.47
Br. Seat 619.19 − 48.70 = 570.49
Br. Curb 623.75 − (48.70 + .33 + 4.23) = 570.49

Use Datum difference of 570.5 from Existing to Current Plans

Low Chord Elevs.: Lt. 48.26 + 570.5 = 618.76 ft. Rt. 49.03 + 570.5 = 619.53'

Use Datum Difference of (570.5 − 56.7 =) 513.8' from 2008 Report to current NAVD '88

Water Marks from Plans

From the Existing Plans

The plans show an "ice-impacted" high spring flood of (46.5+570.5=) Elev. 617.00 and a low water of (38.0+570.5=) 608.5 ft. The Basal flow, between the months of July and October mean flows is between 21 c.f.s. & 35 c.f.s.; Say 28 c.f.s.

From the Survey on 9/8/2010 the stream Elevation ~70' upstream is El. 609.8, at 15' upstream (of the U.S. fascia) ≈ Elev. 608.6'. So 609.8' @ Sta. 418± & 608.6' @ Sta. 363±

Combination of P.O.A.'s Data From "U.S.G.S. Scient. Inv. Report 2008-5099" and WIN 17880.00 SURVEY DATA & M.D.O.T. HYDROLOGY

Though the available data on the Umcolcus Stream Bridge cited in the P.O.A. is quite limited, it is the best data available for modelling the site's hydraulic characteristics. The U.S.G.S report information included in the P.O.A. include:

1.) HEC-RAS: bridge section hydraulics at Q100; and bridge section graph;
2.) HEC-RAS output including W.S. Elevs. at Q10-Q500, Stations 1, 2, 2.5, 3 and 4.
and 3.) Hydraulic depth, velocities and left and right flooded bank at Q73.

C.H.A.'s P.O.A. contains enough information to calculate the datum difference from the 2008 U.S.G.S. report to the Plans of Existing. Therefore the NAVD '88 values for the 2008 report's W.S. Elevations, particularly those at Sta. 4 were calculated and used as target W.S. elevations in this report's HEC-RAS runs.

Discharge Rates used by CHA, Section & Bridge Output

Model	River Sta	Profile	Total Q (cfs)	Avg Vel. (ft/s)	W.S. Elev. (ft)	Crit W.S. (ft)	E.G. Elev. (ft)	E.G. Slope (ft/ft)	Flow Area (sq ft)	Top Width (ft)	Froude Numbr
oxbow rd	4	Q 2	1090	2.77	100.7	98.3	100.83	0.001249	404.9	150.73	0.25
oxbow rd	4	Q 5	1540	3.14	101.5	98.7	101.67	0.001238	539.5	200.51	0.26
oxbow rd	4	Q 10	1860	3.31	102.1	99	102.22	0.001177	653.5	211.62	0.26
oxbow rd	4	Q 25	2260	3.45	102.7	99.3	102.89	0.001088	792.3	213.21	0.25
oxbow rd	4	Q 50	2560	3.53	103.2	99.6	103.37	0.001022	894.7	214.37	0.25
oxbow rd	4	Q 100	2890	3.59	103.7	99.8	103.89	0.000953	1006	215.63	0.24
oxbow rd	4	Q 500	3640	3.69	104.9	100	105.07	0.000811	1261	218.47	0.23
oxbow rd	4	T-Flood	2710	3.56	103.4	99.7	103.61	0.00099	945.5	214.94	0.25
oxbow rd	3	Q 2	1090	3.06	100.7	96.2	100.8	0.000665	356.7	140.97	0.21
oxbow rd	3	Q 5	1540	3.9	101.4	96.8	101.63	0.000943	395.3	175.51	0.25
oxbow rd	3	Q 10	1860	4.42	101.9	97.2	102.18	0.00112	420.5	198.44	0.27
oxbow rd	3	Q 25	2260	5.02	102.5	97.7	102.84	0.001316	450.3	199.74	0.3
oxbow rd	3	Q 50	2560	5.42	102.9	98	103.33	0.001443	472	200.69	0.32
oxbow rd	3	Q 100	2890	5.83	103.3	98.4	103.85	0.001565	495.4	201.72	0.33
oxbow rd	3	Q 500	3640	6.64	104.3	99.1	105.03	0.001768	548.6	204.05	0.36
oxbow rd	3	T-Flood	2710	5.61	103.1	98.2	103.56	0.001501	482.7	201.16	0.32
oxbow rd	2.5		Bridge								
oxbow rd	2.5	Q 2	1090	4.79	100.7		100.8		465.2		
oxbow rd	2.5	Q 5	1540	6.14	101.4		101.63		465.2		
oxbow rd	2.5	Q 10	1860	7.03	101.9		102.18		465.2		
oxbow rd	2.5	Q 25	2260	8.11	102.5		102.84		465.2		
oxbow rd	2.5	Q 50	2560	8.89	102.9		103.33		465.2		
oxbow rd	2.5	Q 100	2890	9.76	103.3		103.85		465.2		
oxbow rd	2.5	Q 500	3640	11.9	104.3		105.03		465.2		
oxbow rd	2.5	T-Flood	2710	9.29	103.1		103.56		465.2		
oxbow rd	2	Q 2	1090	4.32	100.4	97.9	100.46	0.002316	252.4	263.49	0.35
oxbow rd	2	Q 5	1540	5.43	101	98.6	101.15	0.003136	283.6	271.6	0.41
oxbow rd	2	Q 10	1860	6.14	101.4	99	101.6	0.003663	303.1	276.69	0.45
oxbow rd	2	Q 25	2260	6.94	101.8	99.4	102.14	0.004253	325.8	282.58	0.49
oxbow rd	2	Q 50	2560	7.49	102.1	99.8	102.54	0.004646	341.9	286.77	0.51
oxbow rd	2	Q 100	2890	8.05	102.5	100	102.97	0.005025	359.1	290.53	0.54
oxbow rd	2	Q 500	3640	9.13	103.2	101	103.91	0.005636	398.5	298.53	0.58
oxbow rd	2	T-Flood	2710	7.75	102.3	99.9	102.73	0.004825	349.8	288.63	0.53
oxbow rd	1	Q 2	1090	4.16	99.78	98.9	99.99	0.003765	365.9	266.92	0.42
oxbow rd	1	Q 5	1540	4.53	100.2	99.6	100.43	0.003763	479.1	277.52	0.43
oxbow rd	1	Q 10	1860	4.75	100.5	99.7	100.69	0.003761	549.8	281.16	0.44
oxbow rd	1	Q 25	2260	5	100.7	99.9	101	0.003766	630.9	285.29	0.44
oxbow rd	1	Q 50	2560	5.17	100.9	100	101.22	0.003762	688.4	288.17	0.45
oxbow rd	1	Q 100	2890	5.34	101.1	100	101.44	0.003761	748.6	291.17	0.45
oxbow rd	1	Q 500	3640	5.69	101.6	101	101.91	0.003763	876.2	297.41	0.46
oxbow rd	1	T-Flood	2710	5.25	101	100	101.32	0.003762	716.1	289.56	0.45

Boundary Condition: Both U.S. & D.S. slopes set to 0.00376

Consultant Flow Data - Discharges Do Not Match MeDOT

Model	River Sta	Profile	Total Q (cfs)	Avg Vel. (ft/s)	W.S. Elev. (ft)	Crit W.S. (ft)	E.G. Elev. (ft)	E.G. Slope (ft/ft)	Flow Area (sq ft)	Top Width (ft)	Froude Numbr
oxbow rd	4	Q 2	1090	2.77	100.7	98.3	100.83	0.001249	404.9	150.73	0.25
oxbow rd	4	Q 5	1540	3.14	101.5	98.7	101.67	0.001238	539.5	200.51	0.26
oxbow rd	4	Q 10	1860	3.31	102.1	99	102.22	0.001177	653.5	211.62	0.26
oxbow rd	4	Q 25	2260	3.45	102.7	99.3	102.89	0.001088	792.3	213.21	0.25
oxbow rd	4	Q 50	2560	3.53	103.2	99.6	103.37	0.001022	894.7	214.37	0.25
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oxbow rd	4	Q 500	3640	3.69	104.9	100	105.07	0.000811	1261	218.47	0.23
oxbow rd	4	T-Flood	2710	3.56	103.4	99.7	103.61	0.00099	945.5	214.94	0.25
oxbow rd	3	Q 2	1090	3.06	100.7	96.2	100.8	0.000665	356.7	140.97	0.21
oxbow rd	3	Q 5	1540	3.9	101.4	96.8	101.63	0.000943	395.3	175.51	0.25
oxbow rd	3	Q 10	1860	4.42	101.9	97.2	102.18	0.00112	420.5	198.44	0.27
oxbow rd	3	Q 25	2260	5.02	102.5	97.7	102.84	0.001316	450.3	199.74	0.3
oxbow rd	3	Q 50	2560	5.42	102.9	98	103.33	0.001443	472	200.69	0.32
oxbow rd	3	Q 100	2890	5.83	103.3	98.4	103.85	0.001565	495.4	201.72	0.33
oxbow rd	3	Q 500	3640	6.64	104.3	99.1	105.03	0.001768	548.6	204.05	0.36
oxbow rd	3	T-Flood	2710	5.61	103.1	98.2	103.56	0.001501	482.7	201.16	0.32
oxbow rd	2.5		Bridge								
oxbow rd	2.5	Q 2	1090	4.79	100.7		100.8		465.2		
oxbow rd	2.5	Q 5	1540	6.14	101.4		101.63		465.2		
oxbow rd	2.5	Q 10	1860	7.03	101.9		102.18		465.2		
oxbow rd	2.5	Q 25	2260	8.11	102.5		102.84		465.2		
oxbow rd	2.5	Q 50	2560	8.89	102.9		103.33		465.2		
oxbow rd	2.5	Q 100	2890	9.76	103.3		103.85		465.2		
oxbow rd	2.5	Q 500	3640	11.9	104.3		105.03		465.2		
oxbow rd	2.5	T-Flood	2710	9.29	103.1		103.56		465.2		
oxbow rd	2	Q 2	1090	4.32	100.4	97.9	100.46	0.002316	252.4	263.49	0.35
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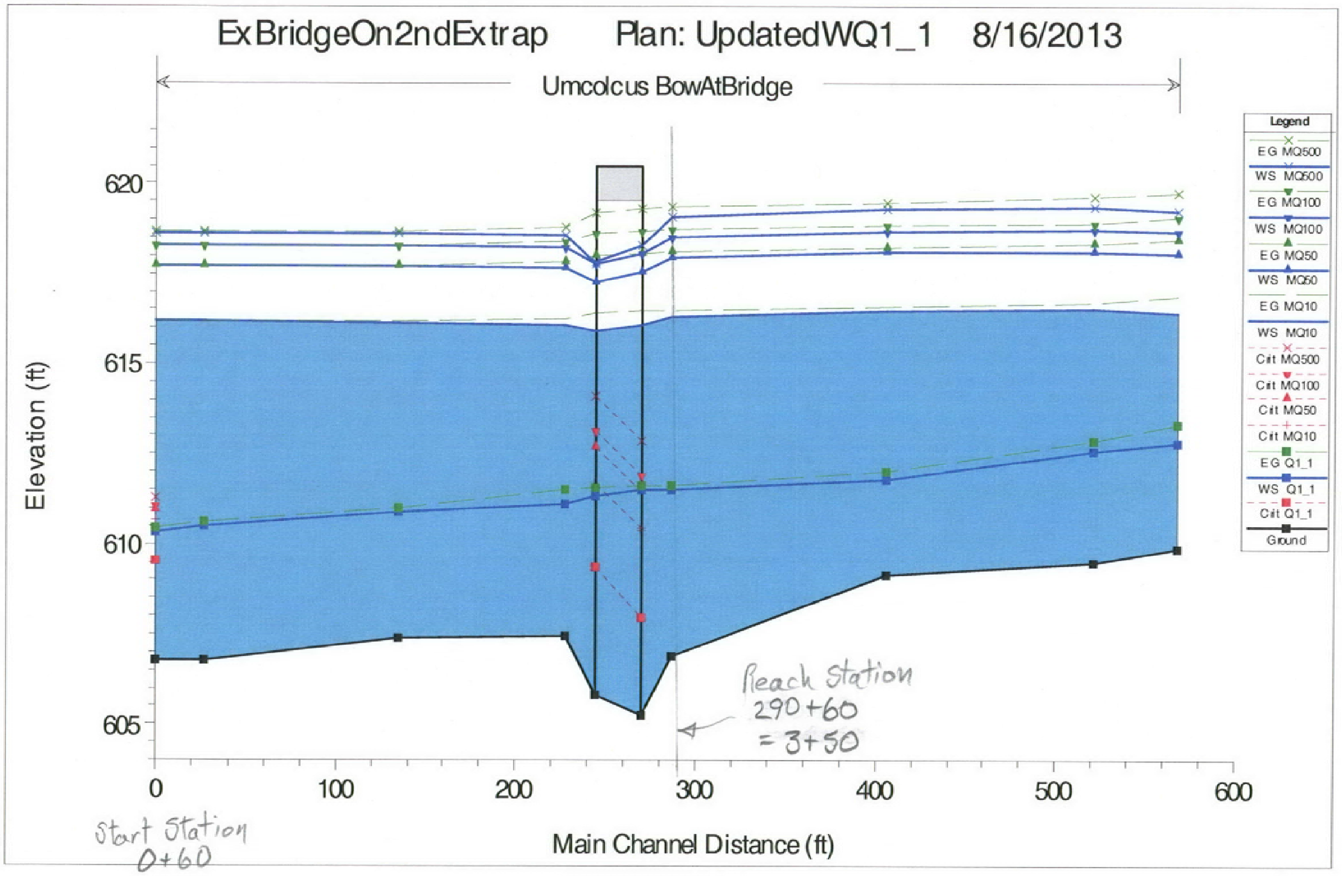
oxbow rd	1	Q 10	1860	4.75	100.5	99.7	100.69	0.003761	549.8	281.16	0.44
oxbow rd	1	Q 25	2260	5	100.7	99.9	101	0.003766	630.9	285.29	0.44
oxbow rd	1	Q 50	2560	5.17	100.9	100	101.22	0.003762	688.4	288.17	0.45
oxbow rd	1	Q 100	2890	5.34	101.1	100	101.44	0.003761	748.6	291.17	0.45
oxbow rd	1	Q 500	3640	5.69	101.6	101	101.91	0.003763	876.2	297.41	0.46
oxbow rd	1	T-Flood	2710	5.25	101	100	101.32	0.003762	716.1	289.56	0.45

Boundary Condition: both U.S. & D.S. slopes set to 0.00376



ExBridgeOn2ndExtrap Plan: UpdatedWQ1_1 8/16/2013
Umcolcus BowAtBridge
Elevation (ft)
620
615
610
605
0
100
200
300
400
500
600
Main Channel Distance (ft)
Legend
EG MQ500
WS MQ500
EG MQ100
WS MQ100
EG MQ50
WS MQ50
EG MQ10
WS MQ10
Crit MQ500
Crit MQ100
Crit MQ50
Crit MQ10
EG Q1_1
WS Q1_1
Crit Q1_1
Ground
Reach Station
290+60
= 3+50
Start Station
0+60

Reach	River Sta	Profile	Q (cfs)	W.S. Elev. (ft)	BR Vel (ft/s)	Vel Lt (ft/s)	Vel Chn (ft/s)	Vel Rt (ft/s)	Flow Area (sq ft)	Ch Shear (lb/sq ft)	Top Width(ft)	E.G. (ft/ft)	Hydr D (ft)	Froude No.
Umcolcus	600.00	MQ1_1	726	612.82		1.16	5.61	1.03	133.96	1.39	70.30	0.010156	1.91	0.66
Umcolcus	600.00	UQ10	1860	615.45		1.71	5.86	1.57	415.60	1.17	140.36	0.003908	2.96	0.47
Umcolcus	600.00	MQ10	2270	616.44		1.70	5.60	1.62	567.18	1.00	163.10	0.002787	3.48	0.41
Umcolcus	600.00	UQ50	2560	616.99		1.72	5.59	1.68	660.85	0.97	173.87	0.002454	3.8	0.39
Umcolcus	600.00	UQ100	2890	617.54		1.75	5.63	1.75	759.15	0.96	184.50	0.002238	4.11	0.38
Umcolcus	600.00	MQ50	3176	618.05		1.77	5.62	1.78	854.73	0.93	194.29	0.002028	4.4	0.36
Umcolcus	600.00	MQ100	3590	618.65		1.81	5.70	1.84	975.66	0.93	206.00	0.001879	4.74	0.35
Umcolcus	600.00	UQ500	3640	618.73		1.81	5.71	1.84	991.10	0.93	207.44	0.001859	4.78	0.35
Umcolcus	600.00	MQ500	4575	619.26		2.10	6.56	2.15	1103.76	1.21	217.72	0.002261	5.07	0.39
Umcolcus	553.00	MQ1_1	726	612.6		1.20	4.24	1.09	191.55	0.81	119.12	0.006287	1.61	0.52
Umcolcus	553.00	UQ10	1860	615.52		1.51	3.91	1.22	629.43	0.51	178.04	0.00166	3.54	0.31
Umcolcus	553.00	MQ10	2270	616.51		1.50	3.82	1.24	815.10	0.46	194.72	0.001245	4.19	0.28
Umcolcus	553.00	UQ50	2560	617.07		1.52	3.86	1.29	925.38	0.46	202.49	0.001128	4.57	0.27
Umcolcus	553.00	UQ100	2890	617.61		1.47	3.97	1.35	1039.28	0.47	215.55	0.001071	4.82	0.26
Umcolcus	553.00	MQ50	3176	618.11		1.34	4.05	1.39	1153.61	0.48	244.66	0.001016	4.72	0.26
Umcolcus	553.00	MQ100	3590	618.71		1.18	4.20	1.45	1312.03	0.50	277.31	0.00099	4.73	0.26
Umcolcus	553.00	UQ500	3640	618.78		1.19	4.20	1.45	1332.97	0.50	278.64	0.000979	4.78	0.26
Umcolcus	553.00	MQ500	4575	619.34		1.39	4.83	1.67	1491.76	0.65	295.55	0.001186	5.05	0.29
Umcolcus	450.00	MQ1_1	726	611.76		0.39	4.15	1.22	183.71	0.85	131.68	0.008735	1.4	0.58
Umcolcus	450.00	UQ10	1860	615.45		0.97	2.84	0.98	812.96	0.27	203.64	0.000812	3.99	0.22
Umcolcus	450.00	MQ10	2270	616.47		0.98	2.82	1.01	1026.12	0.25	215.96	0.000632	4.75	0.2
Umcolcus	450.00	UQ50	2560	617.03		1.01	2.87	1.05	1149.16	0.25	222.08	0.000586	5.17	0.19
Umcolcus	450.00	UQ100	2890	617.58		1.03	2.95	1.10	1273.03	0.26	227.12	0.00056	5.61	0.19
Umcolcus	450.00	MQ50	3176	618.09		0.99	3.01	1.14	1389.28	0.26	233.63	0.000531	5.95	0.19
Umcolcus	450.00	MQ100	3590	618.69		0.98	3.12	1.19	1533.22	0.27	242.43	0.000518	6.32	0.19
Umcolcus	450.00	UQ500	3640	618.77		0.98	3.13	1.20	1551.44	0.27	243.53	0.000515	6.37	0.19
Umcolcus	450.00	MQ500	4575	619.31		1.12	3.66	1.41	1686.55	0.37	251.78	0.000649	6.7	0.21

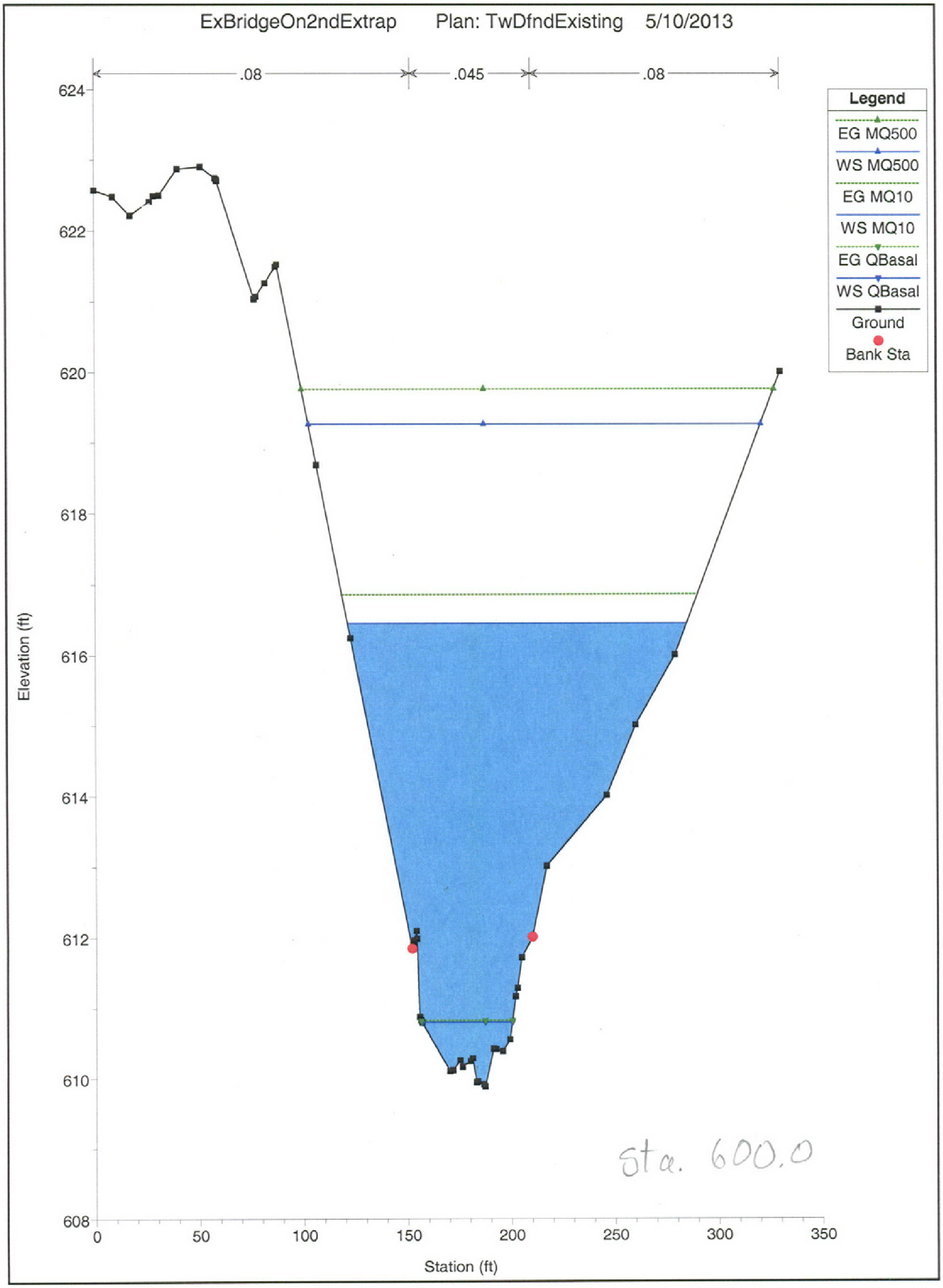
flow Area at bridge 496.36 ft² at Elev. 617.81

for area up to avg. bottom chord add .34'(51.23') = 17.42 ft²   At D.S. fascia Area = 514 ft²   With removal of D.S. block add 86 ft² = 600 ft²

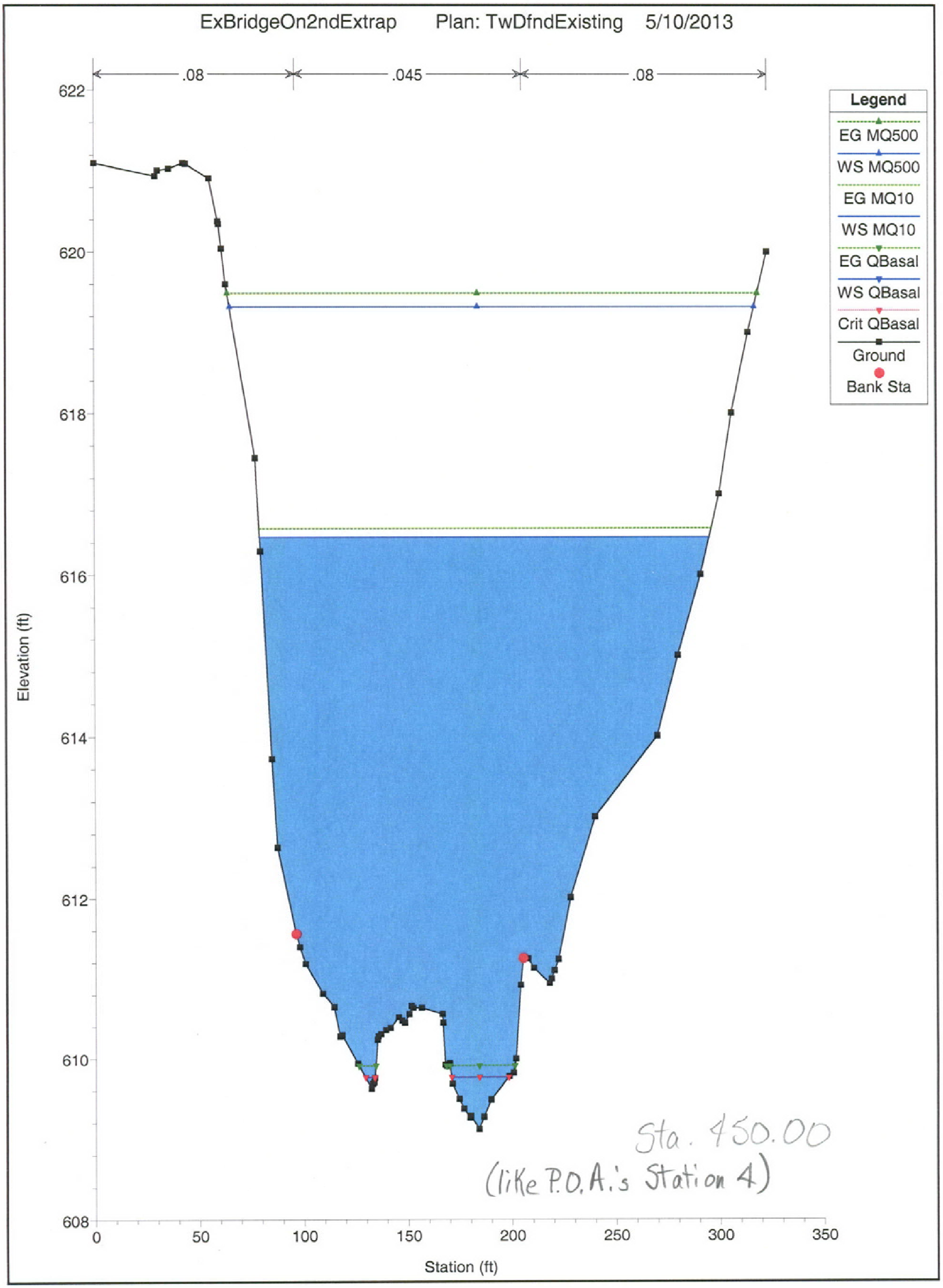
	Reach	River Sta	Profile	Q (cfs)	W.S. Elev. (ft)	BR Vel (ft/s)	Vel Lt (ft/s)	Vel Chn (ft/s)	Vel Rt (ft/s)	Flow Area (sq ft)	Ch Shear (lb/sq ft)	Top Width(ft)	E.G. (ft/ft)	Hydr D (ft)	Froude No.
	Umcolcus	347.77	MQ1_1	726	611.53		0.09	2.75	0.33	264.74	0.29	79.09	0.001345	3.35	0.26
	Umcolcus	347.77	UQ10	1860	615.31		0.70	3.20	0.79	672.88	0.31	141.14	0.00069	4.77	0.21
	Umcolcus	347.77	MQ10	2270	616.33		0.84	3.34	0.81	824.54	0.32	157.56	0.000631	5.23	0.21
End of	Umcolcus	347.77	UQ50	2560	616.88		0.91	3.47	0.84	916.78	0.34	175.95	0.000627	5.21	0.21
Upstream	Umcolcus	347.77	UQ100	2890	617.43		0.97	3.64	0.89	1016.27	0.36	190.07	0.000634	5.35	0.21
Wingwall	Umcolcus	347.77	MQ50	3176	617.92		0.83	3.79	0.94	1117.05	0.39	221.66	0.000643	5.04	0.21
	Umcolcus	347.77	MQ100	3590	618.52		0.94	3.94	1.00	1252.84	0.41	230.46	0.000642	5.44	0.22
	Umcolcus	347.77	UQ500	3640	618.59		0.95	3.96	1.01	1270.16	0.41	232.05	0.000641	5.47	0.22
	Umcolcus	347.77	MQ500	4575	619.08		1.15	4.66	1.21	1389.26	0.56	260.21	0.000838	5.34	0.25
	Umcolcus	330.77	MQ1_1	726	611.49		0.10	2.92	0.05	248.82	0.30	47.82	0.001061	5.2	0.23
	Umcolcus	330.77	UQ10	1860	615.13		0.27	4.40	0.24	424.54	0.56	48.65	0.001192	8.73	0.26
	Umcolcus	330.77	MQ10	2270	616.09		0.31	4.85	0.29	471.02	0.66	48.99	0.001263	9.62	0.27
Upstream	Umcolcus	330.77	UQ50	2560	616.59		0.35	5.20	0.32	495.93	0.75	49.28	0.001358	10.06	0.29
Fascia	Umcolcus	330.77	UQ100	2890	617.08		0.40	5.61	0.35	519.81	0.86	49.63	0.001486	10.47	0.3
	Umcolcus	330.77	MQ50	3176	617.52		0.45	5.92	0.38	542.07	0.94	49.97	0.001567	10.85	0.31
	Umcolcus	330.77	MQ100	3590	618.03		0.51	6.40	0.43	567.76	1.09	50.36	0.001726	11.27	0.33
	Umcolcus	330.77	UQ500	3640	618.1		0.52	6.45	0.44	570.99	1.10	50.41	0.001742	11.33	0.33
	Umcolcus	330.77	MQ500	4575	618.29		0.65	7.98	0.55	580.53	1.68	50.56	0.00261	11.48	0.41
	Umcolcus	317.77	MQ1_1	726	611.49	4.00									
	Umcolcus	317.77	UQ10	1860	615.13	5.23									
℄ ~	Umcolcus	317.77	MQ10	2270	616.09	5.66									
Roadway	Umcolcus	317.77	UQ50	2560	616.59	6.02									
(Bridge)	Umcolcus	317.77	UQ100	2890	617.08	6.45									
	Umcolcus	317.77	MQ50	3176	617.52	6.77									
	Umcolcus	317.77	MQ100	3590	618.03	7.28									
	Umcolcus	317.77	UQ500	3640	618.1	7.33									
	Umcolcus	317.77	MQ500	4575	618.29	9.23									

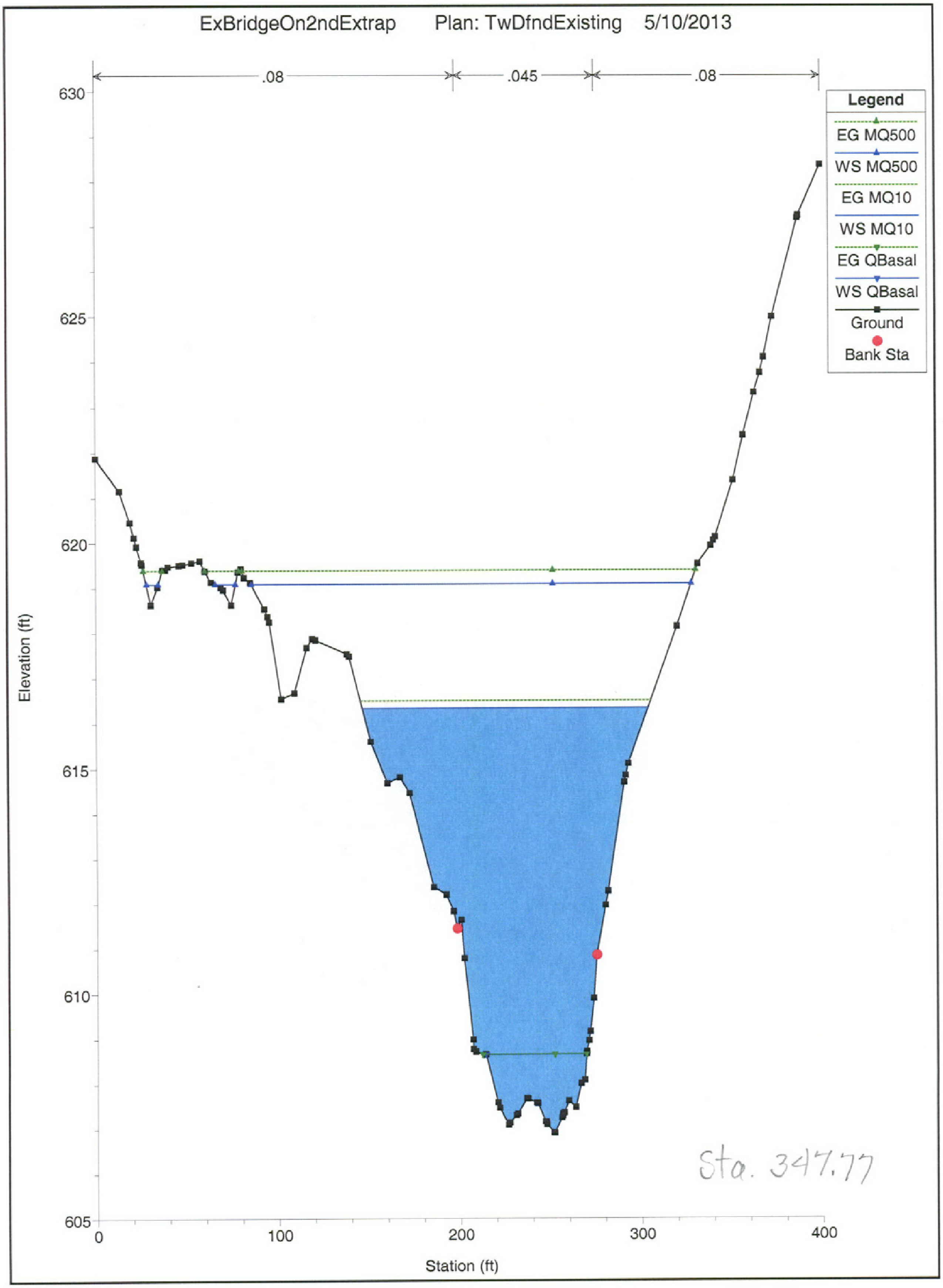
	Reach	River Sta	Profile	Q (cfs)	W.S. Elev. (ft)	BR Vel (ft/s)	Vel Lt (ft/s)	Vel Chn (ft/s)	Vel Rt (ft/s)	Flow Area (sq ft)	Ch Shear (lb/sq ft)	Top Width(ft)	E.G. (ft/ft)	Hydr D (ft)	Froude No.
	Umcolcus	304.77	MQ1_1	726	611.32		0.11	4.01	0.09	181.31	0.61	47.53	0.002875	3.81	0.36
	Umcolcus	304.77	UQ10	1860	614.95		0.34	5.26	0.34	355.60	0.84	48.43	0.002035	7.34	0.34
	Umcolcus	304.77	MQ10	2270	615.89		0.39	5.70	0.39	401.29	0.95	48.84	0.00204	8.22	0.35
Downstream	Umcolcus	304.77	UQ50	2560	616.38		0.44	6.07	0.42	425.26	1.06	49.14	0.002148	8.65	0.36
Fascia	Umcolcus	304.77	UQ100	2890	616.84		0.50	6.52	0.47	447.90	1.20	49.51	0.002314	9.05	0.38
	Umcolcus	304.77	MQ50	3176	617.27		0.56	6.85	0.50	469.30	1.31	49.86	0.002404	9.41	0.39
	Umcolcus	304.77	MQ100	3590	617.75		0.63	7.38	0.57	493.31	1.49	50.25	0.002617	9.82	0.41
	Umcolcus	304.77	UQ500	3640	617.81		0.64	7.43	0.57	496.36	1.51	50.30	0.002638	9.87	0.41
	Umcolcus	304.77	MQ500	4575	617.8		0.81	9.36	0.72	495.60	2.40	50.28	0.004187	9.86	0.51
	Umcolcus	287.77	MQ1_1	726	611.12			4.87	0.12	149.01	0.98	54.87	0.005889	2.72	0.52
End of	Umcolcus	287.77	UQ10	1860	615.07		0.92	4.11	1.12	728.17	0.52	265.78	0.001304	2.74	0.28
	Umcolcus	287.77	MQ10	2270	616.1		1.08	3.85	1.12	1004.67	0.44	272.25	0.000945	3.69	0.25
Downstream	Umcolcus	287.77	UQ50	2560	616.64		1.16	3.84	1.15	1153.52	0.42	275.51	0.000857	4.19	0.24
Wingwall	Umcolcus	287.77	UQ100	2890	617.16		1.25	3.89	1.19	1297.80	0.43	278.58	0.00081	4.66	0.23
	Umcolcus	287.77	MQ50	3176	617.65		1.30	3.90	1.20	1432.71	0.42	281.45	0.000755	5.09	0.23
	Umcolcus	287.77	MQ100	3590	618.2		1.39	3.98	1.25	1591.01	0.43	284.82	0.00073	5.59	0.22
	Umcolcus	287.77	UQ500	3640	618.27		1.40	3.99	1.25	1610.94	0.43	285.24	0.000726	5.65	0.22
	Umcolcus	287.77	MQ500	4575	618.55		1.71	4.79	1.51	1689.85	0.61	286.90	0.001008	5.89	0.27
	Umcolcus	196.08	MQ1_1	726	610.89			2.71		267.77	0.36	159.16	0.00343	1.68	0.37
	Umcolcus	196.08	UQ10	1860	615.15		0.44	1.69	0.52	1461.34	0.09	395.20	0.000248	3.7	0.12
	Umcolcus	196.08	MQ10	2270	616.16		0.51	1.66	0.58	1861.80	0.08	397.00	0.000195	4.69	0.11
	Umcolcus	196.08	UQ50	2560	616.7		0.55	1.69	0.62	2075.97	0.08	397.00	0.000184	5.23	0.11
	Umcolcus	196.08	UQ100	2890	617.22		0.60	1.75	0.66	2282.16	0.09	397.00	0.000179	5.75	0.11
	Umcolcus	196.08	MQ50	3176	617.7		0.63	1.78	0.69	2472.61	0.09	397.00	0.000172	6.23	0.11
	Umcolcus	196.08	MQ100	3590	618.26		0.68	1.85	0.74	2694.64	0.09	397.00	0.000171	6.79	0.11
	Umcolcus	196.08	UQ500	3640	618.33		0.69	1.86	0.75	2722.41	0.10	397.00	0.00017	6.86	0.11
	Umcolcus	196.08	MQ500	4575	618.62		0.84	2.24	0.91	2840.49	0.14	397.00	0.000238	7.15	0.13

Reach	River Sta	Profile	Q (cfs)	W.S. Elev. (ft)	BR Vel (ft/s)	Vel Lt (ft/s)	Vel Chn (ft/s)	Vel Rt (ft/s)	Flow Area (sq ft)	Ch Shear (lb/sq ft)	Top Width(ft)	E.G. (ft/ft)	Hydr D (ft)	Froude No.
Umcolcus	87.00	MQ1_1	726	610.49		0.28	2.54	0.79	368.35	0.34	430.39	0.004036	0.86	0.39
Umcolcus	87.00	UQ10	1860	615.15		0.44	1.00	0.50	2646.76	0.03	501.00	0.000084	5.28	0.07
Umcolcus	87.00	MQ10	2270	616.16		0.47	1.02	0.52	3152.60	0.03	501.00	0.000072	6.29	0.07
Umcolcus	87.00	UQ50	2560	616.7		0.49	1.06	0.54	3423.12	0.03	501.00	0.00007	6.83	0.07
Umcolcus	87.00	UQ100	2890	617.22		0.52	1.11	0.57	3683.56	0.04	501.00	0.00007	7.35	0.07
Umcolcus	87.00	MQ50	3176	617.7		0.54	1.14	0.59	3924.09	0.04	501.00	0.000069	7.83	0.07
Umcolcus	87.00	MQ100	3590	618.26		0.57	1.21	0.63	4204.65	0.04	501.00	0.000071	8.39	0.07
Umcolcus	87.00	UQ500	3640	618.33		0.58	1.21	0.63	4239.73	0.04	501.00	0.000071	8.46	0.07
Umcolcus	87.00	MQ500	4575	618.63		0.70	1.47	0.77	4390.33	0.06	501.00	0.0001	8.76	0.08
Umcolcus	60.00	MQ1_1	726	610.3			3.10	0.75	292.41	0.50	373.02	0.006196	0.78	0.48
Umcolcus	60.00	UQ10	1860	615.15		0.39	0.93	0.48	2647.33	0.03	501.00	0.000077	5.28	0.07
Umcolcus	60.00	MQ10	2270	616.16		0.42	0.95	0.50	3153.31	0.03	501.00	0.000065	6.29	0.06
Umcolcus	60.00	UQ50	2560	616.7		0.44	0.99	0.52	3423.87	0.03	501.00	0.000064	6.83	0.06
Umcolcus	60.00	UQ100	2890	617.22		0.46	1.04	0.55	3684.37	0.03	501.00	0.000064	7.35	0.07
Umcolcus	60.00	MQ50	3176	617.7		0.48	1.07	0.57	3924.87	0.03	501.00	0.000063	7.83	0.07
Umcolcus	60.00	MQ100	3590	618.26		0.51	1.13	0.60	4205.43	0.04	501.00	0.000064	8.39	0.07
Umcolcus	60.00	UQ500	3640	618.33		0.52	1.14	0.60	4240.50	0.04	501.00	0.000064	8.46	0.07
Umcolcus	60.00	MQ500	4575	618.63		0.63	1.38	0.73	4390.80	0.05	501.00	0.000091	8.76	0.08

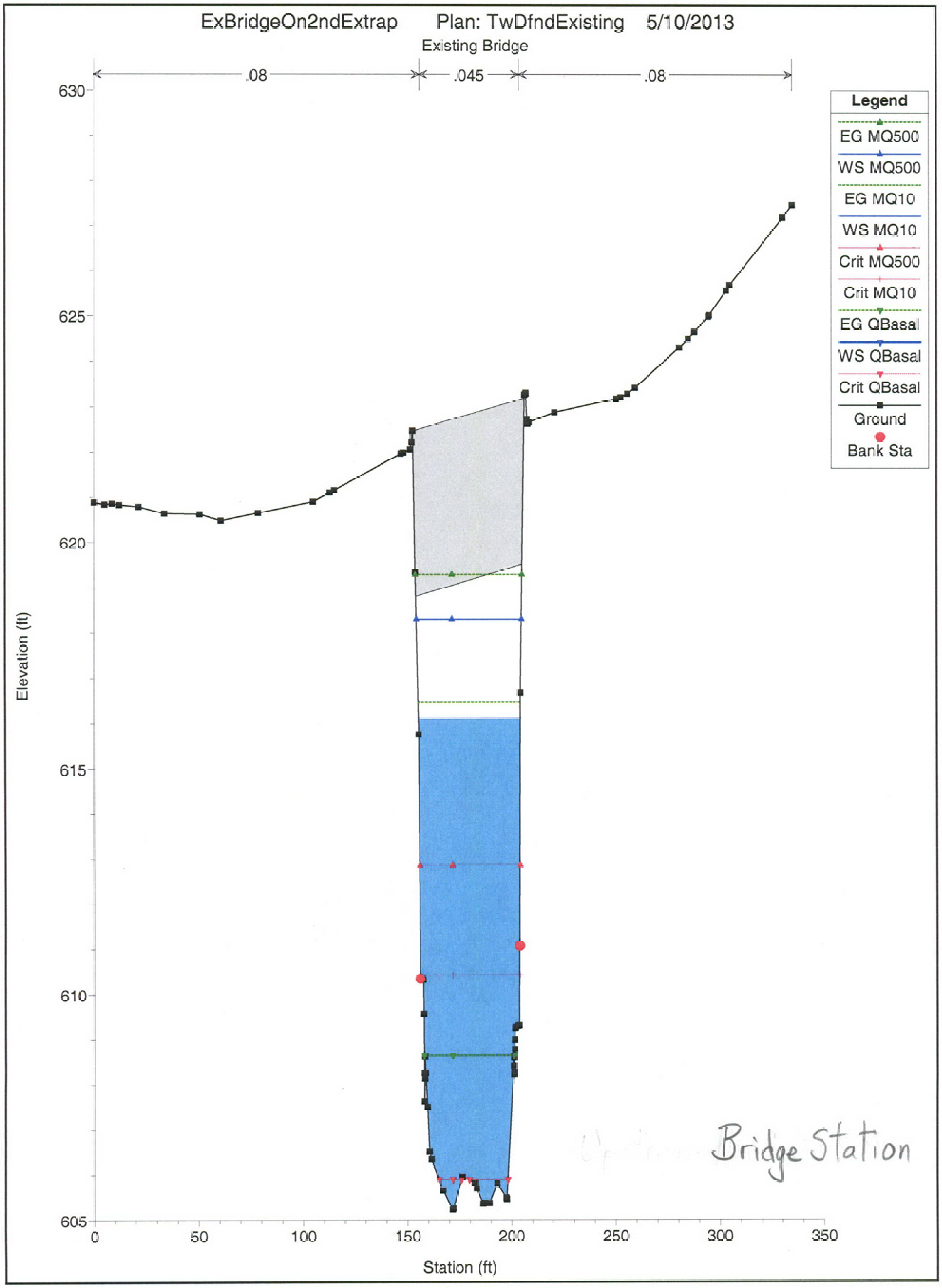


ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
.08
.045
.08
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
EG QBasal
WS QBasal
Ground
Bank Sta
Elevation (ft)
624
622
620
618
616
614
612
610
608
0
50
100
150
200
250
300
350
Station (ft)
Sta. 600.0

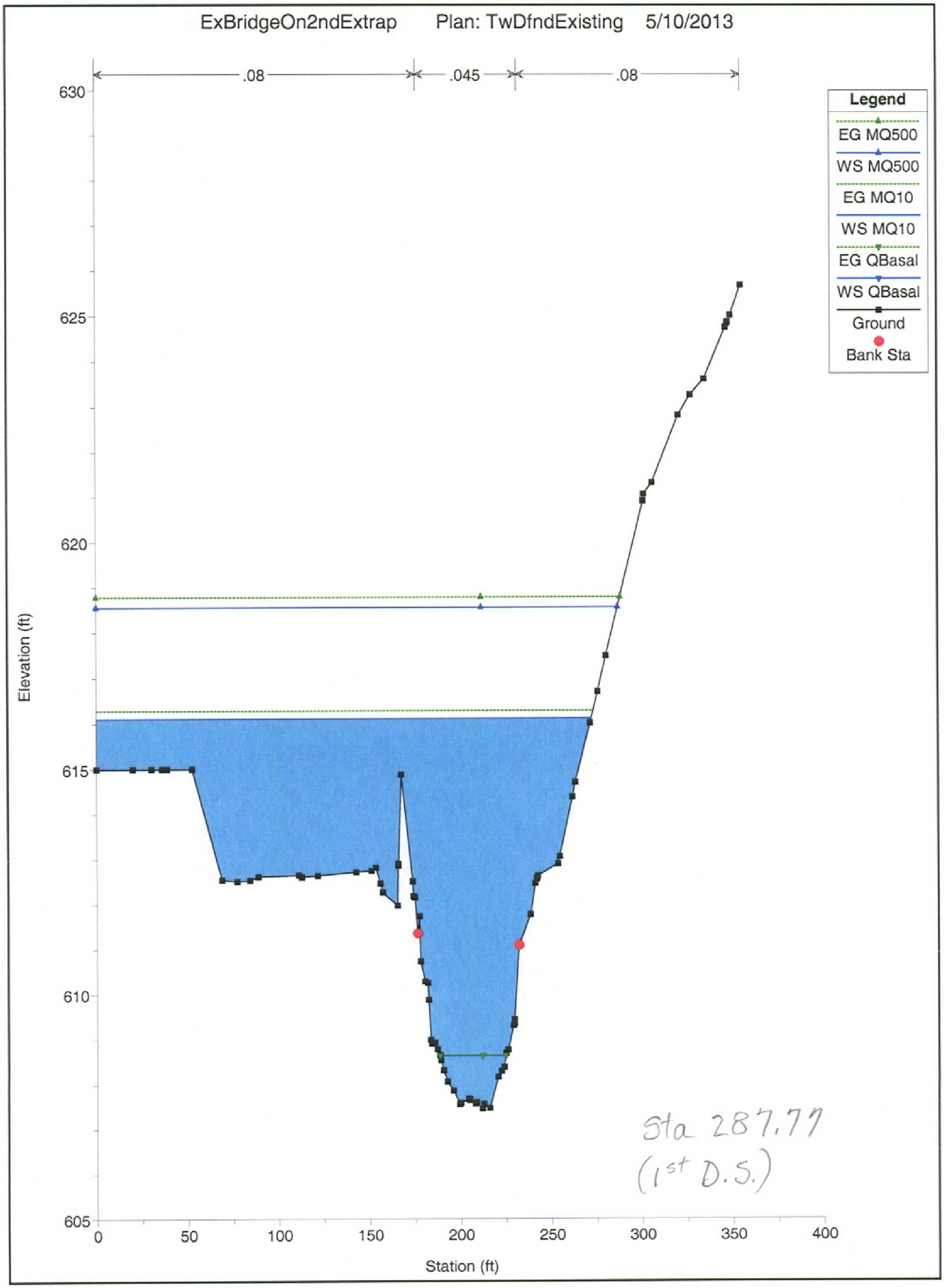

ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
.08
.045
.08
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
EG QBasal
WS QBasal
Crit QBasal
Ground
Bank Sta
Elevation (ft)
622
620
618
616
614
612
610
608
Station (ft)
0
50
100
150
200
250
300
350
Sta. 450.00
(like P.O.A.'s Station 4)



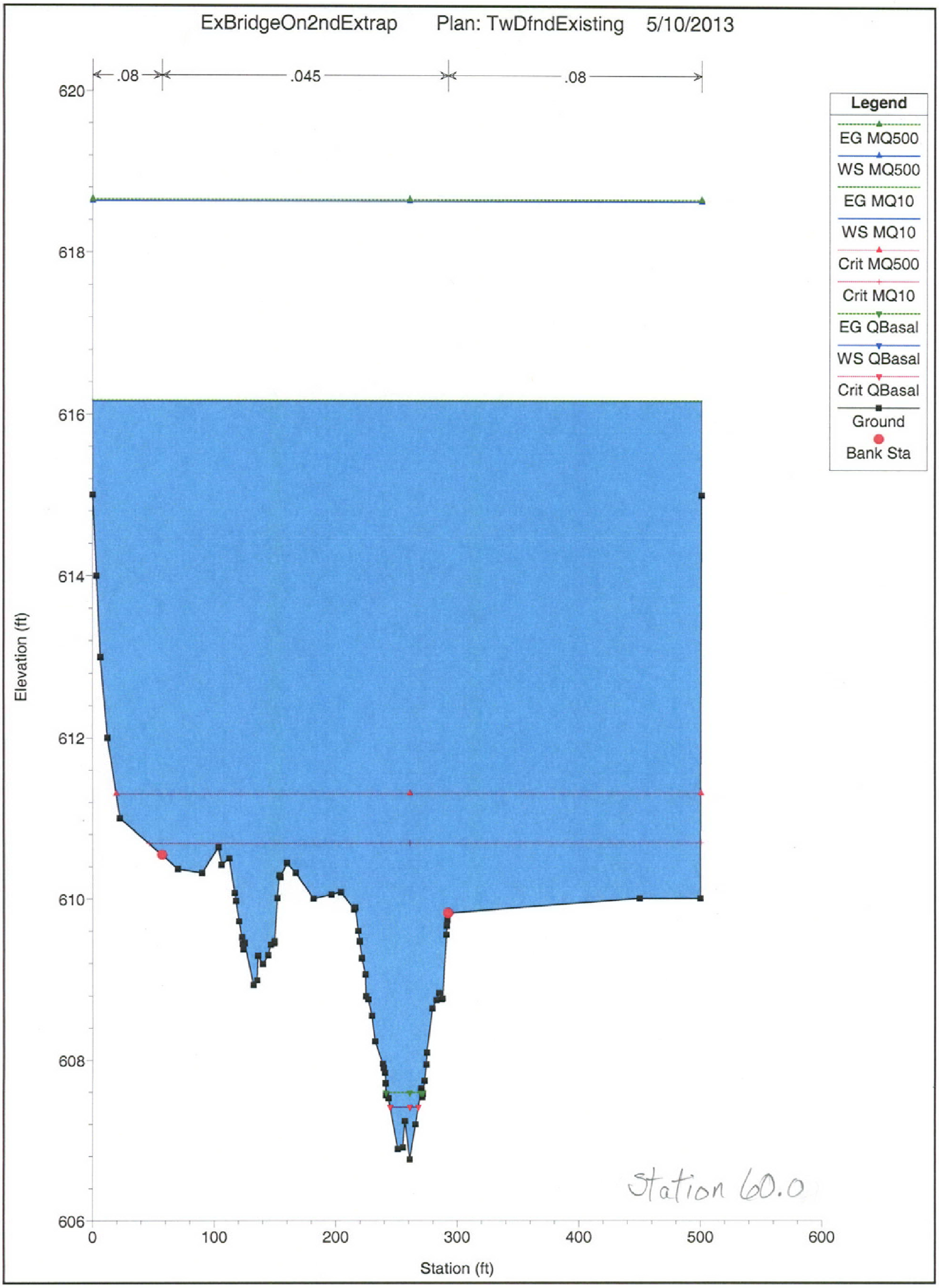
ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
.08
.045
.08
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
EG QBasal
WS QBasal
Ground
Bank Sta
Elevation (ft)
630
625
620
615
610
605
0
100
200
300
400
Station (ft)
Sta. 347.77



ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
Existing Bridge
.08
.045
.08
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
Crit MQ500
Crit MQ10
EG QBasal
WS QBasal
Crit QBasal
Ground
Bank Sta
Elevation (ft)
Station (ft)
630
625
620
615
610
605
0
50
100
150
200
250
300
350
Bridge Station



ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
.08
.045
.08
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
EG QBasal
WS QBasal
Ground
Bank Sta
Elevation (ft)
630
625
620
615
610
605
0
50
100
150
200
250
300
350
400
Station (ft)
Sta 287.77
(1st D.S.)



ExBridgeOn2ndExtrap Plan: TwDfndExisting 5/10/2013
.08
.045
.08
620
618
616
614
612
610
608
606
Elevation (ft)
0
100
200
300
400
500
600
Station (ft)
Legend
EG MQ500
WS MQ500
EG MQ10
WS MQ10
Crit MQ500
Crit MQ10
EG QBasal
WS QBasal
Crit QBasal
Ground
Bank Sta
Station 60.0

SIZING ASSUMPTIONS & TARGETS

In sizing riprap some references and standards are based on weight and some are based on grain size. To evaluate the fit between Maine State specifications and the newer H.E.C. 23 sizing guidance, evaluate the following target values and assumptions.

1) Specific Weight, S_s, assumed 2.65 in HEC 23 ⇒ 165 $\frac{lbs.}{cu.ft.}$
2) d_{85}/d_{15} should range from 1.5 to 2.5, assume 2.5 for flexible grading
3.) from HEC23 V1 Sect. 5.2.8, For the rock shape, say A is the largest dimension, B is the middle, C is the smallest. $A/C \leq 3.0$.
Say $A = 3C$, $B = \sqrt{3}(C)$, for $C = .83'$, $B = 1.44$, $A = 2.5'$ Vol = 3.0 cu.ft.
This is the prototype W_{50} rock, 500 lb. 3.0 cu.ft. × 165 $\frac{lbs.}{cu.ft.}$ ≈ 500 lb.
4.) So, the grain size, d_{50}, ≈ B ≈ 1.44 feet.
5.) This target d_{50} size is between Class IV at 15 inch and Class V at 18 inch in Table 4.1 from HEC23, Vol. 2, which gives riprap recommended gradation values from 2006 NCHRP Rpt. 568
6.) Use the Class V, 18" = d_{50}, criteria to develop d_{15}, d_{85}, & d_{100} sizes.
Also use the relationship where B = 1.0 (grain size) C = .576, A = 1.736

For: $d_{15\,min}$ = 11" Vol. = .77 cu.ft. $W_{15\,min}$ = 127#
$d_{15\,max}$ = 15.5" Vol. = 1.84 cu.ft. $W_{15\,max}$ = 304
$d_{50\,min}$ = 17" Vol. = 2.62 cu.ft $W_{50\,min}$ = 433
$d_{50\,max}$ = 20.5" Vol. = 4.98 cu.ft. $W_{50\,max}$ = 822
$d_{85\,min}$ = 23.5" Vol. = 7.5 cu.ft. $W_{85\,min}$ = 1239 lbs.
$d_{85\,max}$ = 27.5" Vol. = 12.0 cu.ft $W_{85\,max}$ = 1986 lbs.
$d_{100\,max}$ = 36" Vol = 27.0 cu.ft. $W_{100\,max}$ = 4455 lbs.

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM

Scour Protection
Design Computations

Umcolcus Bridge
over
Umcolcus Stream
in
Oxbow Plantation, Maine

Br. # 2877
WIN 017880.00

Design by: R.S. Bulger

DESIGN: Hydraulic Engineering Circular 23, Volume 2, September 2009

ABSTRACT:

For the Designer's ease of understanding, the computations in this file are expanded. This file contain design computations for: 1) grout filled mats; 2) riprap sizing for abutments; 3) partially grouted riprap; and 4) block mat scour protection.

Design procedure for grout filled mats placed on channel beds or banks

Design reference: HEC 23, Volume 2, September 2009, section 9.3.2

Q500 flow shear stress at Upstream fascia cross section

$WS_{Elevation} := 618.29ft$ Water surface elevation at Q 500 flow

$EL_{min\_channel} := 605.5ft$ Minimum channel elevation from HEC RAS output or survey

$y := WS_{Elevation} - EL_{min\_channel} = 12.79 \cdot ft$ Maximum depth of flow on revetment, ft

Note: typically check at upstream end of the bridge for the worst case situation .

$\gamma_w := 62.4 \frac{lb}{ft^3}$ Unit weight of water (constant)

$S_f := 0.002610 \frac{ft}{ft}$ Slope of the energy grade line, ft/ft

Note:The design shear stress is very sensitive to the slope for the energy grade line.

$T_{channel} := 50.56ft$ Top width of the channel, feet

$R_c := 145ft$ Radius of curvature of the channel, ft

Bend Coefficient

The bend coefficient, K_b, is calculated if R_c/T is between 2 and 10. Otherwise it is a set value.

$K_{b1} := 2.0$ for 2 >=Rc/T $\frac{R_c}{T_{channel}} = 2.87$

$$K_{b2} := 2.38 - 0.206\left(\frac{R_c}{T_{channel}}\right) + 0.0073\left(\frac{R_c}{T_{channel}}\right)^2 = 1.849$$

$K_{b3} := 1.05$ for Rc/T>=10 $\frac{R_c}{T_{channel}} = 2.87$

Q500 flow shear stress at Upstream fascia cross section (Cont'd)

$$K_b := \left[\begin{array}{l} K_{b1} \text{ if } \frac{R_c}{T_{channel}} < 2 \\ K_{b3} \text{ if } \left(\frac{R_c}{T_{channel}}\right) > 10 \\ K_{b2} \text{ otherwise} \end{array}\right]$$

So, calculated flow shear stress at Upstream fascia is:

$K_b = 1.849$

$\tau_{des\_usfasc} := K_b \cdot \gamma_w \cdot y \cdot S_f = 3.852 \cdot \frac{lb}{ft^2}$ Design shear stress on mat, lb/ft^2

HEC-RAS value for flow shear stress at Upstream fascia is:

$\tau_{HECRAS\_usfasc} := 1.68 \cdot \frac{lb}{ft^2}$ Design shear stress on mat, lb/ft^2, from HEC RAS bridge output

$$\tau_{usfasc} := \text{if}\left(\tau_{HECRAS\_usfasc} > \tau_{des\_usfasc}, \tau_{HECRAS\_usfasc}, \tau_{des\_usfasc}\right)$$

$$\tau_{usfasc} = 3.852 \cdot \frac{lb}{ft^2}$$

Q500 flow shear stress at End of Upstream Wingwalls

$WS_{Elevation} := 619.08ft$ Water surface elevation at Q 500 flow

$EL_{min\_channel} := 609.2ft$ Minimum channel elevation from HEC RAS output or survey

$y := WS_{Elevation} - EL_{min\_channel} = 9.88 \cdot ft$ Maximum depth of flow on revetment, ft

Note: typically check at upstream end of the bridge for the worst case situation

$S_f := 0.000838 \frac{ft}{ft}$ Slope of the energy grade line, ft/ft

Note:The design shear stress is very sensitive to the slope for the energy grade line.

$T_{channel} := 260ft$ Top width of the channel, feet

Bend Coefficient

The bend coefficient, K_b, is calculated if R_c/T is between 2 and 10. Otherwise it is a set value.

Q500 flow shear stress at End of Upstream Wingwalls (Continued)

$$\frac{R_c}{T_{channel}} = 0.56 \qquad K_{b2} := 2.38 - 0.206\left(\frac{R_c}{T_{channel}}\right) + 0.0073\left(\frac{R_c}{T_{channel}}\right)^2 = 2.267$$

For constant values of K_{b1} and K_{b3}, see above.

$$K_b := \begin{cases} K_{b1} & \text{if } \frac{R_c}{T_{channel}} < 2 \\ K_{b3} & \text{if } \left(\frac{R_c}{T_{channel}}\right) > 10 \\ K_{b2} & \text{otherwise} \end{cases}$$

So, calculated flow shear stress at ends of Upstream wingwalls is:

$K_b = 2$

$\tau_{des\_uswing} := K_b \cdot \gamma_w \cdot y \cdot S_f = 1.033 \cdot \frac{lb}{ft^2}$ Design shear stress on mat, lb/ft^2

HEC-RAS value for flow shear stress at end of Upstream wingwall is:

$\tau_{HECRAS\_uswing} := 0.56 \cdot \frac{lb}{ft^2}$ Design shear stress on mat, lb/ft^2, from HEC RAS bridge output

$$\tau_{uswing} := if(\tau_{HECRAS\_uswing} > \tau_{des\_uswing}, \tau_{HECRAS\_uswing}, \tau_{des\_uswing})$$

$$\tau_{uswing} = 1.033 \cdot \frac{lb}{ft^2}$$

Q500 flow shear stress at Downstream Fascia

$WS_{Elevation} := 617.80ft$ Water surface elevation at Q 500 flow

$EL_{min\_channel} := 607.7ft$ Minimum channel elevation from HEC RAS output or survey

$y := WS_{Elevation} - EL_{min\_channel} = 10.1 \cdot ft$ Maximum depth of flow on revetment, ft

$S_f := 0.004187 \frac{ft}{ft}$ Slope of the energy grade line, ft/ft

Note:The design shear stress is very sensitive to the slope for the energy grade line.

$T_{channel} := 50.3ft$ Top width of the channel, feet

$R_c := 145ft$ Radius of curvature of the channel, ft $\frac{R_c}{T_{channel}} = 2.883$

Q500 flow shear stress at Downstream Fascia (Continued)

Bend Coefficient

The bend coefficient, K_b, is calculated if R_c/T is between 2 and 10. Otherwise it is a set value.

$$K_{b2} := 2.38 - 0.206\left(\frac{R_c}{T_{channel}}\right) + 0.0073\left(\frac{R_c}{T_{channel}}\right)^2 = 1.847$$

For constant values of K_{b1} and K_{b3}, see above.

$$K_b := \begin{cases} K_{b1} & \text{if } \frac{R_c}{T_{channel}} < 2 \\ K_{b3} & \text{if } \left(\frac{R_c}{T_{channel}}\right) > 10 \\ K_{b2} & \text{otherwise} \end{cases}$$

$$K_b = 1.847$$

So, calculated flow shear stress at Downstream fascia is:

$$\tau_{des\_dsfasc} := K_b \cdot \gamma_w \cdot y \cdot S_f = 4.873 \cdot \frac{lb}{ft^2}$$

Design shear stress on mat, lb/ft^2

HEC-RAS value for flow shear stress at Downstream fascia is: $\tau_{HECRAS\_dsfasc} := 2.40 \cdot \frac{lb}{ft^2}$

$$\tau_{dsfasc} := \text{if}\left(\tau_{HECRAS\_dsfasc} > \tau_{des\_dsfasc}, \tau_{HECRAS\_dsfasc}, \tau_{des\_dsfasc}\right)$$

$$\tau_{dsfasc} = 4.873 \cdot \frac{lb}{ft^2}$$

Select Controlling Shear Stress, Size Mat, & Verify Safety Factor

Select worst case shear stress of: us wing end, us fascia, and ds fascia sections

$$\tau_{des} := \max(\tau_{uswing}, \tau_{usfasc}, \tau_{dsfasc}) = 4.873 \cdot \frac{lb}{ft^2}$$

Determine the target factor of safety

See: Hydraulic Engineering Circular 23, Volume 2, September 2009, section 9.3.2, figure 9.4

$SF_B := 1.5$ Base factor of safety

$X_C := 1.9$ Multiplier based on consequences of failure

$X_M := 1.2$ Multiplier based on uncertainty in hydrologic/ hydraulic modeling

$SF_T := SF_B \cdot X_C \cdot X_M = 3.42$ Target safety factor

$\delta_{filter\_fabric} := 25deg$

25 degrees for non cohesive soils
32.5 degrees maximum recommended on cohesive soils
45 degrees on cohesive silts and clay

$\mu := \tan(\delta_{filter\_fabric}) = 0.466$ Coefficient of static friction - dimensionless

List of commercial available thicknesses for grouted mattresses:
6 inch = 0.5 ft
8 inch = 0.67ft
10 inch = 0.83 ft
12 inch = 1.0 ft

Try 8 inch size. $t := 0.67ft$ Thickness of grout mat, ft. FS is directly proportional.

$\gamma_c := 140 \frac{lb}{ft^3}$ Unit weight of grout

$\alpha := 18.435deg$ angle of bed slope, degrees

0.5% = 0.29 degrees
2% grade = 1.14 degrees
10% grade = 5.71 degrees
33.333% grade = 18.435 degrees

$$FS_{actual} := \frac{\mu \cdot t \cdot (\gamma_c - \gamma_w) \cdot \cos(\alpha) - \tau_{des}}{\tau_{des}} = 3.7$$ Equation 9.4 for channel bed

Actual vs. Target Safety Factor

$FS_{actual} = 3.7$ > or = $SF_T = 3.42$

$\text{if}(FS_{actual} \geq SF_T, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

Design Procedure for Sizing Riprap

Design ref.: HEC 23, Volume 2, September 2009, section 14.2. The third paragraph states:

"Where spread footings are placed on erodible soil, the preferred approach is to place the footing below the elevation of total scour. If this is not practicable, a second approach is to place the top of footings below the depth of the sum of contraction scour and long-term degradation and to provide scour countermeasures."

At this site, as with most existing bridges, it is impractical to extend the bottoms of abutments downward below the depth of total scour. Scour countermeasures will be used. It is assumed for Umcolcus Bridge that only local abutment scour needs to be added to the computation of contraction scour and long-term degradation in order to reach a safe estimate of total scour. The P.O.A. for this bridge has an error in the formula for D_{50} because of parenthesis placement. The error was made in their expression, but not their value calculations for Equation 14.1. This is the correct equation to use wherever the Froude number is less than 0.80. It applies to the consultant' analysis as well as the three river sections identified for this broader analysis, i.e. upstream wingwall ends, upstream fascia, and downstream fascia sections. It seems Section 14 implies that the sizing of riprap by either Equation 14.1 or 14.2 (for Froude numbers over 0.80) accounts for both contraction scour and long-term degradation.

Assume: $S_s := 2.65$ common specific gravity of riprap Note: $g = 32.174 \frac{1}{s^2} \cdot ft$

For equation 14.1 and vertical wall abutments: $K := 1.02$

For the Upstream Wingwall Location

$y_{usw} := 9.88 \cdot ft$ $v_{usw} := 4.66 \cdot \frac{ft}{s}$ $y := y_{usw}$ $v := v_{usw}$

$$D_{50usw} := y \cdot \left[\frac{K}{(S_s - 1)}\right] \cdot \left(\frac{v^2}{g \cdot y}\right) = 0.417 \cdot ft$$

For the Upstream Fascia

$y_{usf} := 12.79 \cdot ft$ $v_{usf} := 7.98 \cdot \frac{ft}{s}$ $y := y_{usf}$ $v := v_{usf}$

$$D_{50usf} := y \cdot \left[\frac{K}{(S_s - 1)}\right] \cdot \left(\frac{v^2}{g \cdot y}\right) = 1.224 \cdot ft$$

For the Downstream Fascia

$y_{dsf} := 10.1 \cdot ft$ $v_{dsf} := 9.36 \cdot \frac{ft}{s}$ $y := y_{dsf}$ $v := v_{dsf}$

$$D_{50dsf} := y \cdot \left[\frac{K}{(S_s - 1)}\right] \cdot \left(\frac{v^2}{g \cdot y}\right) = 1.683 \cdot ft$$

CONCLUSION: Say the Design size is 1.75 feet (>= 1.68 feet) and the blanket depth is 3.5 feet, which is greater than 1.5 x 1.75 = 2.625 feet and d_{100} = 3.5 feet.

Specifying and Grading Sized Riprap

Evaluate a Combination of Maine Specifications & Design Guide Data

The Maine Standard Specifications for Riprap are not as detailed as the guide specifications referenced by HEC-23, "Bridge Scour and Stream Instability Countermeasures, Volume 2". The first riprap Design Guideline is "Design Guideline 4 Riprap Revetment". Some sections are *partially revetment specific* including: Section 4.2.2 "Design Guidelines for Revetment Riprap"; Section 4.2.6 Filter Requirements; Section 4.2.7 Edge Treatment and Termination Details and the subsections before Section 4.2.2. Other sections including: Section 4.2.3 Thickness of Riprap; Section 4.2.4 Riprap Shape and Gradation; and Section 4.4 "Field Tests for Riprap Gradation", seem to apply to all riprap scour countermeasure. To support the general use of these subsections, consider that these subsections are based on the general riprap guidelines of the 2006 NCHRP Report 568 "Riprap Design Criteria, Recommended Specifications, and Quality Control", and the subsection subject matter is not repeated or adjusted for the other riprap design guides.

Section 4.2.4 "Riprap Shape and Gradation" states, "Table 4.1 provides recommended gradations for ten standard classes of riprap based on the median particle diameter d_{50} as determined by the dimension of the intermediate ("B") axis. These gradations conform to those recommended in NCHRP Report 568 (Lagasse et al 2006). The proposed gradation criteria are based on a nominal or 'target' d_{50} and a uniformity ratio d_{85}/d_{15} that results in riprap that is well graded. The target uniformity ratio d_{85}/d_{15} is 2.0 and the allowable range is from 1.5 to 2.5."

Table 4.1. Minimum and Maximum Allowable Particle Size in Inches.

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Size	Min	Max	Min	Max	Min	Max	Max
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Note: Particle size d corresponds to the intermediate ("B") axis of the particle.

"Based on Equation 4.5, which assumes the volume of the stone is 85% of a cube, Table 4.2 provides the equivalent particle weights for the same ten classes, using a specific gravity of 2.65 for the particle density."

Specifying and Grading Sized Riprap (Continued)

Evaluate a Combination of Maine Specifications & Design Guide Data

Furthermore, Section 4.4 "Field Tests for Riprap Gradation" provides information on two methods 'designed to determine a size distribution based on a random sampling of individual stones within a matrix." The first method discussed in the design guide is the Wolman Count method. For simplicity, only the Wolman Count method is discussed here. Regarding the Wolman Count method, HEC 23 states: "Material gradations for sand size and small gravel materials are typically determined through a sieve analysis of a bulk sample. The weight of each size class (frequency by weight) retained on each sieve is measured and the total percent of material passing that sieve is plotted versus size (sieve opening). The Wolman (1954) procedure measures frequency by size of a surface material rather than a bulk sample. The intermediate dimension (B axis) is measured for randomly selected particles on the surface.

One field approach for cobble size and larger alluvial materials is to select the particle under one's toe after taking a step with eyes averted to avoid bias in particle selection. Another field approach is to stretch a survey tape over the material and measure each particle located at equal intervals along the tape. The equal interval method is recommended for riprap. The interval should be at least 1 ft for small riprap and increased for larger riprap. The B axis is then measured for one hundred particles. The longer and shorter axes (A and C) can also be measured to determine particle shape. Kellerhals and Bray (1971) provide an analysis that supports the conclusion that a surface sample following the Wolman method is equivalent to a bulk sample sieve analysis. One rule that must be followed is that if a single particle is large enough to fall under two interval points along the tape, then it should be included in the count twice. It is probably better to select an interval large enough that this occurs infrequently.

Once 100 particles have been measured, the frequency curve is developed by counting the number of particles less than or equal to specific sizes. To obtain a reasonably detailed frequency curve, the sizes should increase by (2)1/2. For uniform riprap the sizes may need to increase by (2)1/4 to obtain a detailed frequency curve. The starting size should be small enough to capture the low range of sizes, with 64 mm being adequate for most riprap. This process should be repeated to obtain several samples at the riprap installation."

Design of Cable Mat Concrete Block A.C.B. System

ABSTRACT: A concrete block cable mat system is an articulating concrete block system and therefore will be designed in accordance with Design Guideline 8. These calculations fit the guide's "Application 1 - bank revetment and bed armor". By Section 8.3.1 "Hydraulic Stability Design Procedure", drainage layers between the ACB System and an underlying geotextile filter, whether uniform crushed rock or synthetic drainage net, "can relieve sub-block pressure and has appeared to significantly increase the hyrdraulic stability of ACB systems based on full-scale performance testing." "When evaluating a block system, for which performance testing was conducted with a drainage layer, a drainage layer must also be used in the design."

Design of Cable Mat Concrete Block A.C.B. System (Continued)

ABSTRACT (Continued): The current, applicable Maine Special Provision for "Precast Block Mat" utilizes stainless steel cable connected blocks, with suggested dimensions and a prototype shape. Included in the Special Provision are directions and applicable design parameters that allow manufacturers to design a block mat thtat satisfies the scour protection needs. These design parameters are the governing velocity and shear stress values. A Maine licensed engineer is required to design and stamp the cable mat system. Though manufacturers make shorter blocks, new Maine standard practice indicates that all blocks shall use the second shortest available height, 4.5 inches, as a minimum height. The design values for this project are a maximum velocity of 10 feet per second and a maximum shear stress of 5.0 pounds per square foot. The prototype block have base dimensions of 15.5 inches by 15.5 inches, shaped as a "truncated pyramid". The plan area of the top measures 11.5 inches by 11.5 inches The sides have a small noticeable batter; so say the average plan dimensions for weight are 13.5 inches by 13.5 inches, while keeping the full dimensions for the footprint undergoing the shear stress. Verify the viability of the prototype dimensions for this project.

Target Safety Factor

$SF_B := 1.6$ Abutment $X_C := 1.3$ low end of medium consequence, low volume bridge failure with likely warning, very little chance of loss of life.

$X_M := 1.2$ Hydraulic Model is well determined, good agreement with consultant design values, both HEC-RAS based

$SF_T := SF_B \cdot X_C \cdot X_M = 2.496$

Actual Safety Factor

Parameters & Calculated Hydraulic Forces on Concrete Block Mat

DESCRIPTION: "The safety factor for a single block . . . is defined as the ration of restraining moments to overturning moments." In its most common and useful form, Equation 8.13, the formula for the 'actual safety factor" is harder to recognize. It has as the numerator a ratio of moment arms based on the submerged block weight multiplied by the block's projected area. It has as the denominator two unitless shape components for overturning moments based on the weight force, and one component that is a ratio of drag and lift moments divided by the weight x c.o.g. moment. Based on the prototype concrete blocks, with their dimensions and concrete density, many of the independent vairables are given by the definitions of Table 8.1 of Volume 2 of Hec 23. It seems logical to select the maximum hydraulic gradient considering the above calculations, $S_f = 0.00419$, versus the actual slope of the installation, in this case 0.5 feet over 85 feet, or 0.00588. Use 0.0059.

$\theta_0 := \text{atan}(0.00588) = 0.337 \cdot \text{deg}$ channel slope in degrees

$\theta_1 := \text{atan}(0.3333) = 18.433 \cdot \text{deg}$ side slope in degrees

$\theta := \text{atan}\left(\frac{\tan(\theta_0)}{\tan(\theta_1)}\right) = 1.011 \cdot \text{deg}$ angle between sideslope projection of Ws and vertical in degrees

Design of Cable Mat Concrete Block A.C.B. System (Continued)

Actual Safety Factor

Parameters & Calculated Hydraulic Forces on Concrete Block Mat Continued

Try block based on IECS CC45: $A := 15.5 \cdot in$ $B := 15.5in$ $C := 5.5in$

$Wt := 83.33 \cdot lb$ $l_1 := .431C = 2.37 \cdot in$ $l_3 := C = 5.5 \cdot in$ $v = 9.36 \frac{1}{s} \cdot ft$

Note that l_2 and l_4 are the self-weight righting arms of the base. The example in HEC-23 assumes the concrete block mat is oriented at a 45 degree angle to the streambed. The assumed orientation maximizes the length of the moment arm; it is then the distance from the center to a corner. The IECS computation for the righting moment arm is simply half the width, the shortest distance from the center to an edge. This is correct for this typical cabel mat orientation.

$l_2 := \frac{A}{2} = 7.75 \cdot in$ $l_4 := l_2 = 7.75 \cdot in$

Previous calculations show a very large block is required if the assumed protrusion matches the HEC-23 example, 1/3 the block height. Instead go with the IECS example, a protrusion of 1/4 inch.

$\Delta Z := 0.25 \cdot in$ $b := 2 \cdot l_2 = 15.5 \cdot in$ $C_D := 1.0$ Say the drag coefficient is 1.0

So, the shearing force of the stream on a protruding block (worst case) is:

$$F_D := \frac{C_D}{2} \cdot \left[\Delta Z \cdot b \cdot \gamma_w \cdot \left(v^2\right)\right] = 2.286 \cdot lbf$$

It seems that the above calculation would be very unfamiliar to most MaineDOT bridge designers Therefore, check the force of the column of water that loads the protruding block every second versus the columns weight by using the ratio of the stream's acceleration versus gravity.

$HydAc := \frac{v^2}{2 \cdot 9.36 \cdot ft} = 4.68 \frac{1}{s^2} \cdot ft$ Acceleration in one second from zero to design velocity

$\gamma_w := 62.4 \cdot \frac{lbf}{ft^3}$ $W_{H2O} := 9.36 \cdot ft \cdot b \cdot \Delta Z \cdot \gamma_w = 15.717 \cdot lbf$ Weight of the volume of water creating drag on the block protrusion.

$W_{H2O} \cdot \frac{HydAc}{g} = 2.286 \cdot lbf$ drag force based on ratios of stream acceleration versus gravity acceleration of same volume

$$\text{if}\left[\left|\left[\frac{W_{H2O} \cdot \left(\frac{HydAc}{g}\right) - F_D}{F_D}\right]\right| < 0.002, \text{"Check"}, \text{"N.G."}\right] = \text{"Check"}$$

This tight accuracy check assures the calculation of drag force is accurate.

Design of Cable Mat Concrete Block A.C.B. System (Continued)

Actual Safety Factor

Now calculate intermediate values per page DG8.12 of HEC-18 Volume 2 and the IECS spreadsheet.

$a_0 := \left(\cos(\theta_1)^2 - \sin(\theta_0)^2\right)^{.5} = 0.949$ CrtStress $:= 13.2 \cdot \frac{lb}{ft^2}$ $\tau_{des} = 4.873 \cdot \frac{lb}{ft^2}$

$StabNo := \frac{\tau_{des}}{CrtStress} = 0.3692$ $T_1 := \sin(\theta + \theta_0) = 0.0235$ $T_2 := StabNo \cdot \left(\frac{l_2}{l_1}\right) = 1.207$

$T_3 := \left(\frac{l_4}{l_3}\right) + 1 = 2.409$ $T_4 := \left(1 - a_0^{\,2}\right)^{0.5} = 0.316$ $T_5 := \cos(\theta + \theta_0) = 0.99972$

$\beta := atan\left[\frac{T_5}{\left[\left(T_3 \cdot \frac{T_4}{T_2}\right) + T_1\right]}\right] = 0.991$ $StabNoSloped := \left[\frac{\left[\left(\frac{l_4}{l_3}\right) + \left(\sin(\theta_0 + \theta + \beta)\right)\right]}{\left(\frac{l_4}{l_3}\right) + 1}\right] \cdot StabNo = 0.346$

$Wt = 83.33\,lb$ $SG_{conc} := 2.4$ $\gamma_{ws} := 1.94 \cdot \frac{slug}{ft^3}$ $\frac{\gamma_w}{g} = 62.4 \cdot \frac{lb}{ft^3}$

$F'_D := .5 \cdot \Delta Z \cdot b \cdot \gamma_{ws} \cdot v^2 = 2.287\,lbf$

This calculation is based on 1/3 the block height whereas the IECS calculation is based on 1/4", a factor of 7.33

$\Delta FdBm := 90 \cdot deg - \beta - \theta = 32.21 \cdot deg$ $Wt_{sbm} := Wt \cdot \frac{\left(SG_{conc} - 1\right)}{SG_{conc}} = 48.609\,lb$

$Te_1 := \frac{l_2}{l_1} \cdot a_0 = 3.102$ $Te_2 := \left(\frac{l_2}{l_1}\right) \cdot StabNoSloped = 1.131$

$Te_3 := (\cos(\beta)) \cdot \left(1 - a_0^{\,2}\right)^{0.5} = 0.173$ $Te_4 := \frac{\left[\left[(\cos(\Delta FdBm)) \cdot F'_D \cdot l_3\right] + \left(l_4 \cdot F'_D\right)\right]}{\left(l_1 \cdot Wt_{sbm}\right) \cdot \frac{lbf}{lb}} = 0.246$

$SFnoPtrsn := \frac{Te_1}{\left(Te_2 + Te_3\right)} = 2.377$ $SF_{Ptrsn} := \frac{Te_1}{\left(Te_2 + Te_3 + Te_4\right)} = 2.000$

NO GOOD! Try CC70, 8.5" tall, and try protrusion similar to IECS not HEC-23 example.

Design of Cable Mat Concrete Block A.C.B. System (Continued)

Actual Safety Factor

Parameters & Calculated Hydraulic Forces on Concrete Block Mat Continued

Try block based on IECS CC70: $A := 15.5 \cdot in$ $B := 15.5in$ $C := 8.5in$

$Wt := 129.77 \cdot lb$ $l_1 := .431C = 3.663 \cdot in$ $l_3 := C = 8.5 \cdot in$ $v = 9.36 \frac{1}{s} \cdot ft$

Note that l_2 and l_4 are the self-weight righting arms of the base. The example in HEC-23 assumes the concrete block mat is oriented at a 45 degree angle to the streambed. The assumed orientation maximizes the length of the moment arm; it is then the distance from the center to a corner. The IECS computation for the righting moment arm is simply half the width, the shortest distance from the center to an edge. This is correct for this typical cabel mat orientation.

$l_2 := \frac{A}{2} = 7.75 \cdot in$ $l_4 := l_2 = 7.75 \cdot in$

Like the IECS template, only use 1/4 inch for the protrusion, reasonable: $\Delta Z := 0.25 \cdot in$

So, the shearing force of the stream on a protruding block (worst case) is:

$\gamma_w := 62.4 \cdot \frac{lb}{ft^3}$ $F'_D := \frac{C_D}{2} \cdot \left[\Delta Z \cdot b \cdot \gamma_w \cdot \left(v^2\right)\right] = 2.286 lbf$

It seems that the above calculation would be very unfamiliar to most MaineDOT bridge designers. Therefore, check the force of the column of water that loads the protruding block every second versus the columns weight by using the ratio of the stream's acceleration versus gravity.

$HydAc := \frac{v^2}{2 \cdot 9.36 \cdot ft} = 4.68 \frac{1}{s^2} \cdot ft$ Acceleration in one second from zero to design velocity

$\gamma_w := 62.4 \cdot \frac{lbf}{ft^3}$ $W_{H2O} := 9.36 \cdot ft \cdot b \cdot \Delta Z \cdot \gamma_w = 15.717 \cdot lbf$ Weight of the volume of water creating drag on the block protrusion.

$W_{H2O} \cdot \frac{HydAc}{g} = 2.286 \cdot lbf$ drag force based on ratios of stream acceleration versus gravity acceleration of same volume

$$\text{if}\left[\left|\left[\frac{W_{H2O} \cdot \left(\frac{HydAc}{g}\right) - F'_D}{F'_D}\right]\right| < 0.002, \text{"Check"}, \text{"N.G."}\right] = \text{"Check"}$$

This tight accuracy check assures the calculation of drag force is accurate.

Design of Cable Mat Concrete Block A.C.B. System (Continued)

Actual Safety Factor

Now calculate intermediate values per page DG8.12 of HEC-18 Volume 2 and the IECS spreadsheet.

$a_0 := \left(\cos(\theta_1)^2 - \sin(\theta_0)^2\right)^{.5} = 0.949$ $\quad CrtStress := 20.7\cdot\frac{lb}{ft^2}$ $\quad \tau_{des} = 4.873\cdot\frac{lb}{ft^2}$

$StabNo := \frac{\tau_{des}}{CrtStress} = 0.2354$ $\quad T_1 := \sin(\theta + \theta_0) = 0.0235$ $\quad T_2 := StabNo\cdot\left(\frac{l_2}{l_1}\right) = 0.498$

$T_3 := \left(\frac{l_4}{l_3}\right) + 1 = 1.912$ $\quad T_4 := \left(1 - a_0^2\right)^{0.5} = 0.316$ $\quad T_5 := \cos(\theta + \theta_0) = 0.99972$

$\beta := \operatorname{atan}\left[\frac{T_5}{\left[\left(T_3\cdot\frac{T_4}{T_2}\right) + T_1\right]}\right] = 0.68$ $\quad StabNoSloped := \left[\frac{\left[\left(\frac{l_4}{l_3}\right) + \left(\sin(\theta_0 + \theta + \beta)\right)\right]}{\left(\frac{l_4}{l_3}\right) + 1}\right]\cdot StabNo = 0.192$

$Wt = 129.77\,lb$ $\quad SG_{conc} := 2.4$ $\quad \gamma_{ws} := 1.94\cdot\frac{slug}{ft^3}$ $\quad \frac{\gamma_w}{g} = 62.4\cdot\frac{lb}{ft^3}$

$F'_D := .5\cdot\Delta Z\cdot b\cdot\gamma_{ws}\cdot v^2 = 2.287\,lbf$

This calculation is based on the IECS calculation, a protrusion of 1/4".

$\Delta FdBm := 90\cdot deg - \beta - \theta = 50.055\cdot deg$ $\quad Wt_{sbm} := Wt\cdot\frac{(SG_{conc} - 1)}{SG_{conc}} = 75.699\,lb$

$Te_1 := \frac{l_2}{l_1}\cdot a_0 = 2.007$ $\quad Te_2 := \left(\frac{l_2}{l_1}\right)\cdot StabNoSloped = 0.406$

$Te_3 := (\cos(\beta))\cdot\left(1 - a_0^2\right)^{0.5} = 0.246$ $\quad Te_4 := \frac{\left[\left[(\cos(\Delta FdBm))\cdot F'_D\cdot l_3\right] + \left(l_4\cdot F'_D\right)\right]}{\left(l_1\cdot Wt_{sbm}\right)\cdot\frac{lbf}{lb}} = 0.109$

$SF_{noPtrsn} := \frac{Te_1}{(Te_2 + Te_3)} = 3.078$ $\quad SF_{Ptrsn} := \frac{Te_1}{(Te_2 + Te_3 + Te_4)} = 2.638$

$\text{if}\left(SF_{Ptrsn} > SF_T, \text{"Check"}, \text{"N.G."}\right) = \text{"Check"}$ $\quad \text{if}\left(SF_{noPtrsn} > SF_T, \text{"Check"}, \text{"N.G."}\right) = \text{"Check"}$

The CC70, 8.5 inches tall, works in the basic loading and when a 1/4 inch protrusion is assumed.

IMPERIAL UNITS

Block Type / Cell Type	Bed Slope	CC20 Closed	CC35 Closed	CC45 Closed	CC70 Closed
L1 (inches)		1.13	2.03	2.48	3.83
L2 and L4 (inches)		7.75	7.75	7.75	7.75
L3 (inches)		2.5	4.5	5.5	8.5
Block Loading (pounds per square foot)		21.01	37.82	46.22	71.43
Block submerged weight (pounds)		40.89	65.78	83.33	129.77

ALLOWABLE SHEAR STRESSES (lb./sq. ft.) (Factor of Safety = 1.0)

	Bed Slope	CC20 Closed	CC35 Closed	CC45 Closed	CC70 Closed
Level Bed Critical Shear Stress - Over-Turning - No Cable Interaction	0.000	24.7	24.6	25.3	27.1
Level Bed Critical Shear Stress - Sliding - With Cable Interaction	0.000	10.1	18.1	22.1	34.2
Level Bed Critical Shear Stress - Sliding - No Cable Interaction	0.000	6.0	10.7	13.2	20.7
Bed Slope 1.5H:1V (34 degrees) - Over-Turning - No Cable Interaction	0.667	17.2	15.4	17.0	15.5
Bed Slope 1.5H:1V (34 degrees) - Sliding - With Cable Interaction	0.667	1.8	3.3	4.0	6.2
Bed Slope 1.5H:1V (34 degrees) - Sliding - No Cable Interaction	0.667	1.1	2.0	2.4	3.7
Bed Slope 2H:1V (26.5 degrees) - Over-Turning - No Cable Interaction	0.500	20.0	18.5	19.5	18.8
Bed Slope 2H:1V (26.5 degrees) - Sliding - With Cable Interaction	0.500	3.7	6.7	8.2	12.7
Bed Slope 2H:1V (26.5 degrees) - Sliding - No Cable Interaction	0.500	2.2	4.0	4.8	7.5
Bed Slope 3H:1V (18.5 degrees) - Over-Turning - No Cable Interaction	0.333	22.2	21.2	22.0	22.1
Bed Slope 3H:1V (18.5 degrees) - Sliding - With Cable Interaction	0.333	5.8	10.4	12.7	19.6
Bed Slope 3H:1V (18.5 degrees) - Sliding - No Cable Interaction	0.333	3.4	6.2	7.5	11.6
Slope 4H:1V (14 degrees) - Over-Turning - No Cable Interaction	0.250	23.1	22.3	23.2	23.7
Slope 4H:1V (14 degrees) - Sliding - With Cable Interaction	0.250	6.9	12.4	15.2	23.4
Bed Slope 4H:1V (14 degrees) - Sliding - No Cable Interaction	0.250	4.1	7.3	9.0	13.9
Bed Slope 5H:1V (11.3 degrees) - Over-Turning - No Cable Interaction	0.200	23.5	22.9	23.8	24.6
Bed Slope 5H:1V (11.3 degrees) - Sliding - With Cable Interaction	0.200	7.5	13.5	16.5	25.5
Bed Slope 5H:1V (11.3 degrees) - Sliding - No Cable Interaction	0.200	4.4	8.0	9.8	15.1

MAXIMUM ALLOWABLE VELOCITIES (ft./sec.)

Note: Velocities above 25.0 feet per second are not recommended.

	Bed Slope	CC20 Closed	CC35 Closed	CC45 Closed	CC70 Closed
Level Bed Maximum Velocity - Over-Turning - No Cable Interaction	0.000	26.0	25.9	26.3	27.2
Level Bed Maximum Velocity - Sliding - With Cable Interaction	0.000	17.4	23.4	25.9	32.2
Level Bed Maximum Velocity - Sliding - No Cable Interaction	0.000	13.5	18.0	20.0	25.0
Bed Slope 1.5H:1V (34 degrees) - Over-Turning - No Cable Interaction	0.667	21.7	20.5	21.6	20.6
Bed Slope 1.5H:1V (34 degrees) - Sliding - With Cable Interaction	0.667	7.5	10.0	11.1	13.7
Bed Slope 1.5H:1V (34 degrees) - Sliding - No Cable Interaction	0.667	5.7	7.7	8.5	10.6
Bed Slope 2H:1V (26.5 degrees) - Over-Turning - No Cable Interaction	0.500	23.4	22.5	23.1	22.6
Bed Slope 2H:1V (26.5 degrees) - Sliding - With Cable Interaction	0.500	10.6	14.2	15.7	19.5
Bed Slope 2H:1V (26.5 degrees) - Sliding - No Cable Interaction	0.500	8.1	10.9	12.1	15.0
Bed Slope 3H:1V (18.5 degrees) - Over-Turning - No Cable Interaction	0.333	24.6	24.1	24.5	24.6
Bed Slope 3H:1V (18.5 degrees) - Sliding - With Cable Interaction	0.333	13.3	17.8	19.7	24.5
Bed Slope 3H:1V (18.5 degrees) - Sliding - No Cable Interaction	0.333	10.2	13.7	15.1	18.8
Bed Slope 4H:1V (14 degrees) - Over-Turning - No Cable Interaction	0.250	25.1	24.7	25.2	25.5
Bed Slope 4H:1V (14 degrees) - Sliding - With Cable Interaction	0.250	14.4	19.3	21.3	26.5
Bed Slope 4H:1V (14 degrees) - Sliding - No Cable Interaction	0.250	11.1	14.8	16.4	20.4
Slope 5H:1V (11.3 degrees) - Over-Turning - No Cable Interaction	0.200	25.4	25.0	25.5	25.9
Bed Slope 5H:1V (11.3 degrees) - Sliding - With Cable Interaction	0.200	15.1	20.2	22.3	27.8
Bed Slope 5H:1V (11.3 degrees) - Sliding - No Cable Interaction	0.200	11.6	15.5	17.2	21.4

Testing Authority -- Colorado State University, University of Minnesota, University of Windsor

Failure Mode -- Loss of intimate contact.

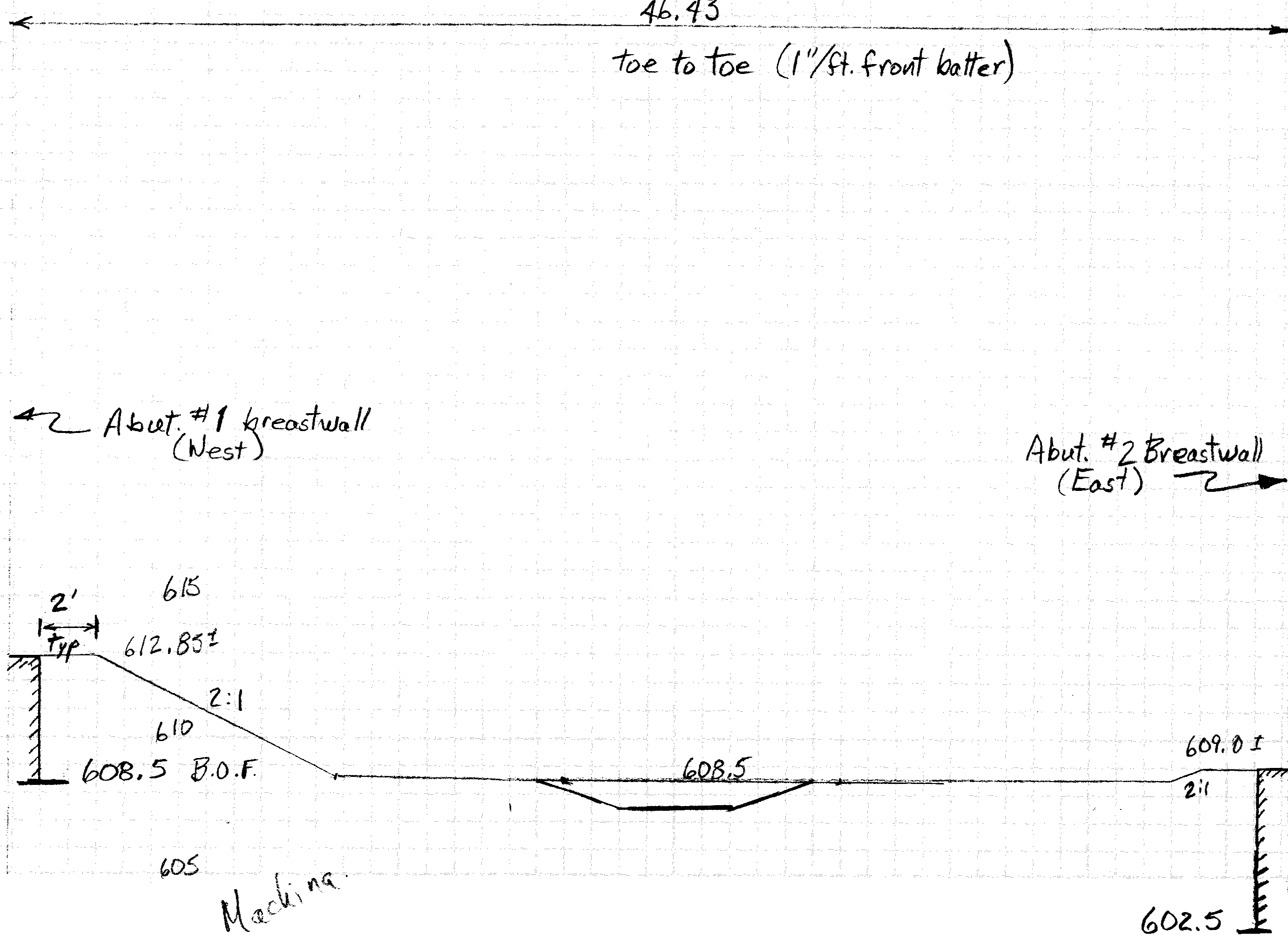
PRELIM. BY R.S. Bulger DATE 7/15/13 PROJ. NO. 17880.00 FILE NO. ____ OF ____

FINAL CHK. BY ____ DATE ____ LOCATION Oxbow Plt, Umcolcus SH. NO. 1 OF 1

ITEM NO. ____ SUBJECT Cross-Section of Channel Along ℄ Construction

℄ Brg. to Br. Seat — Both Batters

Horizontal Distance Between Breastwalls: $50.50' - 2(1.375)' - \left(\frac{15.83}{12}\right)' = 46.43'$



Machina

KNOX SUMMARY OF PRELIMINARY DESIGN

Alternatives for scour countermeasures:

The contractor must use Concrete Cable Mat System for scour countermeasure.

1. Concrete Cable Mat Systems

Cable Mat System – Interconnected block mats are typically installed with a crane using a special spreader beam. The individual mats to be tied together. Edges along abutments or piers need to be sealed to prevent possible loss of fines. The upstream and downstream edges are toed in at least 2 feet.

**See the attached Special Provision for the Concrete Cable Mat System.

Heavy Rip Rap - Item 610.16 shall meet the requirements of 703.28 Heavy Rip Rap, page 7-21. This item will be paid per 610.06 basis of payment which is located in page 6-38 and 6-39. **This item is not an option for the scour protection** but shall be used for the rip rap required behind the wingwalls for erosion control. The reason this item is not to be used for scour protection is because after looking at Heavy Rip Rap and the requirements for installation which requires 3 feet of excavation plus 1 additional foot of excavation for the filter layer puts the total depth to 4 feet which needs to be excavated. The footers are only 3 feet thick and some of them are exposed so this creates a serious concern for undermining by excavating beyond the bottom of the footer. 4' minimum stone layer with 1' thick filter layer. Stone thickness needs to be increased 50% in locations with deep water. Requires extensive excavation and associated high disposal costs for dredge spoils.

NOTE: ***For Knox WIN 19921.00 Heavy Rip Rap (610.16) and Partially Grouted Rip Rap (610.522) were looked at as an option but due to the elevation of the stream bed and the footings, combined with exposed footing, both these options would cause potential issues of undermining for installation of either system. Because of these concerns, the only option acceptable for scour protection on this project is the Concrete Cable Mat System.***

WELD BOWLEY BROOK SUMMARY OF PRELIMINARY DESIGN

Alternatives for scour countermeasures:

The contractor has the choice from 1 of 2 options for scour countermeasures which are listed as follows:

1. Partially Grouted Rip Rap
2. Concrete Cable Mat Systems

Please note the following:

- Use grout bags (item 502.248 Underwater Grout Bags) to repair minor undermining under the granite portion of the north abutment.
- Grout large gaps / voids between granite blocks at the bottom of the north abutment.

The extra grout required to fill gaps / voids between granite blocks will be incidental to item 502.248 Underwater Grout Bags

Partially Grouted Rip Rap - Item 610.522 (9,12 and 15 inch). See the attached Special Provisions for Partially Grouted Rip Rap.

Cable Mat System –See the attached Special Provision for the Concrete Cable Mat System.

Heavy Rip Rap -Item 610.16 shall meet the requirements of 703.28 Heavy Rip Rap, page 7-21. This item will be paid per 610.06 basis of payment which is located in page 6-38 and 6-39. **This**

item is not an option for the scour protection but shall be used for the rip rap required behind the wingwalls for erosion control. The reason this item is not to be used for scour protection is because after looking at Heavy Rip Rap and the requirements for installation which requires 3 feet of excavation plus 1 additional foot of excavation for the filter layer puts the total depth to 4 feet which needs to be excavated. The footers are only 3 feet thick and some of them are exposed so this creates a serious concern for undermining by excavating beyond the bottom of the footer. **NOTE:** ***For Weld WIN 19924.00 Heavy Rip Rap (610.16) was looked at as an option but due to possibility of severe undermining do to depth required for excavation to install, this option is not feasible. Because of this issue, the only options acceptable for scour protection on this project is the Concrete Cable Mat System or the Partially Grouted Rip Rap (PGR).***

WELD HOUGHTON BRIDGE SUMMARY OF PRELIMINARY DESIGN

Alternatives for scour countermeasures:

The contractor has the choice from 1 of 2 options for scour countermeasures which are listed as follows:

1. Partially Grouted Rip Rap
2. Concrete Cable Mat Systems

Partially Grouted Rip Rap - Item 610.522 (9,12 and 15 inch). 1.5' minimum thick stone layer with 6" min. thick filter layer. PGR needs to be installed in the dry. The exact size of stone (9", 12" or 15') to be specified shall be determined in final design based on hydraulic analysis. The final stone layer thickness is two times the D_{50} size stone selected. Upstream and downstream edges are keyed in at twice the depth of the stone.

**See the attached Special Provisions for Partially Grouted Rip Rap.

Cable Mat System – Interconnected block mats are typically installed with a crane using a special
spreader beam. The individual mats to tied together. Edges along abutments or piers need to be sealed to prevent possible loss of fines. The upstream and downstream edges are toed in at least 2 feet.

**See the attached Special Provision for the Concrete Cable Mat System.

Heavy Rip Rap -Item 610.16 shall meet the requirements of 703.28 Heavy Rip Rap, page 7-21. This item will be paid per 610.06 basis of payment which is located in page 6-38 and 6-39. **This item is not an option for the scour protection** but shall be used for the rip rap required behind the wingwalls for erosion control. The reason this item is not to be used for scour protection is because after looking at Heavy Rip Rap and the requirements for installation which requires 3 feet of excavation plus 1 additional foot of excavation for the filter layer puts the total depth to 4 feet which needs to be excavated. The footers are only 3 feet thick and some of them are exposed so this creates a serious concern for undermining by excavating beyond the bottom of the footer. 4' minimum stone layer with 1' thick filter layer. Stone thickness needs to be increased 50% in locations with deep water. Requires extensive excavation and associated high disposal costs for dredge spoils.

NOTE: ***For Weld WIN 19925.00 Heavy Rip Rap (610.16) was looked at as an option but due to possibility of severe undermining do to depth required for excavation to install, this option is not feasible. Because of this issue, the only options acceptable for scour protection on this***

SUMMARY OF PRELIMINARY DESIGN

Alternatives for scour countermeasures:

The contractor has the choice from 1 of 2 options for scour countermeasures which are listed as follows:

1. Partially Grouted Rip Rap
2. Concrete Cable Mat Systems

Partially Grouted Rip Rap - Item 610.522 (9,12 and 15 inch). 1.5' minimum thick stone layer with 6" min. thick filter layer. PGR needs to be installed in the dry. The exact size of stone (9", 12" or 15') to be specified shall be determined in final design based on hydraulic analysis. The final stone layer thickness is two times the D $_{50}$ size stone selected. Upstream and downstream edges are keyed in at twice the depth of the stone.

**See the attached Special Provisions for Partially Grouted Rip Rap.

Cable Mat System – Interconnected block mats are typically installed with a crane using a special
spreader beam. The individual mats to tied together. Edges along abutments or piers need to be sealed to prevent possible loss of fines. The upstream and downstream edges are toed in at least 2 feet.

**See the attached Special Provision for the Concrete Cable Mat System.

Heavy Rip Rap -Item 610.16 shall meet the requirements of 703.28 Heavy Rip Rap, page 7-21. This item will be paid per 610.06 basis of payment which is located in page 6-38 and 6-39. **This item is not an option for the scour protection** but shall be used for the rip rap required behind the wingwalls for erosion control. The reason this item is not to be used for scour protection is because after looking at Heavy Rip Rap and the requirements for installation which requires 3 feet of excavation plus 1 additional foot of excavation for the filter layer puts the total depth to 4 feet which needs to be excavated. The footers are only 3 feet thick and some of them are exposed so this creates a serious concern for undermining by excavating beyond the bottom of the footer. 4' minimum stone layer with 1' thick filter layer. Stone thickness needs to be increased 50% in locations with deep water. Requires extensive excavation and associated high disposal costs for dredge spoils.

NOTE: ***For Weld WIN 19925.00 Heavy Rip Rap (610.16) was looked at as an option but due to possibility of severe undermining do to depth required for excavation to install, this option is not feasible. Because of this issue, the only options acceptable for scour protection on this***

Block Selection Guide ver. 3.0

IMPERIAL UNITS

Block Type / Cell Type	Bed Slope	CC20 Closed	CC35 Closed	CC45 Closed	CC70 Closed
L1 (inches)		1.13	2.03	2.48	3.83
L2 and L4 (inches)		7.75	7.75	7.75	7.75
L3 (inches)		2.5	4.5	5.5	8.5
Block Loading (pounds per square foot)		21.01	37.82	46.22	71.43
Block submerged weight (pounds)		40.89	65.78	83.33	129.77
ALLOWABLE SHEAR STRESSES (lb./sq. ft.) (Factor of Safety = 1.0)					
Level Bed Critical Shear Stress - Over-Turning - No Cable Interaction	0.000	24.7	24.6	25.3	27.1
Level Bed Critical Shear Stress - Sliding - With Cable Interaction	0.000	10.1	18.1	22.1	34.2
Level Bed Critical Shear Stress - Sliding - No Cable Interaction	0.000	6.0	10.7	13.2	20.7
Bed Slope 1.5H:1V (34 degrees) - Over-Turning - No Cable Interaction	0.667	17.2	15.4	17.0	15.5
Bed Slope 1.5H:1V (34 degrees) - Sliding - With Cable Interaction	0.667	1.8	3.3	4.0	6.2
Bed Slope 1.5H:1V (34 degrees) - Sliding - No Cable Interaction	0.667	1.1	2.0	2.4	3.7
Bed Slope 2H:1V (26.5 degrees) - Over-Turning - No Cable Interaction	0.500	20.0	18.5	19.5	18.8
Bed Slope 2H:1V (26.5 degrees) - Sliding - With Cable Interaction	0.500	3.7	6.7	8.2	12.7
Bed Slope 2H:1V (26.5 degrees) - Sliding - No Cable Interaction	0.500	2.2	4.0	4.8	7.5
Bed Slope 3H:1V (18.5 degrees) - Over-Turning - No Cable Interaction	0.333	22.2	21.2	22.0	22.1
Bed Slope 3H:1V (18.5 degrees) - Sliding - With Cable Interaction	0.333	5.8	10.4	12.7	19.6
Bed Slope 3H:1V (18.5 degrees) - Sliding - No Cable Interaction	0.333	3.4	6.2	7.5	11.6
Bed Slope 4H:1V (14 degrees) - Over-Turning - No Cable Interaction	0.250	23.1	22.3	23.2	23.7
Bed Slope 4H:1V (14 degrees) - Sliding - With Cable Interaction	0.250	6.9	12.4	15.2	23.4
Bed Slope 4H:1V (14 degrees) - Sliding - No Cable Interaction	0.250	4.1	7.3	9.0	13.9
Bed Slope 5H:1V (11.3 degrees) - Over-Turning - No Cable Interaction	0.200	23.5	22.9	23.8	24.6
Bed Slope 5H:1V (11.3 degrees) - Sliding - With Cable Interaction	0.200	7.5	13.5	16.5	25.5
Bed Slope 5H:1V (11.3 degrees) - Sliding - No Cable Interaction	0.200	4.4	8.0	9.8	15.1
MAXIMUM ALLOWABLE VELOCITIES (ft./sec.)					
Note: Velocities above 25.0 feet per second are not recommended.					
Level Bed Maximum Velocity - Over-Turning - No Cable Interaction	0.000	26.0	25.9	26.3	27.2
Level Bed Maximum Velocity - Sliding - With Cable Interaction	0.000	17.4	23.4	25.9	32.2
Level Bed Maximum Velocity - Sliding - No Cable Interaction	0.000	13.5	18.0	20.0	25.0
Bed Slope 1.5H:1V (34 degrees) - Over-Turning - No Cable Interaction	0.667	21.7	20.5	21.6	20.6
Bed Slope 1.5H:1V (34 degrees) - Sliding - With Cable Interaction	0.667	7.5	10.0	11.1	13.7
Bed Slope 1.5H:1V (34 degrees) - Sliding - No Cable Interaction	0.667	5.7	7.7	8.5	10.6
Bed Slope 2H:1V (26.5 degrees) - Over-Turning - No Cable Interaction	0.500	23.4	22.5	23.1	22.6
Bed Slope 2H:1V (26.5 degrees) - Sliding - With Cable Interaction	0.500	10.6	14.2	15.7	19.5
Bed Slope 2H:1V (26.5 degrees) - Sliding - No Cable Interaction	0.500	8.1	10.9	12.1	15.0
Bed Slope 3H:1V (18.5 degrees) - Over-Turning - No Cable Interaction	0.333	24.6	24.1	24.5	24.6
Bed Slope 3H:1V (18.5 degrees) - Sliding - With Cable Interaction	0.333	13.3	17.8	19.7	24.5
Bed Slope 3H:1V (18.5 degrees) - Sliding - No Cable Interaction	0.333	10.2	13.7	15.1	18.8
Bed Slope 4H:1V (14 degrees) - Over-Turning - No Cable Interaction	0.250	25.1	24.7	25.2	25.5
Bed Slope 4H:1V (14 degrees) - Sliding - With Cable Interaction	0.250	14.4	19.3	21.3	26.5
Bed Slope 4H:1V (14 degrees) - Sliding - No Cable Interaction	0.250	11.1	14.8	16.4	20.4
Bed Slope 5H:1V (11.3 degrees) - Over-Turning - No Cable Interaction	0.200	25.4	25.0	25.5	25.9
Bed Slope 5H:1V (11.3 degrees) - Sliding - With Cable Interaction	0.200	15.1	20.2	22.3	27.8
Bed Slope 5H:1V (11.3 degrees) - Sliding - No Cable Interaction	0.200	11.6	15.5	17.2	21.4

Testing Authority -- Colorado State University, University of Minnesota, University of Windsor

Failure Mode -- Loss of intimate contact.

Maine Department of Transportation

Scour Critical Bridge Plan of Action (POA) Report

☑ **Full POA**
☐ **Abbreviated POA**

Town: Oxbow Plt
Bridge Number: 2877
Bridge Name: Umcolcus Stream
Feature Carried: Oxbow Road (#318)
Waterway Crossed: Umcolcus Stream

Final Recommended Action:

Increased Inspection Frequency: ☑ Yes ☐ No Annual

Flood Monitoring:

☑ Flood Warning Issued by National Weather Service
☐ USGS Gage Station Station#:

Frequency of Flood Monitoring: 12 hrs

Closure Trigger:

☐ Water Surface Elevation Reaches Low Chord
☑ Water Reaches Closure Elevation:
El. 46.0 ft (approx. 50-yr event)

SEE SECTION 8 FOR OTHER CLOSURE CONSIDERATIONS

Interim Reopening Trigger:

SEE FIELD VERIFICATION CARD

SEE SECTION 9 FOR OTHER REOPENING CONSIDERATIONS

Hydraulic/Structural Countermeasures: ☑ Yes ☐ No

The Following Materials Are Being Submitted With This Report:

- ☑ **POA Report**
- ☑ **Attachment A: Hydraulic and Hydrologic Summary Page**
- ☑ **Attachment B: Photos**
- ☑ **Attachment C: Map Showing Detour Route(s)**
- ☑ **Attachment D: Bridge Elevation Summary Showing Existing Streambed and Foundation Depth(s)**
- ☐ **Attachment E: Boring Logs and/or Other Subsurface Information**
- ☑ **Attachment F: Supporting Documentation, Calculations, Estimates, and Conceptual Designs for Scour Countermeasures**
- ☑ **Attachment G: Plan View Showing Location of Scour Holes, Debris, etc,**
- ☑ **Attachment H: Post Flood Inspection Documentation**
- ☑ **Attachment I: Field Verification Card**
- ☑ **Attachment J: MaineDOT Underwater Inspection Reports**
- ☐ **Attachment K: T.Y. LIN 1995 Bridge Scour Evaluation Report**
- ☑ **Attachment L: 2008 USGS Report**
- ☑ **Attachment M: Scour / H&H Backup Calculations**

SCOUR CRITICAL BRIDGE - PLAN OF ACTION
Oxbow Plt 2877

1. GENERAL INFORMATION

Structure: Umcolcus Stream	**City, County:** Oxbow Plt, Aroostook	**Bridge Number:** 2877
Feature On: Oxbow Road (#318)	**Waterway Crossed:** Umcolcus Stream	**Owner:** State Highway Agency
Year Built: 1954	**Year Reconstruction:** NA	

Structure Size and Description:

This single span bridge was built in 1954 and has had no documented subsequent rehabilitation. Bridge has a concrete deck supported by steel beams. Both abutments are concrete, founded on spread footings with varying thicknesses and are located in the channel. Plans indicate that concrete abutments of a previous bridge were covered with a concrete facade during construction which effectively raised the visible top of the footing by a minimum of 1 foot. Furthermore, the new facade is indicated on the plans to have a minimum vertical height of 4.3 feet at the Left Abutment and 6.5 feet at the Right Abutment.

Foundation Details: (Looking Downstream L to R)

KNOWN ☑
UNKNOWN ☐

Left Abutment (Looking Downstream)	Embedment (ft): 1	Exposure (ft): 3.3
Right Abutment (Looking Downstream)	Embedment (ft): 1.2	Exposure (ft): 5.3
Pier 1	Embedment (ft): N/A	Exposure (ft): N/A
Pier 2	Embedment (ft): N/A	Exposure (ft): N/A
Pier 3	Embedment (ft): N/A	Exposure (ft): N/A
Pier 4	Embedment (ft): N/A	Exposure (ft): N/A

Reference Datum: Plans

Scour Critical Footing Elevation(s) (ft):

(Note: Measurements are made to 1 foot above the bottom of the footing elevation)

Left Abutment (Looking Downstream)	39	Pier 1: N/A
		Pier 2: N/A
Right Abutment (Looking Downstream)	33	Pier 3: N/A
		Pier 4: N/A

2. RESPONSIBLE FOR POA

Author(s) of POA (name, title, agency/organization, telephone, email):

Robert Faulkner P.E., CHA Principal Eng. VI, (603) 357-2445, chakeene@chacompanies.com

Signature: [signature] **Date:** 5/28/2010

STATE OF MAINE ROBERT J. FAULKNER No. 11944 LICENSED PROFESSIONAL ENGINEER

Concurrences on POA (name, title, agency/organization, telephone, email):

Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580 **Date:** ______

POA Updated by: Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

Date of Update: ______ **Items Updated:** ______

Reason for Update: ☐ Inspection Cycle ☐ Monitoring Event ☐ Post Flood Inspection

POA Updated Every ____ **months by (name, title, agency, organization):** Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

Next Update: ____ months

3. SCOUR VULNERABILITY

Current Item 113 Code: [x] 3 [ ] 2 [ ] 1 **Other:**

Source of Scour Critical Code: [x] Observation [ ] Assessment [ ] Calculation **Other:**

Scour Evaluation Summary: Water is relatively shallow, but a diving inspection was performed in 2005 which recorded scour along the footings, but no undermining. At the time of CHA's June 2009 field visit, both abutments were exposed with a remaining embedment of 1.2 ft for the Right abutment and 1 ft at the Left abutment. The Maine State Planning Office does not provide flood history info for this bridge, but a 2008 USGS study provides scour estimates for the bridge. The primary bed material was field estimated as gravel and the secondary bed material was estimated as cobbles. The bed armoring potential is high. There was no protection present at either abutment. CHA recommends that partially grouted riprap countermeasures be installed along both abutments. It is also recommended that the bridge be monitored during and after significant flood events. CHA recommends this bridge be inspected annually.

Scour History: Bridge was built in 1954. No FEMA flood history for Umcolcus Stream in Oxbow Plantation. In addition, the Maine State Planning Office does not provide flood history info for Umcolcus Stream.

4. NBI CODING INFORMATION

		Current	Previous
	Date	5/28/2010	2/21/2007
Item 113	Scour Critical	3	3
Item 60	Substructure	6	6
Item 61	Channel & Channel Protection	5	5
Item 71	Waterway Adequacy	9	9

5. RECOMMENDED ACTION(S)

	Recommended		Implemented		Date
a. Increased Inspection Frequency:	[x] Yes	[ ] No	[ ] Yes	[ ] No	
b. Fixed Monitoring Device(s):	[ ] Yes	[x] No	[ ] Yes	[ ] No	
c. Flood Monitoring Program:	[x] Yes	[ ] No	[ ] Yes	[ ] No	
d. Hydraulic/Structural Countermeasures:	[x] Yes	[ ] No	[ ] Yes	[ ] No	

6. MONITORING PROGRAM RECOMMENDATION

6a. Regular Inspection Cycle

- [ ] Biennial
- [x] Annual
- [x] Riverbed Profile Readings - Upstream Face
- [x] Riverbed Profile Readings - Downstream Face
- [ ] Surveyed Cross-Section

Items to Watch: Changes to embedment at the abutments. The embedment is estimated to be 1.0 ft at the left abutment and 1.2 ft at the right abutment. Perform a drop-line survey to establish the riverbed profile at the upstream and downstream fascia and along each substructure. Bridge should be inspected on an annual cycle.

Underwater Inspection Required

- [ ] 4 Year Cycle
- [ ] 2 Year Cycle
- [ ] 1 Year Cycle

Items to Watch:

- [ ] **6b. Fixed Monitoring Device(s)** — [x] **Not Applicable**

Type of Instrument:

Installation Location(s):

Routine Sample Interval: [ ] 30 minute [ ] 1 hr [ ] 6 hrs [ ] 12 hrs [ ] Other

Frequency of Data Download and Review: [ ] Daily [ ] Weekly [ ] Monthly [ ] Other:

Action(s) Required if Scour Critical Elevation Detected: *(See Section 7 and Section 8)*

Criteria of Termination For Fixed Monitoring:

Event Sample Interval: [ ] Continual [ ] 10 min [ ] 30 min [ ] 1 hour [ ] Other

Frequency of Data Download and Review: [ ] Daily [ ] Weekly [ ] Monthly [ ] Other:

Action(s) Required if Scour Critical Elevation Detected: *(See Section 7 and Section 8)*

Criteria of Termination For Event Monitoring

- [x] **6c. Flood Monitoring Program**

During Inspection Event, Look For:

Water at or above placard elevation.

Type: [x] Visual Inspection

[ ] Instrument *(check all that apply)*

[ ] Portable [ ] Geophysical [ ] Sonar [ ] Other

Flood Monitoring event defined by: *(check all that apply)*

- [x] Notified By Public
- [x] Flood Warning Issued by National Weather Service
- [x] DOT Situation Report

- [ ] USGS Gage Station Station#:
  - [ ] Stage (Water Surf. Elev.)
  - [ ] Discharge

Frequency of Flood Monitoring: [ ] Continual [ ] 3 hrs [x] 12 hrs [ ] Daily

Criteria to End Flood Monitoring:
- [x] Revisit Bridge
- [x] Recommended Post Flood Inspection
- [x] Close Bridge (See Section 8)
- [x] Conditions Stable / Water Receding

Action(s) Required if Scour Critical Elevation Detected: *(See Section 7 and Section 8)*

Assess changes in channel/riverbed profiles, possible undermining evidence, and overall stability. Consider closing bridge.

☑ **6d. Post-Flood Inspection Tasks Required**

- ☑ Visual Inspection
- ☑ Riverbed Profile Readings - Upstream and Downstream face
- ☑ Profile at Substructure
- ☐ Undermining
- ☐ Underwater Inspection
- ☑ Probing

Items to Watch: Changes to embedment at the piers or abutments. The embedment is estimated to be 1.2 ft at the left abutment and 1 ft at the right abutment. Perform a drop-line survey to establish the riverbed profile at the upstream and downstream fascia and along each substructure.

Date of Event: ________ **Date of Post Flood Inspection:** ________

Agency and Department Responsible for Monitoring:

Maine DOT

Contact Person (name, title, agency/organization, telephone, e-mail):

- Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580
- Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

7. DETOUR ROUTE

Detour Route Description:

No Public Road Detour.

Bridges on Detour Route: To be provided by Maine DOT

Bridge Number	Feature On	Feature Under	Item 113	Load Posting (tons) / Date	Vertical Clearance (feet)	Width Restrictions (feet)

Traffic Control Equipment and Storage location(s):

Additional Considerations or Critical Issues:

News Release, Other Public Notice Information to be provided and limitations:

Public Information Officer, Office of Communications - (207) 624-3030

8. BRIDGE CLOSURE PLAN

Criteria For Consideration of Bridge Closure: *Check all that apply*

- [ ] Water Surface Elevation Reaches Low Chord
- [x] Water Reaches Critical Monitoring Elevation: El. 46.0 ft (approx. 50-yr event)
- [x] Overtopping Road or Structure
- [x] Scour Measurement Results / Monitoring Device (See Section 6)
- [x] Observed Structure Movement / Settlement
- [ ] USGS Gage Station #
- [ ] Stage (WSE)
- [ ] Discharge
- [x] Other: [x] Loss of Road Embankment [x] Ice Jam [x] Debris Accumulation [ ] Movement of RipRap / Other Armor Protection

Agency and Department Responsible for Closure:

[x] DOT [ ] Municipality: [ ] Other

Contact Person(s) (name, title, telephone, email):

Maine DOT Radio Operations, (207) 624-3339

9. BRIDGE REOPENING PLAN

9a. Criteria for Consideration to Complete Interim Bridge Reopening:

- [x] Water Surface Levels Dropping
- [x] Critical Elevation Marker Is Visible
- [x] Reasons for Closure Have Abated
- [x] Verify Riverbed Elevation (drop line readings)
- [x] Streambed Elevation Drops Less than 0.1 Feet

Agency and Person Responsible for Re-Opening Bridge After Inspection:

- Region Bridge Manager
- Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580
- Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

9b. Criteria for Completing Bridge Reopening Process:

- [x] Post Flood Inspection Completed
- [ ] Diving Inspection Completed within 7 calendar days

10. COUNTERMEASURE RECOMMENDATIONS

Conceptual Structural / Hydraulic Countermeasures:

	Countermeasure	Priority	Estimated Cost
(1)	Place partially grouted riprap along both abutments.	[x] Yes [ ] No	$ 120,000
(2)		[ ] Yes [ ] No	$
(3)		[ ] Yes [ ] No	$

Basis for the Selection of the Preferred Scour Countermeasure: PGR installation will help protect footings from further scour and/or undermining.

Recommended Countermeasures to be Performed by:

- [ ] Bridge Maintenance
- [ ] Bridge Program
- [ ] Highway Program
- [ ] Other

Recommended Completion Date: ______________

Contact Person: *(include name, title, telephone, e-mail)*

- Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580
- Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

MaineDOT Scour Investigation East PIN:15631.10
Hydraulic and Hydrologic Summary Page (POA Attachment A)

Bridge ID: Oxbow Plt 2877 **Name:** Umcolcus Stream **Date:** 8/4/2009

GENERAL INFORMATION

Town:	Oxbow Plt
Feature Carried:	Oxbow Road (#318)
Functional Class:	09 - Rural Local
Detour Length:	43 (miles)
Year Built:	1954
Year of Reconstruction:	NA

Owner:	State Highway Agency		
Feature Crossed:	Umcolcus Stream		
Major Basin (HU8):	Aroostook		
Bridge Plan File Loc:	NO DATA		
Capacity (Actual Metric Tons, Signed) **:**	30.8 ,		
ADT:	108	**Year of ADT:**	2006
Overall Fed Sufficiency Rating:	63.9		

Current: (scale of 1-9) (worst - best)	Substructure Item 60:	Channel Stability Item 61:	Hydraulic Adequacy Item 71:	Scour Risk Item 113:
	6	5	9	3

HYDRAULIC INFORMATION

FEMA Study:	☐	
USGS Report:	☑	1/1/2008
Tidal Influence:	☐	
Watershed Area (sq. mi):	82.4	
100-Yr Overtopping Relief:	On Bridge	

T.Y. Lin Information (Bridge Scour Report Nov 1995) **:**	☐
100-Yr Water Velocity (feet per sec):	10
Angle Of Attack (Flood Flow) **:**	20

Other Hydrologic & Hydraulic Data:

2008 USGS Scour Study for bridge includes a HEC-RAS model (Plan Datum = USGS model elevations - 56.7 ft). Maine DOT also provides flow data at bridge (Q10=2271, Q50=3176, Q100=3591, Q500=4575). No FEMA FIS for Oxbow Plantation.

Table:	Flow (cfs)	Elevation (ft)	Flow Impacting Bridge	Flow Overtopping Bridge
Low Chord		49		
Roadway		51.9		
10 year	1860	44.9	No	No
50 year	2560	46.5	No	No
100 year	2890	47.1	No	No
500 year	3640	48.2	No	No

BRIDGE INFORMATION

Bridge Width (in feet):	25.50
Plans Available:	☑
Worst Abutment (Looking Downstream L to R) **:**	Left
Abutment Foundation:	Spread Footing

Bridge Length (in feet):	55.00
Number of Spans:	1
Borings Available:	☐
Worst Pier (Looking Downstream L to R) **:**	0
Pier Foundation:	N/A

Flood / Scour History Comments: Bridge was built in 1954. No FEMA flood history for Umcolcus Stream in Oxbow Plantation. In addition, the Maine State Planning Office does not provide flood history info for Umcolcus Stream.

RECOMMENDATIONS

Field Investigation Rec: ☑	**Agree w/ Item 113 Rating:** ☑	**Scour POA Recommended:** ☑
Comp: ☑	**CHA Rec Item 113 Rating:** 3	**CHA Recommended POA Choice:** Partially Grouted Riprap

Bridge: Oxbow Plt 2877
Date Taken: 6/10/2009 7:48:22 PM
Source: Nobis
Description: Upstream channel from bridge

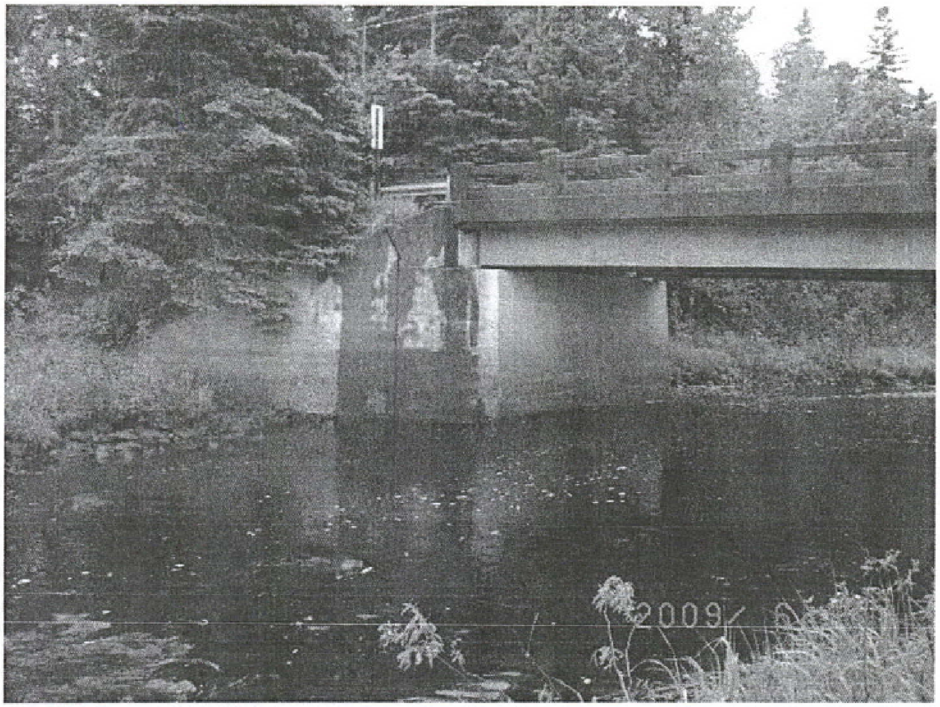
Bridge: Oxbow Plt 2877
Date Taken: 6/10/2009 7:48:38 PM
Source: Nobis
Description: Upstream Bridge Elevation

Bridge: Oxbow Plt 2877

Date Taken: 6/10/2009 7:39:13 PM

Source: CHA

Description: Right Abutment Downstream

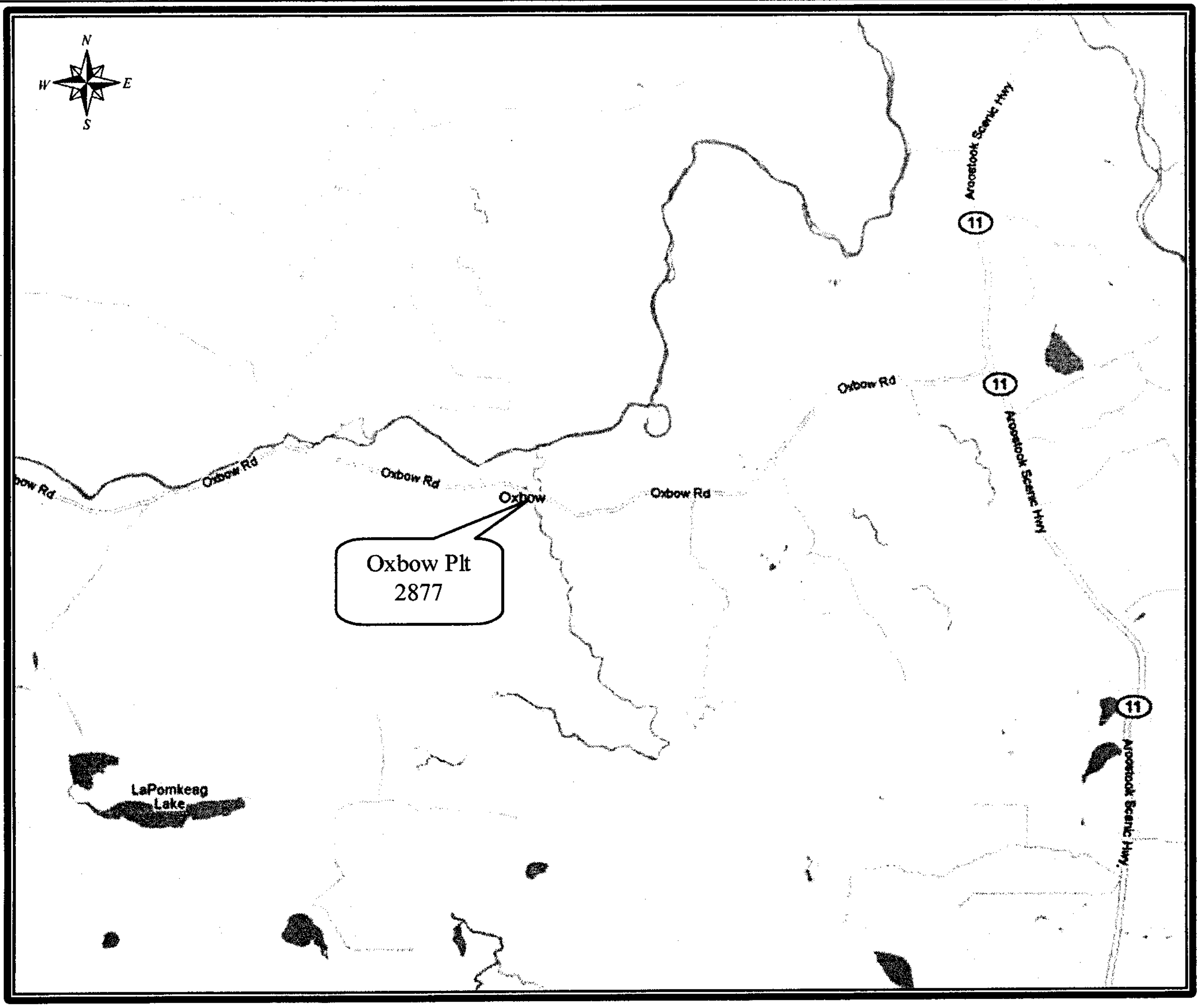


Bridge: Oxbow Plt 2877

Date Taken: 6/10/2009 7:48:46 PM

Source: Nobis

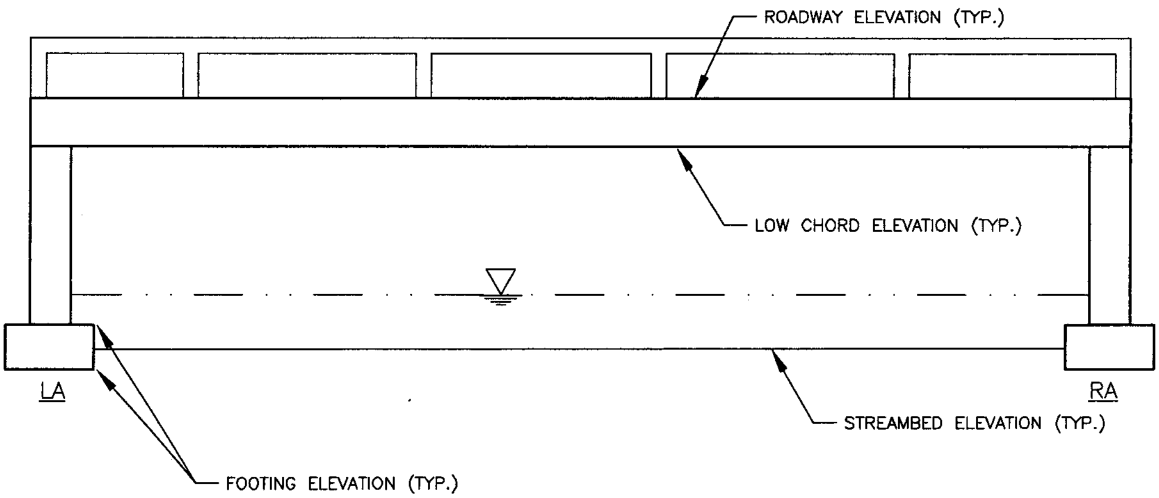
Description: Downstream Bridge Elevation



DETOUR MAP

NO PUBLIC ROAD DETOUR.

N	CHA	**Oxbow Plt 2877** **Attachment C: Detour Route**
	11 King Court Keene, NH 03431-4648 Main: (603)-357-2445	**Maine Scour Investigation**



UPSTREAM ELEVATION

NOT TO SCALE

SUBSTRUCTURE UNIT	ROADWAY ELEVATION (FT)	LOW CHORD ELEVATION (FT)	FOOTING ELEVATION TOP	FOOTING ELEVATION BOTTOM	STREAMBED ELEV. ORIGINAL DESIGN PLANS	STREAMBED ELEV. CHA FIELD (2009)	100 YEAR SCOUR ELEV.
LA	51.94	47.76	42.35	38.00	41.00	39.00	N/A
RA	52.72	48.70	38.54	32.00	36.00	33.2	N/A

NOTES:

1. UNLESS OTHERWISE SPECIFIED, ALL ELEVATIONS ARE REFERENCED TO THE DESIGN PLAN DATUM.
2. THE SUBSTRUCTURE LABELING CONVENTION ESTABLISHED BY CHA DEFINES LEFT AND RIGHT LOOKING DOWNSTREAM.
3. STREAMBED ELEVATIONS GIVEN ARE APPROXIMATE ELEVATIONS AT EACH SUBSTRUCTURE UNIT AND ALONG THE FASCIA INDICATED. FOR DETAILED SUBSTRUCTURE RIVERBED PROFILE MEASUREMENTS, SEE ATTACHMENT H. ATTACHMENT G PROVIDES THE LOCATION OF SCOUR RELATED FEATURES AND OTHER MINOR STREAMBED VARIATIONS.

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MAINE BRIDGE SCOUR INVESTIGATION

OXBOW PLT 2877

BRIDGE ELEVATION SUMMARY

ATTACHMENT D

DATE: 4/15/10

Drawn By: ML

Attachment F:
Supporting Documentation, Calculations, Estimates, and Conceptual Designs for Scour Countermeasures

CHA

Oxbow Plt 2877
Attachment F: Scour Countermeasures

11 King Court
Keene, NH 03431-4648
Main: (603)-357-2445

Maine Scour Investigation

ESTIMATED COST OF PROPOSED COUNTERMEASURE - Partially Grouted Riprap:

Bridge No.	2877	SSU's Protected =>	Right and Left Abutments
Town	Oxbow Pit	Spans Protected =>	N/A
Dist. Between Abutment Faces	50'	Approved Countermeasure Type =>	Partially Grouted Riprap
Bridge Width	50	Low Flow Water Depth =>	4'
Max. Vertical Opening	10'	Design Flow water depth =>	11

SSU	CM TYPE	Item	Length (ft)	Width (ft)	Depth (ft)	No. Items	Quantity	Unit	Unit Price	Subtotal
Access to work area						1	1	LS	$7,500	$7,500
Right and Left Abutments	Partially Grouted Riprap	Water Diversion System	N/A	N/A	N/A	2	2	LS	$7,000	$14,000
		Excavation and site prep	94	22	N/A	2	460	SY	$20	$9,191
		Provide and place filter	94	22	N/A	2	460	SY	$50	$22,978
		Provide and place riprap	94	22	2	2	306	CY	$75	$22,978
		Partial grouting	94	22	N/A	2	460	SY	$50	$22,978
		Proposed work: - divert water - excavate and level area - place filter - place riprap - partial grout	Note - width at abutments = 2 x design flow depth and extended US & DS same amount past fascia.						**Mobilization**	$5,000
									Subtotal	$97,124
							20%		**Contingency**	$19,425
									Total Estimated Cost	$116,549
									Conceptual Budget	**$120,000**

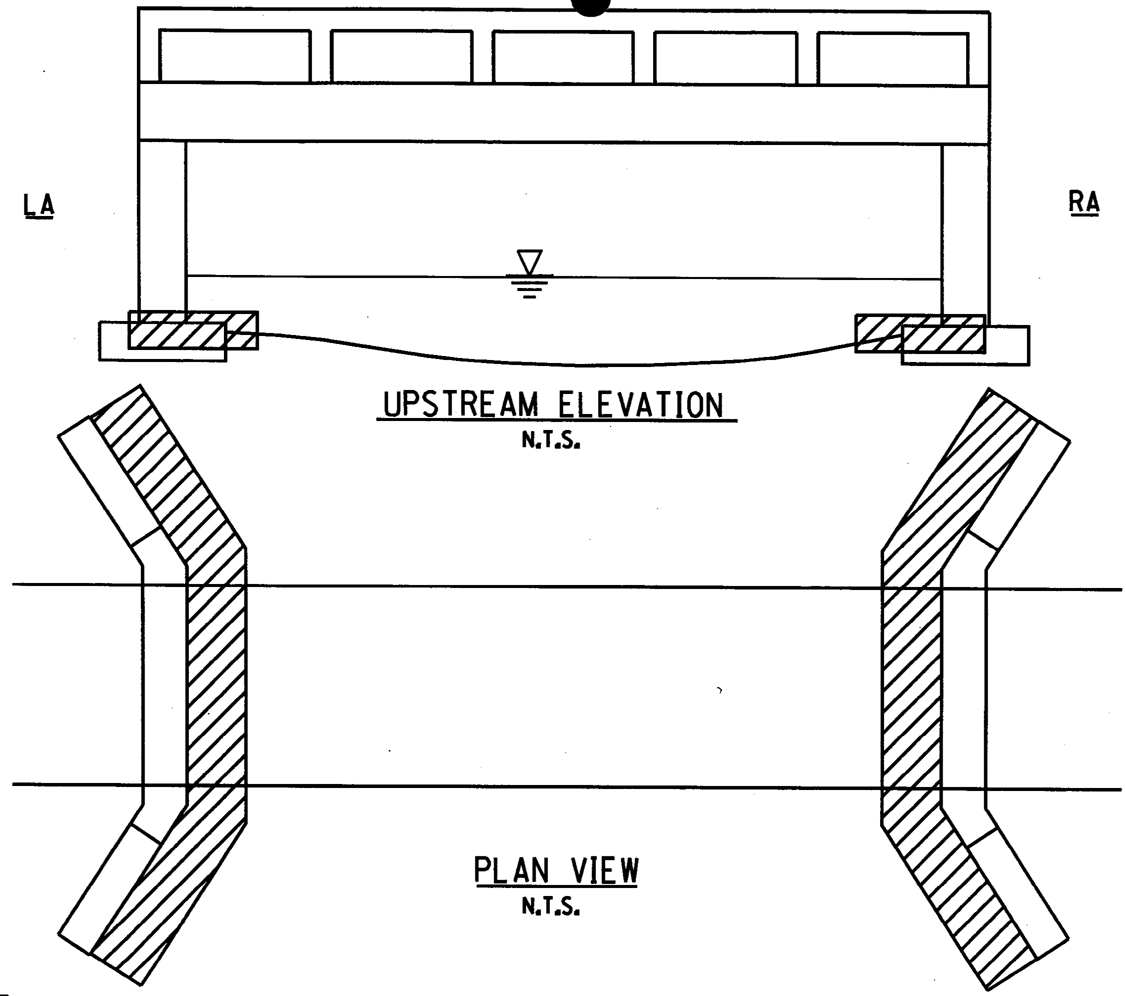
Assumptions:

Environmental impact associated with the proposed work is judged to be: *minor*

Given the estimated quantities for this item, the unit price is judged to be *equal to* average unit prices.

Notes:

The cost of mitigating or eliminating potential environmental impacts is not included in this estimate.
The cost of environmental permit preparation is also not included herein.
Length and Width dimensions are for outer extent of countermeasures; actual quantity accounts for rounded corners of application area.



LEGEND

INSTALL PARTIALLY GROUTED RIPRAP

CHA

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MAINE BRIDGE SCOUR INVESTIGATION	ATTACHMENT
OXBOW PLT 2877	F
CONCEPTUAL COUNTERMEASURE SUMMARY	DATE: 5/10

RIP-RAP Sizing at Bridge Abutments

Project Number: 82620

Project Name: Oxbow Rd over Umcolcus Stream in Oxbow Pit, Maine (Bridge # 2877)
100-year flood event data

D_{50} Equation Used for Froude Number <=0.80 = $y((k/(S_s-1)(V^2)/gy))$

D_{50} Equation Used for Froude Number >0.80 = $y((k/(S_s-1)(V^2)/gy)^{0.14})$

Where:

V = Characteristic average velocity in the contracted section or velocity at toe of abutment from model output m/s (ft/s)
S_s = Specific Gravity of Riprap
y = Depth of flow adjacent to the abutment
g = Gravitational acceleration m/s^2 (ft/s^2)

User Input:

Enter M for Metric units. Enter E for English units: E

S (Spill-through abutment) or V (Vertical wall abutment) : V (for k value)

Velocity provided by USGS HEC-RAS model V = 7.5 ft/s

S_s = 2.6500

Flow depth provided by USGS HEC-RAS model y = 11.0000 ft

Constants / Calculated: g = 32.2 ft/s2

k = 1.02

Froude Number = (V/gy)1/2 = 0.3985

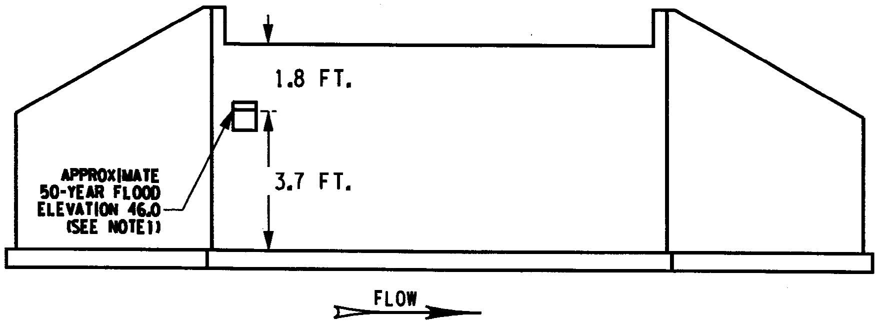
Result: D_{50} = **1.1 ft**

Equations obtained from "Bridge Scour and Stream Instability Counter Measures". US FHWA. March 2001. Equations 8.2 and 8.3

NOTE: Insufficient embedment for standard riprap. Use partially grouted riprap.
NOTE: Partially grouted riprap to run along entire length of abutment and wingwalls

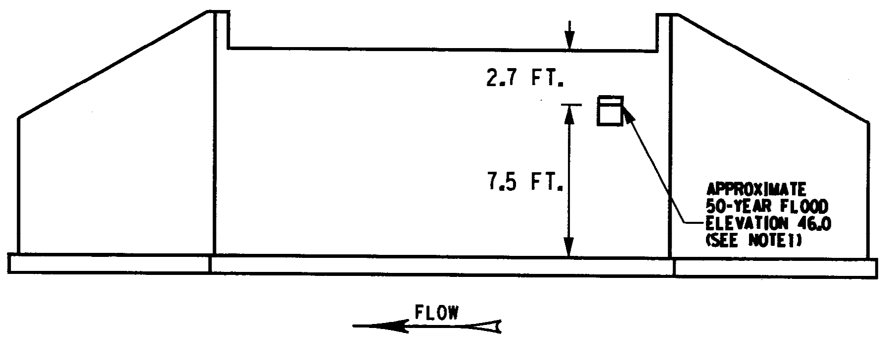
Quantity:			
	Left abutment	10' width, 70' long abutment	700.0 sf
	L abutment corners	10' width, semicircle	157.0 sf
	Right abutment	10' width, 70' long abutment	700.0 sf
	R abutment corners	10' width, semicircle	157.0 sf
	Thickness	2 x D_{50} for Partially-Grouted Riprap	2.0 feet
		Total =	127.0 CY
		Total Riprap Application Area =	190.4 SY

Designed by: DC
Checked by: BV



LEFT ABUTMENT ELEVATION

N.T.S.

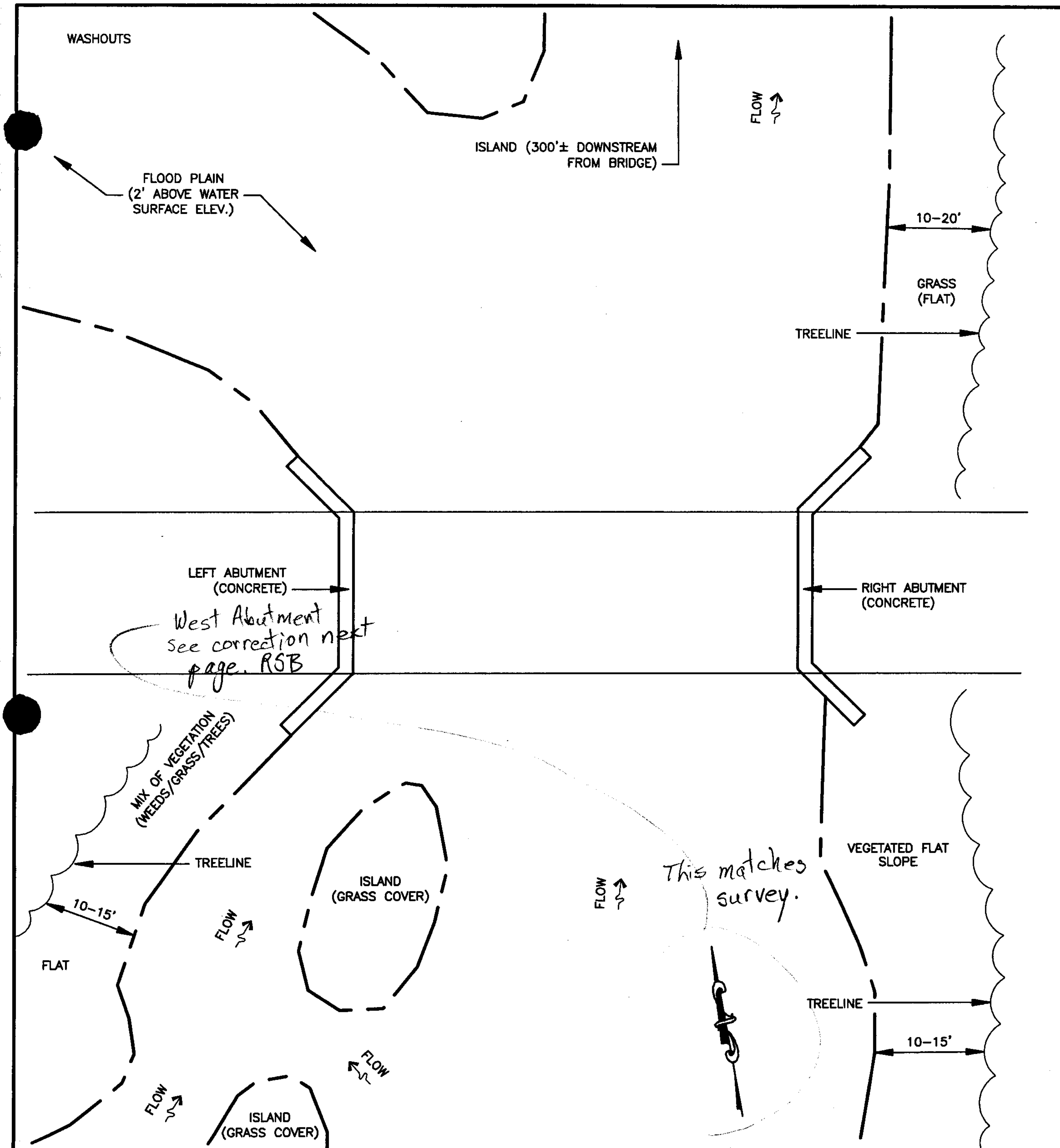


RIGHT ABUTMENT ELEVATION

N.T.S.

NOTES:
1. ELEVATION 46.0 CORRESPONDS TO AN APPROXIMATE 50 YEAR FLOOD EVENT.
2. PLACARD LOCATIONS ARE AT THE UPSTREAM END OF BOTH ABUTMENTS.
3. PLACARD LOCATION MEASURED TO LOW CHORD AND TOP OF FOOTING.
4. ALL WORK IS TO BE PERFORMED IN DRY CONDITIONS.
5. SKETCH IS CONCEPTUAL ONLY, NOT FOR CONSTRUCTION, AND NOT TO SCALE.

Drawing Copyright © 2009 CHA 11 King Court • Keene, NH 03431-4648 Main: (603) 357-2445 • www.chacompanies.com	MAINE BRIDGE SCOUR INVESTIGATION	ATTACHMENT
	OXBOW PLT 2877	F
	PLACARD INSTALLATION LOCATION	DATE: 5/10



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Maine DOT Scour Plan of Action #15631

Field Sketch

Town/Bridge#: Oxbow Plt/2877

ATTACHMENT G:

Plan View of Bridge

DATE: 6/10/09

Drawn By: BV

MAINE DOT CHANNEL PROFILE

Bridge Information

Town/Bridge No.	Oxbow Plt 2877
Feature Carried	Oxbow Road
Feature Crossed	Umcolcus Stream
Owner	MEDOT

Date of Inception 6/9/2009
Inspector KJH

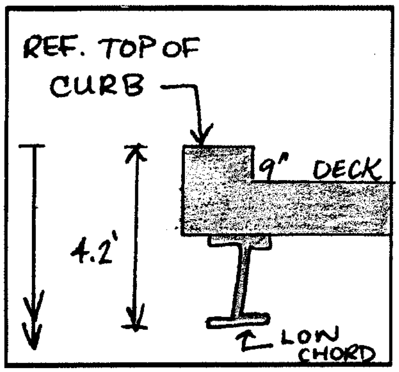
This conflicts w/ survey previous page.

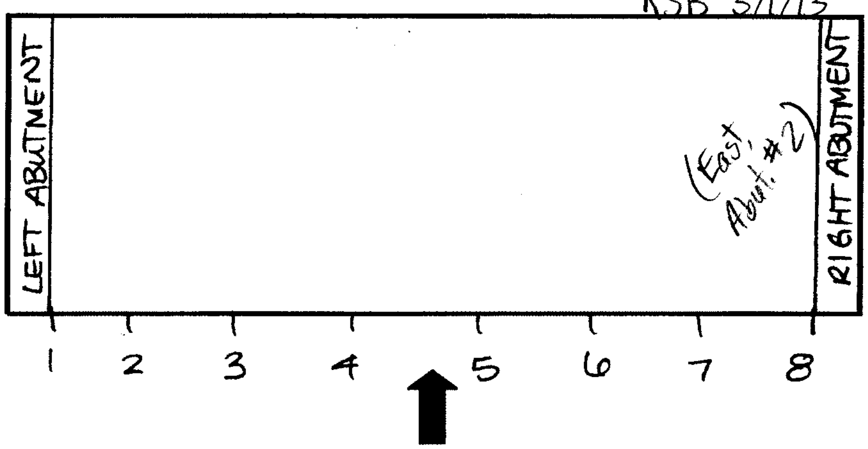
North

Plan View of Dropline Readings

RSB 5/1/13

Datum Reference





Direction of Flow

******Actual skew, if present, not shown*

Fascia Location	Location/Description	Date: 6/9/2009		Date:		Date:	
		Upstream Readings (ft)	Downstream Readings (ft)	Upstream Readings (ft)	Downstream Readings (ft)	Upstream Readings (ft)	Downstream Readings (ft)
Sta. 1	Left Abutment	15.4	13.8				
Sta. 2	Post 1	17.0	14.6				
Sta. 3	Post 2	18.2	14.9				
Sta.4	Post 3	17.0	15.7				
Sta. 5	Post 4	18.0	16.0				
Sta. 6	Post 5	17.3	16.2				
Sta. 7	Post 6	17.8	17.0				
Sta .8	Right Abutment	17.4	17.3				
	Water Surface	**13.9**	**14.3**				

Notes: Stations are numbered from left to right looking downstream. All measurements taken from top of concrete curb.

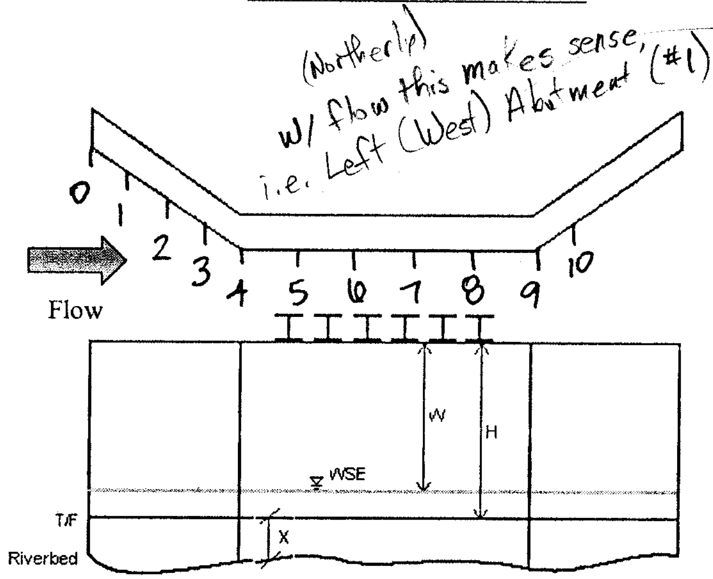
******All units are in English and in decimal form*

MAINE DOT ABUTMENT RIVERBED PROFILE FORM

Bridge Information

Town/Bridge No. Oxbow Plt 2877
Feature Carried Oxbow Road
Feature Crossed Umcolcus Stream
Owner MEDOT

Date of Inspection 7/17/2009
Inspector KJH



Abutment: Left

W = Distance from Low Chord to Water Surface = 9.6 ft

H = Distance from Low Chord to top of footing = 8.4 ft

Location	Location Description	DATE: 7/17/09	DATE:	DATE:
		X (ft)	X (ft)	X (ft)
0	Upstream End Wingwall	-		
1	5' from End Wingwall	0.0		
2	10' from End Wingwall	0.2		
3	15' from End Wingwall / Step Footing	0.0 / 1.8		
4	Upstream Corner	3.3		
5	5' from U/S Corner	3.3		
6	10' from U/S Corner	2.5		
7	15' from U/S Corner	2.3		
8	20' from U/S Corner	1.8		
9	Downstream Corner	1.4		
10	2' from D/S Corner	0.0		

Notes:

**All units are in English and in decimal form*

MAINE DOT ABUTMENT RIVERBED PROFILE FORM

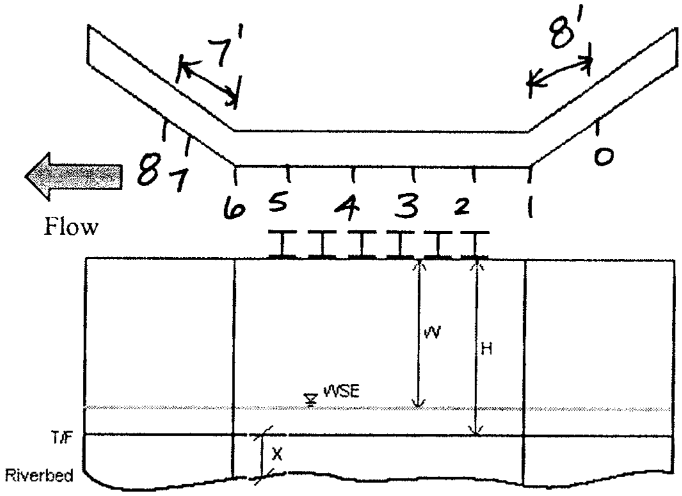
Bridge Information

Town/Bridge No. Oxbow Plt 2877
Feature Carried Oxbow Road
Feature Crossed Umcolcus Stream
Owner MEDOT

Date of Inspection 7/17/2009
Inspector KJH

yes, Abut #2 East. RSB 5/2/13

Abutment: Right



W = Distance from Low Chord to Water Surface = 10.2 ft

H = Distance from Low Chord to top of footing = 10.2 ft

Location	Location Description	DATE: 7/17/09 X (ft)	DATE: X (ft)	DATE: X (ft)
0	U/S Wingwall – Step Footing	2.4		
1	U/S Corner	3.8		
2	5' from U/S Corner	3.9		
3	9' from U/S Corner	5.3		
4	15' from U/S Corner	3.5		
5	20' from U/S Corner	3.3		
6	D/S Corner	3.4		
7	5' from D/S Corner	1.7		
8	D/S Wingall – Step Footing	0.6		
9				
10				

Notes:

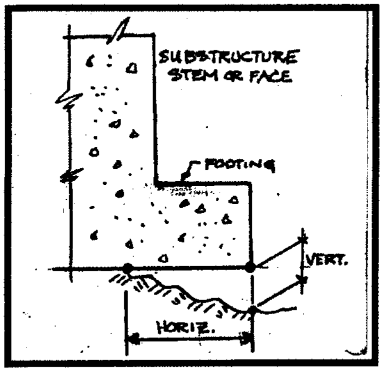
**All units are in English and in decimal form*

MAINE DOT UNDERMINING FORM

Bridge Information

Town/Bridge No. Oxbow Plt 2877
Feature Carried Oxbow Road
Feature Crossed Umcolcus Stream
Owner MEDOT

Date of Inspection 6/9/2009
Inspector KJH



Flow

SSU: ____________

FACE: ____________

Start Of Measurement ____________

Interval of Measurement: ____________

Location	Date:		Date:		Date:		Date:	
	Vertical (ft)	Horizontal (ft)	Vertical (ft)	Horizontal (ft)	Vertical (ft)	Horizontal (ft)	Vertical (ft)	Horizontal (ft)
0								
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								

Notes: No undermining found during 2009 site visit at any substructure unit.

*All units are in English and in decimal form

FIELD VERIFICATION CARD (POA Attachment I)

GENERAL INFORMATION

Structure: Umcolcus Stream **Bridge Number:** 2877

Owner: State Highway Agency **Feature On:** Oxbow Road (#318)

City, County: Oxbow Plt - Aroostook **Waterway Crossed:** Umcolcus Stream

Contact Person: - Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580
- Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

BRIDGE INFORMATION

Detour (miles): 43 **Last ADT:** **Total:** 108 **Year:** 2006

USGS Gage Station: **USGS Station Prox To Bridge:**

Superstructure Type: Stringer/Multi-Beam or Girder **Number of Spans:** 1

Superstructure Material: Steel **Diving Insp Reports:** ☑ **Dates:** 8/13/2001 8/13/2006

Foundation Details: ☐ UNKNOWN	**Worst Abutment:** Left	**Embedment (feet):** 1	**Exposure:** Footing Exposed
(Looking Downstream L to R) ☑ KNOWN	**Worst Pier:** 0	**Embedment (feet):**	**Exposure:**

Scour Critical Feature: abutment footings

Placard Location: Upstream ends of both abutments.

Items to Watch: Changes to embedment at the piers or abutments. The embedment is estimated to be 1.2 ft at the left abutment and 1 ft at the right abutment. Perform a drop-line survey to establish the riverbed profile at the upstream and downstream fascia and along each substructure.

CRITERIA FOR CONSIDERATION OF BRIDGE CLOSURE

- ☐ Water Surface Elevation Reaches Low Chord
- ☑ Water Reaches Critical Monitoring Elevation
  El. 46.0 ft (approx. 50-yr event)
- ☑ Overtopping Road or Structure
- ☑ Debris Accumulation
- ☑ Loss of Road Embankment
- ☑ Scour Measurement Results / Monitoring Device (See Section 6)
- ☑ Observed Structure Movement / Settlement
- ☐ Movement of RipRap / Other Armor Protection
- ☑ Ice Jam
- ☐ USGS Gage Station # ☐ Stage (WSE) ☐ Discharge

ACTION TAKEN

☐ **Post-Flood Inspection Recommendations** ☐ **Revisit Bridge** ☐ **Close Bridge**

POST-FIELD VERIFICATION

☐ **Completed Proper Notification** **Date/Time Notified:** **Agency:**

CRITERIA FOR CONSIDERATION TO COMPLETE INTERIM BRIDGE REOPENING:

- ☑ Reasons for Closure Have Abated
- ☑ Water Surface Levels Dropping
- ☑ Critical Elevation Marker Is Visible
- ☑ Verify Riverbed Elevation
- ☑ Streambed Elevation Drops Less than 0.1 Feet

Agency and Person Responsible for Re-Opening Bridge After Inspection:

- Region Bridge Manager
- Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580
- Assistant Bridge Maintenance Engineer, Maine DOT Bridge Maintenance Division - (207) 624-3580

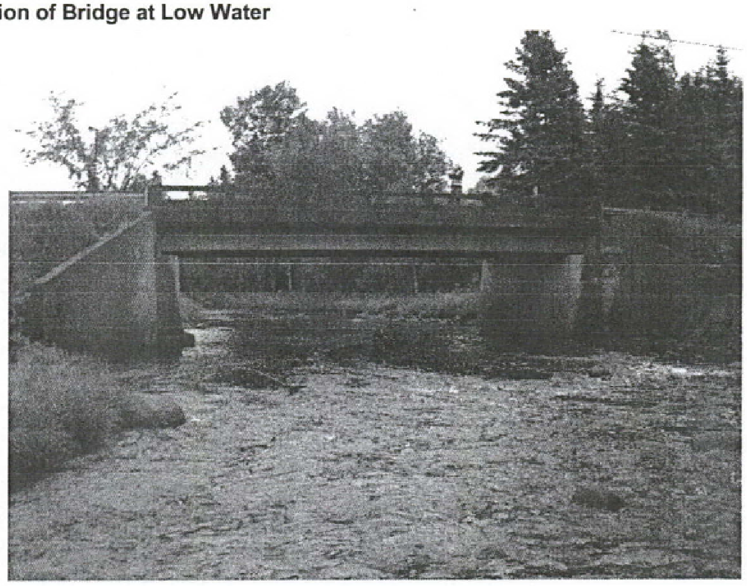
Interim Reopening Approved By: **Interim Reopening Date:** **Time:**

INTERIM REOPENING COMMENTS

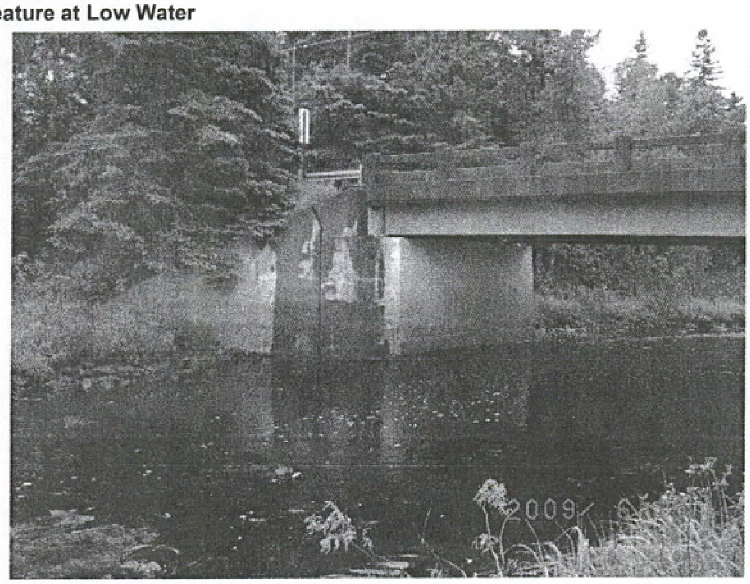
REFERENCE PHOTOS (Left to Right Convention Looking Downstream)

Upstream Low Water

Upstream Elevation of Bridge at Low Water



Scour Critical Feature at Low Water



Attachment J: MaineDOT Underwater Inspection Reports

UNDERWATER DIVE INSPECTION FIELD REPORT

Revised May 5, 2005

BridgeNo: 2877 DiveID: 4737
Town1: Oxbow Plt
Town2:
Bridge Name: UMCOLCUS STREAM
Location: 2.7 MI W TOWNLINE

Feature On Struc: OXBOW ROAD (#318)
Feature Under Struc: UMCOLCUS STREAM
Region: 5

Tidal: (Y/N) No
Tide Information:
Dive Entry Loc: DS, lft corner
Photos: topside
Comments/Haz.: Stream low, didn't have to use scuba. Swift current-in high water. Used mask & snorkel, good vis.
STREAMBED/Comp Material: Scattered gravel, head size stones

CHANNEL DESCRIPTION / CONDITION:

Bridge has a history of scour.
Exposed footings but no undermining found.
Irregular bottom, hilly.
Strong current during high water. Poor alignment.

STRUCTURE DESCRIPTION / CONDITION:

Footings exposed.
37" max on lft footing.
49 1/2" max. on rt. footing. 1 1/4" water on footing.
14' top of curb, center bridge, DS side.
Concrete in good cond. No undermining found.

Footing maximum exposure is more on the higher, right (outside of stream curve). Right is easterly Abut. #2. Yet, page 1 of Field Verification card (3 pages back) identifies the left as "Worst Abutment". Perhaps because right is 6 feet deeper to minimum footing elevation. ~~Accounting for distances from water~~ RSB 5/2/13

INSPECTION TEAM:

Name	Role
Edwards	TL, D
Hannum	SD
Barden	D
Benner	T

TL = Team Leader, D = Diver, SD = Safety Diver, T = Tender

DIVE CONDITIONS:

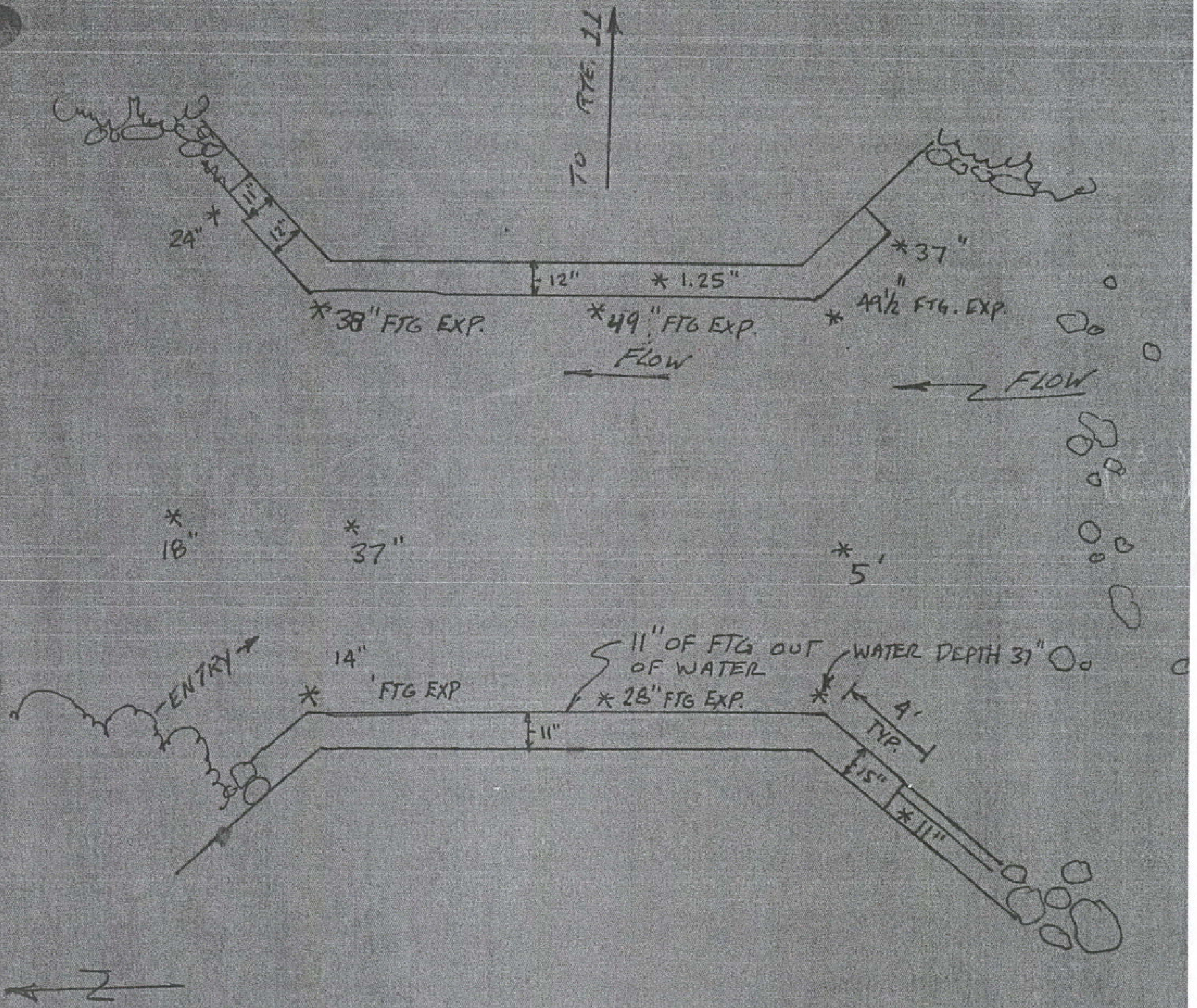
Time: Entry:	11:15
Time: Exit:	11:40
Water Temp:	68
Visibility: (ft.)	6
Max Depth: (ft.)	5
Current:	Slight
Weather:	cloudy

INSPECTION DATE 07242006

Signature: Carl Edwards

Channel Condition: 5
Substruc/Culvert Condition: 7

BRIDGE #2877 OXBOW PLT, UMCOLCUS STREAM



WATER TEMP: 67°
OVERCAST
CURRENT: SLIGHT

2877
OXBOW PLT.
UMCOLCUS
7-2406

Attachment L: 2008 USGS Report
(SIR 2008-5099)

CHA	Oxbow Plt 2877 Attachment L: 2008 USGS Report
11 King Court Keene, NH 03431-4648 Main: (603)-357-2445	Maine Scour Investigation

Comparison of Observed and Predicted Abutment Scour at Selected Bridges in Maine

By Pamela J. Lombard and Glenn A. Hodgkins

Prepared in cooperation with the
Maine Department of Transportation

Scientific Investigations Report 2008–5099

U.S. Department of the Interior
U.S. Geological Survey

U.S. Department of the Interior
DIRK KEMPTHORNE, Secretary

U.S. Geological Survey
Mark D. Myers, Director

U.S. Geological Survey, Reston, Virginia: 2008

For more information on the USGS—the Federal source for science about the Earth, its natural and living resources, natural hazards, and the environment:
World Wide Web: http://www.usgs.gov
Telephone: 1-888-ASK-USGS

Suggested citation:
Lombard, P.J., and Hodgkins, G.A., 2008, Comparison of observed and predicted abutment scour at selected bridges in Maine: U.S. Geological Survey Scientific Investigations Report 2008–5099, 23 p., available only online at http://pubs.usgs.gov/sir/2008/5099.

Contents

Figures

Tables

Conversion Factors and Datum

Inch/Pound to SI

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Area	
square mile (mi^2)	2.590	square kilometer (km^2)
	Flow rate	
foot per second (ft/s)	0.3048	meter per second (m/s)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
	Mass	
pound (lb)	0.4536	kilogram (kg)

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

Horizontal coordinate information is referenced to the North American Datum of 1983 (NAD 83).

Comparison of Observed and Predicted Abutment Scour at Selected Bridges in Maine

By Pamela J. Lombard and Glenn A. Hodgkins

Abstract

Maximum abutment-scour depths predicted with five different methods were compared to maximum abutment-scour depths observed at 100 abutments at 50 bridge sites in Maine with a median bridge age of 66 years. Prediction methods included the Froehlich/Hire method, the Sturm method, and the Maryland method published in Federal Highway Administration Hydraulic Engineering Circular 18 (HEC-18); the Melville method; and envelope curves. No correlation was found between scour calculated using any of the prediction methods and observed scour. Abutment scour observed in the field ranged from 0 to 6.8 feet, with an average observed scour of less than 1.0 foot. Fifteen of the 50 bridge sites had no observable scour. Equations frequently overpredicted scour by an order of magnitude and in some cases by two orders of magnitude. The equations also underpredicted scour 4 to 14 percent of the time.

Introduction

Scour at bridge piers and at abutments are two of the leading causes of bridge failure nationwide (Barkdoll and others, 2006). Excessive abutment scour can cause high maintenance costs and (or) bridge collapse. Abutment scour is a problem in Maine that requires resources for maintenance, repair, and bridge replacement (J. Foster, Maine Department of Transportation, written commun., 2002). Accurate estimation of abutment scour and design of bridges to minimize scour are important for bridge safety and lower construction costs.

Most equations currently used to estimate abutment scour in the Froehlich/Hire method, the Sturm method, and the Maryland method from Hydraulic Engineering Circular 18 (HEC-18; Richardson and Davis, 2001), and the Melville method (Melville, 1997) were developed on the basis of experiments in the laboratory and have not been widely tested for field application. Only the Hire equation in the Froehlich/Hire method was based on field data; it involved scour data at the end of spurs in the Mississippi River (Richardson and Davis, 2001). The accuracy of these methods to predict abutment scour at bridges in Maine is unknown. A study conducted by the U.S. Geological Survey (USGS) to evaluate bridge pier scour in Maine found that HEC-18 pier-scour equations worked reasonably well as envelope equations for bridges in Maine, over-predicting scour by 0.7 to 18.3 ft, and rarely underpredicting scour (Hodgkins and Lombard, 2002).

Scour at bridge sites is generally divided into three components: aggradation or degradation of the river bed, contraction scour at the bridge, and local scour at the bridge piers or abutments. Aggradation and degradation are long-term streambed-elevation changes due to natural or man-induced processes. Contraction scour is a lowering of the streambed in the vicinity of the bridge that is caused by contracted widths at the bridge compared to natural widths upstream. Local scour is caused by an acceleration of flow due to obstructions such as piers or abutments. HEC-18 (Richardson and Davis, 2001) notes that abutments located at or near the channel banks (as opposed to abutments located in the floodplains) are most vulnerable to scour.

Conditions during scour can be clear-water or live-bed. Clear-water scour occurs where there is no movement of bed material into a scour hole during the time of scour. Live-bed scour occurs when there is movement of bed material during the time of scour that often fills or partially fills the scour hole during the falling stage of a flood hydrograph. Most abutment-scour estimation methods require a numerical determination of whether the scour is clear-water or live-bed to determine which equation should be applied.

To test methods that predict maximum abutment-scour depths for bridges in Maine, the USGS began a study in cooperation with the Maine Department of Transportation (MaineDOT) in 2003 to provide a quantitative evaluation of abutment-scour estimation methods that are currently used for designing bridges in Maine, and to evaluate additional methods that have the potential to more accurately predict abutment-scour depths. This report deals primarily with abutment scour; however, some analysis of contraction scour is presented where needed to separate contraction scour from abutment scour for methods that combine both types of scour.

Table 1. Location, age, and physical characteristics of 50 bridges in Maine evaluated for abutment scour.—Continued

Map site identifier	Stream name and town	Road crossing	Longitude (decimal degrees)	Latitude (decimal degrees)	Bridge age (years)	Embankment skew to flow (degrees)	Abutment skew to flow (degrees)	Width of bridge opening (feet)
41	Black Stream at Dover-Foxcroft	Route 7	-69.2361	45.1506	29	30	40	18.2
42	Alder Stream at Atkinson	Atkinson Road	-69.0253	45.1817	42	0	0	24.5
43	Macwahoc Stream at Macwahoc	Route 170	-68.2589	45.6242	85	20	20	37.0
44	South Branch Meduxnekeag River at Cary Plantation	Horseback Road	-67.8661	45.9967	50	28	10	32.8
45	Dunn Brook at New Limerick	Route 2	-67.9917	46.1250	86	40	10	29.3
46	Moose Brook at Houlton	County Road	-67.8903	46.1292	80	50	50	20.0
47	Umcolcus Stream at Oxbow Plantation	Oxbow Road	-68.4914	46.4186	50	20	20	17.6
48	Prestile Stream at Blaine	Pierce Road	-67.8453	46.4958	47	20	20	44.2
49	Salmon Brook at Washburn	Route 228	-68.1583	46.7908	79	15	15	53.0
50	Little Madawaska Stream at Stockholm	Old Route 161	-68.1694	47.0303	75	0	0	23.7

Figure 2. Total station theodolite used to collect survey data at the East Branch of the Wesserunsett Stream at Athens, Maine, August 25, 2004.

Table 2. Variables used in abutment-scour prediction methods for 50 bridges in Maine.—Continued

[ft, feet; ft/s, feet per second; ft^3/s, cubic feet per second; D_{50}, median grain size; D_{max}, maximum grain size]

Site identifier	Stream name and town	Length of active flow blocked by embankment (ft)		Depth of flow upstream from the bridge (ft)			Velocity of flow upstream from the bridge (ft/s)			Flow upstream from the bridge (ft^3/s)			D_{50} (ft)	D_{max} (ft)
		Left bank	Right bank	Left floodplain	Main channel	Right floodplain	Left floodplain	Main channel	Right floodplain	Left floodplain	Main channel	Right floodplain		
31	West Brook at Weld	98.4	103.8	1.9	7.6	2.0	1.0	5.0	1.1	346.2	1,858.0	245.9	0.001	0.06
32	West Branch Souadabscook River at Hampden	5.8	31.8	2.2	6.9	2.6	2.6	8.6	3.0	78.4	2,156.0	415.6	0.148	2.03
33	West Branch Wesserunsett Stream at Skowhegan	15.4	20.5	2.7	9.5	2.6	1.8	6.6	1.8	55.9	3,695.8	98.3	0.210	1.54
34	East Branch Wessurunsett Stream at Athens	0.0	6.6	2.0	6.7	2.4	5.1	12.1	5.6	51.1	1,465.4	603.5	0.295	1.31
35	West Branch Carrabassett River at Kingfield	0.0	0.0	1.2	9.7	1.7	1.5	10.2	2.0	6.1	9,300.1	33.8	[1]0.210	[2]1.02
36	Paul Brook at Corinth	23.0	30.7	2.7	5.0	2.2	1.5	3.6	1.4	99.2	351.8	129.0	0.148	1.25
37	Pushaw Stream at Hudson	8.6	14.0	2.1	8.4	2.2	0.5	4.5	1.2	1.4	1,778.4	10.2	0.295	1.87
38	Sunkhaze Stream at Greenfield	11.9	11.6	2.0	5.8	2.0	1.6	4.6	1.4	35.2	960.0	14.8	0.295	2.46
39	West Branch Dead Stream at Bradford	7.3	8.8	1.4	5.0	1.5	0.4	2.9	0.5	1.2	440.9	3.9	0.003	0.42
40	Middle Branch Dead Stream at Bradford	35.5	3.1	1.5	3.6	0.7	2.0	4.3	0.6	78.2	472.4	1.4	0.105	0.82
41	Black Stream at Dover-Foxcroft	11.0	27.2	1.2	4.8	1.1	0.8	2.8	0.7	9.2	742.3	30.5	0.105	0.95
42	Alder Stream at Atkinson	13.5	17.0	0.0	3.4	0.0	0.0	4.1	0.0	0.0	925.0	0.0	0.148	0.98
43	Macwahoc Stream at Macwahoc	12.6	15.1	1.8	7.0	1.6	1.6	5.9	1.0	11.2	2,167.8	11.0	0.105	0.98
44	South Branch Meduxnekeag River at Cary Plantation	25.4	7.2	0.9	5.2	0.4	1.0	4.8	0.2	67.3	1,232.6	0.1	0.295	1.41
45	Dunn Brook at New Limerick	53.1	27.3	2.0	9.3	1.6	0.8	3.5	0.8	76.3	2,314.6	69.1	0.210	1.25
46	Moose Brook at Houlton	71.3	5.4	2.1	7.1	2.5	1.2	3.0	1.3	422.1	523.0	38.9	0.148	0.69
47	Umcolcus Stream at Oxbow Plantation	30.1	128.5	1.6	6.5	2.5	0.7	3.5	1.1	5.7	2,429.4	274.9	0.210	1.90
48	Prestile Stream at Blaine	60.2	66.8	2.4	7.1	3.7	1.0	4.0	1.2	70.5	3,605.5	164.1	0.105	0.85
49	Salmon Brook at Washburn	52.7	25.2	4.0	9.1	3.8	2.6	7.0	2.3	888.0	1,906.0	106.0	0.295	1.08
50	Little Madawaska Stream at Stockholm	218.4	45.86	5.6	15.3	3.6	0.8	3.1	0.5	913.8	5,254.7	21.5	0.295	1.08

Σ = 2710

[1] D_{50} at left floodplain is .210 ft and at right floodplain is 0.003 ft.

[2] D_{max} at left floodplain is 1.02 ft and at right floodplain is 0.04 ft.

CMA's Q50 = 2560 Q100 = 2890

At main channel d = 6.5 Left Bank flow ≅ 30.1' Right Bank flow ≅ 128.5'

Table 3. Hydrologic information for 50 bridges in Maine evaluated for abutment scour.—Continued

[Peak flow, flow that is assumed to have caused scour based on the age of the bridge; ft^3/s, cubic feet per second]

Site identifier	Stream name and town	Slope in vicinity of bridge	Drainage basin (square miles)	Wetlands (percent)	Estimated recurrence interval of peak flow (years)	Estimated peak flow (ft^3/s)
36	Paul Brook at Corinth	0.001	7.1	0.1	74.1	580
37	Pushaw Stream at Hudson	0.032	46.7	0.2	92.8	1,789
38	Sunkhaze Stream at Greenfield	0.007	16.0	0.2	71.2	1,007
39	West Branch Dead Stream at Bradford	0.001	16.4	0.3	75.5	446
40	Middle Branch Dead Stream at Bradford	0.005	7.9	0.2	98.6	552
41	Black Stream at Dover-Foxcroft	0.002	26.7	0.2	42.3	782
42	Alder Stream at Atkinson	0.004	32.7	0.2	61.1	925
43	Macwahoc Stream at Macwahoc	0.002	57.7	0.2	123.1	2,188
44	South Branch Meduxnekeag River at Cary Plantation	0.001	46.0	0.2	72.6	1,307
45	Dunn Brook at New Limerick	0.012	17.6	0.1	124.6	2,459
46	Moose Brook at Houlton	0.006	15.6	0.2	115.9	984
47	Umcolcus Stream at Oxbow Plantation	0.002	82.4	0.2	72.6	2,711
48	Prestile Stream at Blaine	0.001	88.5	0.2	68.3	3,839
49	Salmon Brook at Washburn	0.017	29.5	0.1	114.5	2,904
50	Little Madawaska Stream at Stockholm	0.001	84.6	0.1	108.7	6,188

Recurrence 73 years.

2001). Surveyed geometric data and Manning's roughness factors for cross sections at each bridge were entered into HEC-RAS. Site hydrologic characteristics (peak flows of a given recurrence interval, *T*) were also entered. The one-dimensional steady-flow water-surface-profile computation component of HEC-RAS was used in the analysis. In addition to the computed *T*-year recurrence-interval flow, a range of flows (with recurrence intervals from 2 to 500 years) was modeled as part of the calibration to test the functionality of the model. Although hydraulic variables generated from one-dimensional HEC-RAS models are estimates and are an additional source of error, the large sample size of 50 bridges makes the evaluation of the scour equations with estimated variables useful. Furthermore, in a similar study comparing observed and predicted abutment-scour depths, Wagner and others (2006) determined that the abutment-scour equations were a larger source of error than the model used to estimate hydraulic variables.

Values of variables used in the abutment-scour methods are sensitive to the location of the channel bank points, those points that define the top of the stream channel. In Maine, the transition between the main channel and the floodplain is not always indicated by a clear break in slope. To determine channel bank locations, the break in slope nearest to the 2-year recurrence-interval flow was chosen. If the break in slope was not clear, the edge of the main channel was defined as the intersection of the water surface of the 2-year recurrence-interval flow with the bed surface.

Although summaries of the tested methods are given below, the original sources of the equations (Richardson and Davis, 2001; Melville, 1997) should be consulted if the user wishes to calculate scour. The description of the methods below is not sufficiently comprehensive for use in estimating abutment-scour depths. In equations where predicted depths of abutment scour in the equations combine abutment scour and contraction scour (Sturm and Maryland methods, below), an equation for contraction scour developed by Laursen and described in HEC-18 (Richardson and Davis, 2001) was used to subtract contraction scour from the combined scour.

Table 4. Predicted and observed maximum abutment-scour depths at 50 bridges in Maine.

[GPR, ground-penetrating radar; ft, feet; --, no GPR measurements made; *italicized* values estimated with Hire equation]

Site identifier	Stream name and town	Visible observed scour (ft)		Scour measured with GPR (ft)		Total observed scour (ft)	
		Left abutment	Right abutment	Left abutment	Right abutment	Left abutment	Right abutment
1	Ogunquit River at Ogunquit	1.2	0	--	--	1.2	0.0
2	Webhannet River at Wells	0	2.8	--	--	0.0	2.8
3	Merriland River at Wells	0	0.8	--	--	0.0	0.8
4	Littlefield River at Alfred	0	0	--	--	0.0	0.0
5	Little River at Gorham	2.5	0	3	0	5.5	0.0
6	Quaker Brook at East Baldwin	0	0	--	--	0.0	0.0
7	Pleasant River at Windham	0	0	--	--	0.0	0.0
8	Pleasant River at Windham	2	0	3	0	5.0	0.0
9	Breakneck Brook at Baldwin	0	0.5	--	--	0.0	0.5
10	Royal River at North Yarmouth	0	0	--	--	0.0	0.0
11	Hancock Brook at Hiram	0	0	--	--	0.0	0.0
12	Tenmile River at Brownfield	0	0.7	--	--	0.0	0.7
13	Gillespie Brook at Bowdoin	2.8	1.8	4	0	6.8	1.8
14	Medomak River at Waldoboro	2.5	0	3	--	5.5	0.0
15	Little Cold River at Stow	1.4	0	0	0	1.4	0.0
16	Meadow Brook at Norway	0	2.3	--	--	0.0	2.3
17	Crooked River at Albany Township (1)	0	0	--	--	0.0	0.0
18	Crooked River at Albany Township (2)	0	0	--	--	0.0	0.0
19	West Branch Sheepscot River at Windsor	1.5	1.4	--	0	1.5	1.4
20	Saint George River at Appleton	0	3	0	0	0.0	3.0
21	Stony Brook at Batchelders Grant	0.5	0	--	--	0.5	0.0
22	Lovejoy Pond Outlet Stream at Wayne	0	1.4	--	--	0.0	1.4
23	Whites Brook at Gilead	1.7	0	--	--	1.7	0.0
24	Concord Brook at Rumford	0	0	--	--	0.0	0.0
25	Stony Brook at Hanover	0	2.7	--	--	0.0	2.7
26	Tunk Stream at Steuben	1.5	0	0	--	1.5	0.0
27	Wilson Stream at Farmington	0	0	--	--	0.0	0.0
28	Stony Brook at Andover	0	1	--	--	0.0	1.0
29	Webb Brook at Waltham	0.4	0	--	--	0.4	0.0
30	Tannery Brook at Waltham	0	2.2	--	0	0.0	2.2
31	West Brook at Weld	2	1.9	0	0	2.0	1.9
32	West Branch Souadabscook River at Hampden	0	2.2	--	--	0.0	2.2
33	West Branch Wesserunsett Stream at Skowhegan	0	2.7	--	0	0.0	2.7
34	East Branch Wessurunsett Stream at Athens	0	0	--	--	0.0	0.0
35	West Branch Carrabassett River at Kingfield	1.2	2.5	--	--	1.2	2.5
36	Paul Brook at Corinth	0.9	0.9	0.5	0.5	1.4	1.4
37	Pushaw Stream at Hudson	0	2.5	--	0.5	0.0	3.0
38	Sunkhaze Stream at Greenfield	1.1	0	--	--	1.1	0.0
39	West Branch Dead Stream at Bradford	0	0	--	--	0.0	0.0
40	Middle Branch Dead Stream at Bradford	0	0	--	--	0.0	0.0
41	Black Stream at Dover-Foxcroft	0	1	--	--	0.0	1.0
42	Alder Stream at Atkinson	1.2	0	1	--	2.2	0.0
43	Macwahoc Stream at Macwahoc	0	0	--	--	0.0	0.0
44	South Branch Meduxnekeag River at Cary Plantation	0	0.9	--	--	0.0	0.9
45	Dunn Brook at New Limerick	0	0	--	--	0.0	0.0
46	Moose Brook at Houlton	0	0	--	--	0.0	0.0
47	Umcolcus Stream at Oxbow Plantation	0	1	--	--	0.0	1.0
48	Prestile Stream at Blaine	1	0.8	--	--	1.0	0.8
49	Salmon Brook at Washburn	0.6	5.2	--	0	0.6	5.2
50	Little Madawaska Stream at Stockholm	0	0.2	--	--	0.0	0.2

Laursen's contraction scour (ft)	Scour predicted by Froehlich/Hire method (ft)		Scour predicted by Sturm method (ft)		Scour predicted by Maryland method (ft)		Scour predicted by Melville method (ft)	
	Left abutment	Right abutment	Left abutment	Right abutment	Left abutment	Right abutment	Left abutment	Right abutment
-1.7	10.7	10.0	12.2	11.5	1.0	0.9	0.0	0.0
-0.5	3.2	1.7	2.6	0.2	1.2	1.2	1.1	0.4
-0.8	4.2	3.6	3.0	2.4	3.5	3.3	1.6	1.3
17.8	11.9	*15.0*	11.2	4.8	10.0	10.0	0.0	0.0
-6.6	15.5	22.7	16.6	2.9	4.5	4.2	5.9	21.3
22.7	*26.1*	*26.1*	-5.2	-17.7	-0.7	-0.7	0.0	0.0
16.8	15.6	16.0	-15.9	-12.7	4.2	4.4	14.7	18.1
2.6	14.4	17.7	5.0	10.6	5.8	6.1	4.2	4.5
-1.4	*16.1*	4.9	7.6	3.5	1.6	1.6	5.0	1.8
2.6	13.9	9.6	20.1	22.4	200.3	166.4	18.1	10.9
-1.6	1.8	13.2	0.8	3.1	2.7	3.1	0.0	7.3
-0.6	12.7	11.0	7.9	8.3	8.5	8.9	7.1	6.1
4.5	8.4	7.7	6.7	7.4	3.8	5.0	10.2	9.1
-1.1	1.6	0.5	15.1	15.1	5.9	5.5	2.3	0.4
5.5	6.0	*9.2*	12.7	24.6	27.9	38.4	3.6	4.5
-0.9	7.5	*22.0*	2.9	6.7	0.9	0.9	0.7	1.6
1.2	10.7	10.2	18.7	14.2	13.6	15.7	3.8	4.7
0.6	5.6	*33.2*	0.7	11.4	6.8	8.1	5.8	12.1
3.4	*23.8*	*23.8*	-3.4	1.5	2.4	2.4	0.0	0.0
-4.6	18.5	15.7	50.9	8.8	1.0	1.3	0.0	0.0
0.4	6.4	11.3	4.7	5.4	10.2	10.7	2.0	7.3
-3.0	10.1	15.0	6.9	8.2	1.3	0.9	3.8	9.0
2.1	8.5	16.6	3.0	7.3	1.7	2.1	4.1	6.3
0.6	7.1	5.8	10.0	6.9	50.4	67.2	1.4	8.5
-0.7	7.4	*21.5*	2.6	5.3	0.9	1.0	1.9	4.8
3.8	9.1	8.6	1.2	2.4	3.4	4.6	6.9	7.0
-3.8	19.3	14.5	19.8	9.7	6.7	6.3	4.7	3.5
-1.3	1.1	4.9	-1.0	-0.6	5.8	8.8	0.0	0.7
1.5	10.5	8.8	10.5	7.4	3.9	3.2	0.0	0.0
-0.1	15.8	7.1	25.8	4.9	2.0	1.6	4.8	1.4
4.3	11.3	12.5	8.4	5.4	8.3	8.5	19.5	18.5
-0.2	5.2	12.7	-0.4	4.7	19.6	23.2	1.1	1.5
-0.0	12.6	12.9	13.5	11.1	11.7	13.5	5.3	5.6
-3.2	0.0	4.6	-2.3	2.3	10.2	11.6	0.0	6.7
2.7	0.0	0.0	13.9	-7.5	33.7	38.5	0.0	0.0
-0.9	8.5	8.2	3.1	5.0	3.2	3.4	1.6	1.7
-0.7	13.4	14.8	11.5	12.0	4.3	5.0	2.2	3.2
-0.3	9.3	9.5	12.5	12.7	5.3	4.6	1.8	1.9
1.8	6.9	5.9	7.4	8.9	7.9	8.5	0.0	0.0
0.6	9.0	3.6	7.5	1.7	6.2	5.2	3.8	1.6
-0.5	5.9	7.2	6.4	6.1	1.6	1.8	0.9	1.4
-0.2	7.3	7.5	0.2	0.2	4.2	4.2	3.8	4.3
1.1	11.0	10.5	15.6	10.9	12.0	11.4	4.6	4.8
-1.0	5.9	10.3	2.1	6.7	6.0	4.9	5.8	2.9
1.0	14.4	7.6	6.0	7.0	2.7	2.1	4.7	2.7
-0.7	9.5	5.0	7.6	2.0	1.6	1.4	4.6	1.3
-0.3	14.4	15.9	5.8	6.0	2.7	3.2	2.2	3.5
1.7	16.7	16.8	13.7	13.5	5.5	5.1	5.6	4.9
1.1	17.1	14.5	13.3	11.3	16.1	14.3	1.3	11.3
-7.6	22.6	21.4	5.9	20.0	1.8	2.5	10.0	8.3

Summary and Conclusions

The U.S. Geological Survey (USGS) conducted a study in cooperation with Maine Department of Transportation (MaineDOT) in which 100 values of maximum abutment scour computed with four prediction methods were compared with maximum observed abutment scour measured at these same sites in the field. Methods tested included the Froehlich/Hire method, the Sturm method, the Maryland Department of Transportation method, and the Melville method. All of the equations from these methods except the Hire equation were developed in the laboratory and have not been tested extensively with field data. The fifth method investigated relations between individual bridge site variables and observed scour in an attempt to create envelope equations. No correlation existed between scour calculated with any of these five methods and scour observed in the field. Maximum abutment scour observed in the field ranged from 0 to 6.8 ft, with an average observed scour of less than 1.0 ft. Equations frequently overpredicted scour by an order of magnitude, and in some cases by two orders of magnitude. All equations also underpredicted scour at 4 to 14 percent of sites.

It may be useful for bridge design purposes to estimate maximum abutment scour at Maine bridges using a single value based on the maximum abutment scour observed at bridges in the current study plus a factor of safety. This conclusion is based on the lack of correlation between predicted and observed abutment scour for the methods tested in this study, the large overpredictions of abutment scour, the less frequent underpredictions of abutment scour, and the relatively small maximum observed abutment scour depths measured in the field.

There are limitations, however, to using a single value based on maximum observed abutment scour depth for bridge design because of the data and assumptions used in this study. One limitation is that the data set is finite. Although 100 abutments at 50 older bridges were examined, and anecdotal evidence by MaineDOT bridge engineers supports a maximum abutment scour depth observed in the field of roughly 7 ft, maximum abutment scour depth could exceed this amount. If historical scour were so great as to require bridge replacement, the maximum scour amounts at these bridges would not have been taken into account in this study. Also, it is possible that infilled scour holes were missed at some bridges.

To use the maximum observed scour depth for bridge design in Maine at a given site, the site should have hydraulic and site characteristics similar to the sites measured in this study. Comparable bridges would have vertical wall abutments and wingwalls; pass over streams with drainage areas between 4 and 100 mi^2; and have bridge openings between 15 and 65 ft. For example, the conclusions from this study may not apply to bridges with spill-through abutments.

An additional limitation of this study is that the hydrologic and hydraulic conditions that caused the observed scour are estimations based on the age of the bridge, peak-flow regression equations, and HEC-RAS step-backwater models. Although the estimated hydrologic and hydraulic data introduce error into the final values of predicted scour, these errors are likely typical of errors encountered by bridge engineers applying the tested methods. The median bridge age of 66 years and the large sample size gives confidence that the conditions observed were typical of those found at this type of bridge in Maine. Furthermore, the recurrence intervals of peak flows occurring at eight continuous-record streamflow-gaging stations in the same region as a few of the bridges examined in this study were as large or larger than estimated peak-flow recurrence intervals. Because observed peak flows were generally greater than estimated peak flows, abutment-scour prediction methods would be expected to underpredict rather than overpredict observed scour.

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Attachment M: Scour/H&H Backup Calculations

CHA

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Keene, NH 03431-4648
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Oxbow Plt 2877
Attachment M: Scour/H&H Backup Calculations

Maine Scour Investigation

COMPLETED BY	DWC	CHA	DATE	4/12/2010
CHECKED BY	BV		PROJ. NO.	82620
PROJ. NAME & LOCATION	Oxbow Plt 2877		SUBJECT	Scour Calcs

	VARIABLE		CALC / NOTES	SOURCE
Contraction Scour Data				
	Q_{100} (cfs)	4285	100-yr Flow through Bridge	FEMA Flood Study
	D_m (ft)	0.20	$D_m = D_{50}$*1.25, D_{50} = 0.16 ft	Field Estimated (Cobbles-Gravel Bed)
	W (ft)	56	Channel Width in Bridge Section	Bridge Plans
	Y_o (ft)	9.5	(100-yr WSE - Avg streambed)	FEMA Flood Study

NHDOT Bridge Scour Evaluations
FHWA HEC-18 Evaluating Scour At Bridges Fourth Edition May 2001
Clear Water Contraction Scour Estimate

COMP'D BY: DWC **DATE:** 04/12/10

CHECK BY: BV **BRIDGE NO.** Oxbow Plt 2877

CHA Proj. No. 82620

HEC-18 Equation for estimating the clear-water contraction scour potential:

$$Y = [Q^2 / (131 D_m^{2/3} W^2)]^{3/7}$$

$$Y_s = Y - Y_o$$

Y = Depth of flow in the contracted section after scour, ft.
Y_0 = Depth of flow in the contracted section prior to scour, ft.
Y_s = Depth of scour, ft.
Q = Flow in the contracted main channel, cfs.
W = Bottom width of the channel at the bridge less pier widths, ft.
D_m = Effective mean diameter of the bed material (1.25 D_{50}), ft.

Compute magnitude of the clear water contraction scour:

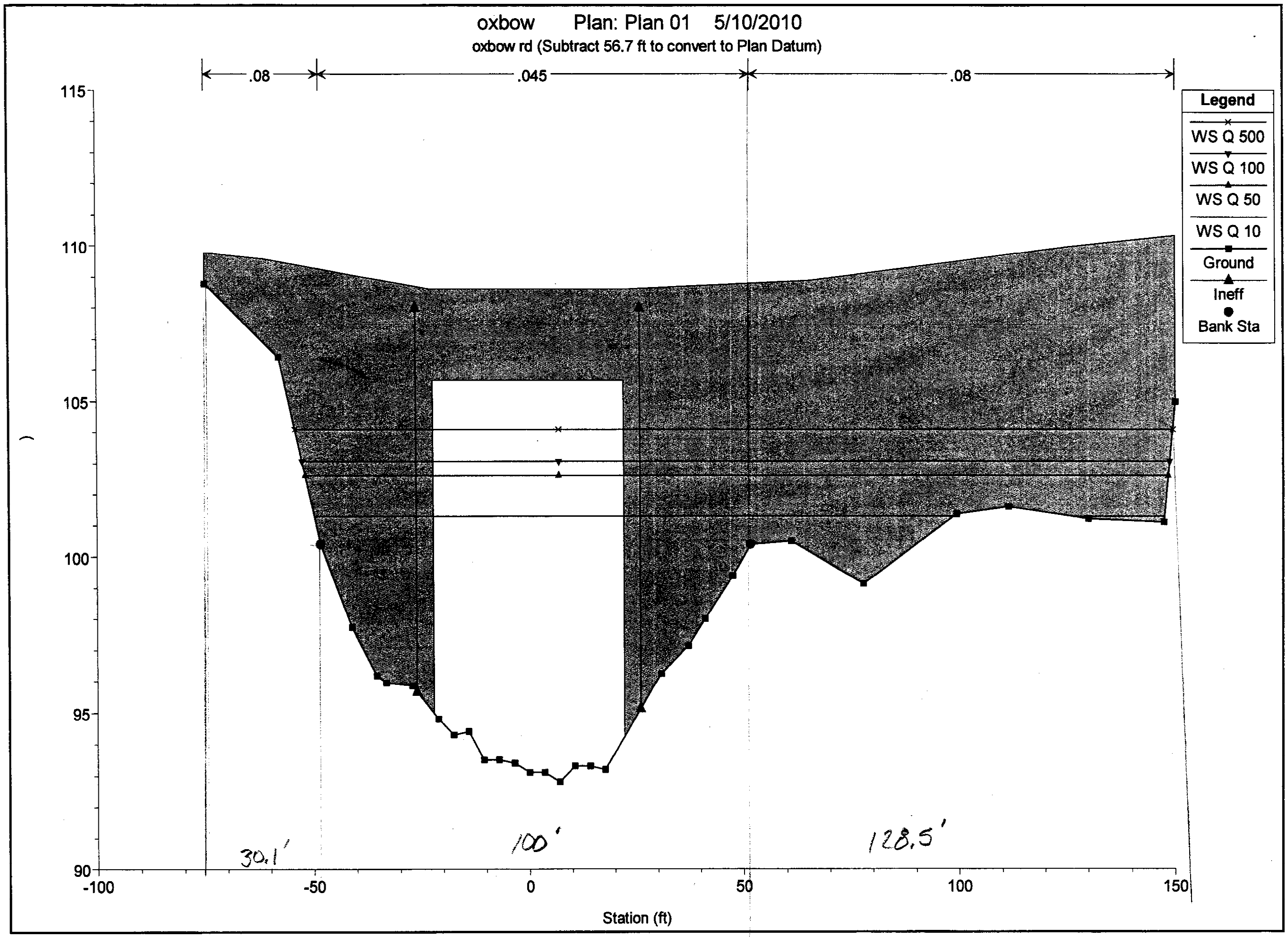
Flood Event (years)	Flow [1] (cfs)	D_m [2] (ft)	W (ft)	Y_0 [3] (ft)	Y (ft)	Y_s (ft)
100	4285	0.20	56	9.5	8.1	**0.0**

(1) Flow referenced from the Aroostook County FEMA FIS.
(2) Mean diameter of the streambed based on the field verified cobble armoring layer.
(3) Average depth of flow in the bridge section is based on computed water surface elevations and field measurements.

NOTES: The above analysis indicates that there is no potential for clear-water contraction scour at the bridge during the 100-year flood event. The effective mean diameter of the channel material reflects the presence of a cobble armoring layer.

HEC-RAS Plan: Plan 01 River: Umcolcus Stream Reach: oxbow rd

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)	
oxbow rd	4	Q 10	1860.00	95.86	101.61	99.00	101.82	0.001673	3.71	558.82	210.53	0.30
oxbow rd	4	Q 50	2560.00	95.86	103.24	99.56	103.41	0.000994	3.50	904.08	214.47	0.25
oxbow rd	4	Q 100	2890.00	95.86	103.82	99.80	104.00	0.000894	3.52	1030.33	215.90	0.24
oxbow rd	4	Q 500	3640.00	95.86	104.93	100.32	105.11	0.000793	3.67	1270.84	218.58	0.23
oxbow rd	3	Q 10	1860.00	92.81	101.42	97.20	101.77	0.001358	4.69	396.88	178.68	0.30
oxbow rd	3	Q 50	2560.00	92.81	102.89	98.01	103.34	0.001432	5.41	473.08	200.74	0.32
oxbow rd	3	Q 100	2890.00	92.81	103.41	98.36	103.92	0.001519	5.78	499.92	201.92	0.33
oxbow rd	3	Q 500	3640.00	92.81	104.39	99.12	105.07	0.001740	6.60	551.21	204.16	0.36
oxbow rd	2.5		Bridge									
oxbow rd	2	Q 10	1860.00	93.91	100.74	98.95	101.60	0.005351	6.88	270.53	268.21	0.53
oxbow rd	2	Q 50	2560.00	93.91	101.47		102.54	0.006554	8.30	308.34	278.05	0.60
oxbow rd	2	Q 100	2890.00	93.91	101.70		102.97	0.007341	9.02	320.52	281.21	0.64
oxbow rd	2	Q 500	3640.00	93.91	102.61	100.87	103.91	0.007373	9.90	367.62	292.26	0.66
oxbow rd	1	Q 10	1860.00	95.58	100.45	99.74	100.69	0.003761	4.75	549.78	281.16	0.44
oxbow rd	1	Q 50	2560.00	95.58	100.93	100.08	101.22	0.003762	5.17	688.41	288.17	0.45
oxbow rd	1	Q 100	2890.00	95.58	101.14	100.23	101.44	0.003761	5.34	748.62	291.17	0.45
oxbow rd	1	Q 500	3640.00	95.58	101.57	100.50	101.91	0.003763	5.69	876.20	297.41	0.46



oxbow Plan: Plan 01 5/10/2010
oxbow rd (Subtract 56.7 ft to convert to Plan Datum)
.08
.045
.08
Legend
WS Q 500
WS Q 100
WS Q 50
WS Q 10
Ground
Ineff
Bank Sta
115
110
105
100
95
90
-100
-50
0
50
100
150
Station (ft)
30.1'
100'
128.5'

Plan: Plan 01 Umcolcus Stream oxbow rd RS: 2.5 Profile: Q 100

E.G. US. (ft)	103.92	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	103.41	E.G. Elev (ft)	103.81	103.26
Q Total (cfs)	2890.00	W.S. Elev (ft)	103.08	101.71
Q Bridge (cfs)	2890.00	Crit W.S. (ft)	98.68	100.27
Q Weir (cfs)		Max Chl Dpth (ft)	10.27	7.80
Weir Sta Lft (ft)		Vel Total (ft/s)	6.88	9.99
Weir Sta Rgt (ft)		Flow Area (sq ft)	419.81	289.18
Weir Submerg		Froude # Chl	0.38	0.63
Weir Max Depth (ft)		Specif Force (cu ft)	2617.85	1858.23
Min El Weir Flow (ft)	108.63	Hydr Depth (ft)	9.50	6.54
Min El Prs (ft)	105.69	W.P. Total (ft)	61.48	57.31
Delta EG (ft)	0.96	Conv. Total (cfs)	49890.7	28091.5
Delta WS (ft)	1.70	Top Width (ft)	44.20	44.20
BR Open Area (sq ft)	465.19	Frctn Loss (ft)	0.15	0.00
BR Open Vel (ft/s)	9.99	C & E Loss (ft)	0.41	0.29
Coef of Q		Shear Total (lb/sq ft)	1.43	3.33
Br Sel Method	Energy only	Power Total (lb/ft s)	9.85	33.32

SPECIAL PROVISION
SECTION 502
STRUCTURAL CONCRETE
(Precast Block Mat)

Add the following to the end of Section 502- Structural Concrete:

Description. This work shall consist of excavating, grading, and placing an articulating concrete block system hereinafter, Precast Block Mat, designated on plans as precast block mat, on designated channels in accordance with these specifications and in reasonably close conformity with the lines, grades and thickness as shown in the plans or as directed by the Resident. The Contractor shall furnish all labor, materials, equipment, and incidentals required to perform all operations in connection with the installation of the Precast Block Mat. This Precast Block Mat system shall be made up of mattresses of concrete blocks and connecting cables placed on top of a geotextile filter layer~~material~~. The Precast Block Mats are made up of precast concrete blocks interlocked by cables cast within each block, forming an articulating concrete block armor layer. Refer to plans for approximate limits required. Multiple irregular mat sizes may be designed for side by side placement and clamped together to provide one homogeneous erosion protection system. An Erosion Control geotextile, suitable as a filter layer, shall be installed on the subbase prior to the installation of the Precast Block Mat.

Design. The Precast Block Mat system shall be comprised of concrete blocks that are wet-cast. The size of the concrete blocks shall be approximately 15.5 inches x 11.5 inches at the base and 4.5 inches thick forming a truncated pyramid shape, or as recommended by the manufacturer. No holes will be allowed in the concrete blocks. The Contractor may submit a site specific design for an alternate size mat. Any alternate design considered shall meet the requirements of the specifications listed herein.

Concrete. The minimum required concrete strength is 4000 psi at 28 days. Air entrainment of 4 percent to 7 percent shall also be added. All applicable ASTM standards will be met in the production of the concrete. The finished concrete product shall consist of a minimum density of 140lbs/cf, in an average of 3 units. No individual block shall consist of a minimum concrete density lower than 135lbs/cf.

Cables. Component cables of the articulating block system shall be constructed of high tenacity, low elongating, and continuous stainless steel strands. Cable shall be integral (cast into) to the concrete block, and shall traverse through each block in both longitudinal and lateral directions of the mat system.

Geotextile. The geotextile used as a filter is to be specified by the manufacturer of the Precast Block Mat. ~~The standard geotextile material used on non specific projects is a~~ The geotextile filter shall be a Class I, non-woven fabric meeting the requirements of Standard Specification 722.03. The geotextile filter~~abric~~ is to be placed on the prepared subbase prior to the installation of the Precast Block Mat. It may be necessary to secure the geotextile to the Precast Block Mat, or to weight down the geotextile outside the limits of the Precast Block Mat to be placed, prior to installation of the Precast Block Mat.

Comment [BR1]: What is meant by "non-specific projects"?

Clamps. Stainless steel wire rope or 3/16" stainless steel U-type clamps shall be used to secure loops of adjoining Precast Block Mats. The standard placement of clamps shall be placed evenly at 4 foot centers interlocking adjoining mats together. Clamps shall be installed as close to the concrete blocks as possible.

Anchoring. Precast Block Mats shall be anchored in accordance with the manufacturer's recommendations. Anchorage shall be provided along the perimeter of the mat system areas. Anchorage of the leading upstream edge and trailing downstream edge of mat area shall be accomplished by complete burial of at least one entire block row. Anchorage of mat systems shall provide for possible traversal by maintenance vehicles and designed to withstand the following flow velocities and shear values:

Flow	Velocity (ft/s)	Shear Stress (psf)
Q500	~~15~~10	~~6.0~~5.0

Ground Preparations. The subbase of the Precast Block Mat area shall be clear of all deformities such as roots, grade stakes and large stones. The entire area shall be smooth so that intimate contact with each individual block can be achieved. To obtain required streambed elevations, clean borrow meeting the requirements of Subsection 703.12, Aggregate for Crushed Stone Surface, may be used as a leveling base. Minor excavation and shaping shall be accomplished to the extent required to remove obstructions, to prepare an optimal contact surface for the mat systems and to place the top of mat systems in a way that conforms to~~with~~ the established streambed elevations. Additionally, the streambed through the bridge site shall be shaped to provide a low flow channel within the stream that will sustain fish passage in low flow conditions. The location of the low flow channel will be determined by the Resident. For a single span bridge, the bottom of the low flow channel shall be three feet wide and two feet lower than established streambed. There shall be a 2:1 slope from the bottom of the low flow channel to the established streambed elevation. Diagram of low flow channel configuration can be seen in Figure 1 below. For a multiple span bridge, the low flow channel only needs to be done for one span as determined by the Resident. Once the streambed/ground preparations are complete and the Contractor can demonstrate the cable mats will be installed at the desired streambed elevations (top and bottom of sag), the streambed/ground preparations shall be approved by the Resident so installation can proceed.

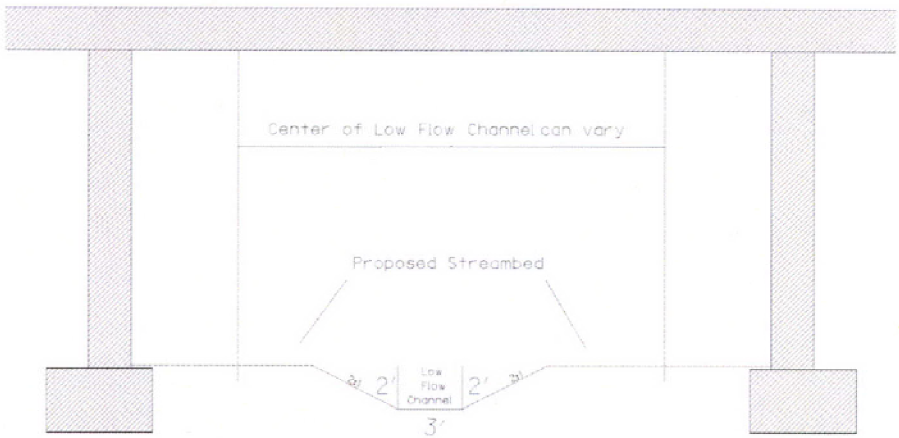


Figure 1.

Installation. Upon installation of geotextile material on the prepared bed surfaces, the installation and placement of the Precast Block Mats shall start at the downstream end of the channel. The Precast Block Mat shall be placed directly on the geotextile. Rips or damage in the geotextile material shall be repaired in accordance with the manufacturer's recommendations. No individual block within the plane of placed articulating concrete block systems shall protrude more than one inch. The Contractor shall ensure that the concrete blocks are flush and develop intimate contact with the subbase.

If assembled and placed as large mattresses, Precast Block Mats shall be attached to a spreader bar or other approved device to aid in the lifting and placing of the mats in their proper position by the use of a crane or other approved equipment. The equipment used should have adequate capacity to place the mats without bumping, dragging, tearing or otherwise damaging the underlying geotextile. The mats shall be placed side-by-side and/or end-to-end, so that the mats adjoin each other. The gaps between each mat and seams between mats shall not be greater than 2 inches, both below and above water. Grouting of seams, openings, or staggered half-blocks will not be allowed. Grouting will only be permitted where the Precast Block Mats are sealed along structures.

Installation of the Precast Block Mats shall be done during low-flow stream conditions and during the in-stream work window.

Anchor trenches and flanking trenches along upstream and downstream terminations shall be backfilled and compacted flush with the top of the blocks. The integrity of the trench backfill must be maintained so as to ensure a surface that is flush with the top surface of the concrete blocks for its entire service life. Backfilling and compaction of trenches shall be completed in a timely fashion.

Once all clamps and anchors have been installed, inspected and accepted, the gaps in the articulating concrete block system shall be partially backfilled from the geotextile material up to the flush surface of the concrete block. For Precast Block Mats within the stream bed, the mats shall be backfilled with replaced streambed material or a suitable alternative approved by the resident.

Precast Block Mat – Concrete Structure Interface. The interface between the Precast Block Mat and any concrete structure, such as an abutment, pier, wing wall, or retaining wall, shall be sealed tightly to prevent streambed fines from migrating up and out from beneath the Precast Block Mat or the concrete wall. The methods listed below are acceptable methods to accomplish this. The Contractor may propose other methods, but must receive approval in writing from the Resident to proceed.

1. Grout Placement. The interface between the Precast Block Mats and the existing structure shall be sealed using 4000 psi concrete or grout. The concrete or grout shall be minimally as thick as the Precast Block Mat and shall completely encapsulate at least two (2) rows of concrete blocks. The entire joint between the Precast Block Mat and structure shall be closed at the face of the structure.

2. Grout Filled Bags. Grout filled bags shall be a minimum of one (1) foot thick, three (3) feet wide, and six (6) feet long and placed directly over the interface of the structure and Precast Block Mat so that the completed position of the grout-filled bag is resting atop the Precast Block Mat and against the structure. The bag shall be made of material meeting the properties of a Class 1 erosion control geotextile and shall be equipped with a self-sealing fill valve. If the bag is longer than twenty (20) feet, a second self-sealing fill valve shall be installed.

   The grout bags shall be filled using 4000 psi concrete or grout as recommended by the manufacturer.

3. Crushed Stone Wrapped Geotextile. The Precast Block Mat shall be placed atop a crushed stone filled geotextile placed at the wall interface. See the attached sketches for placement of the stone and geotextile.

Test Standards and Specifications.

ASTM C31	Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C33	Specifications for Concrete Aggregates
ASTM C39	Compressive Strength of Cylindrical Concrete Specimens
ASTM C42	Obtaining & Testing Drilled Cores and Sawed Beams of Concrete
ASTM C140	Sampling and Test Concrete Masonry Units
ASTM C150	Specification for Portland Cement
ASTM C207	Specification for Hydrated Lime Types
ASTM C618	Specifications for Fly Ash and Raw or Calcined Natural Pozzolans for use in Portland Cement Concrete.
ASTM D18.25.04	Specifications for Articulated Concrete Clock Systems (In Design)
ASTM D698	Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D3786	Hydraulic Burst Strength of Knitted Goods and Non-woven Fabrics
ASTM D4355	Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water
ASTM D4491	Water permeability of Geotextiles by Permittivily
ASTM D4533	Trapezoidal Tearing Strength of Geotextiles
ASTM D4632	Breaking Load and Elongation of Geotextiles (grab Method)
ASTM D4751	Determining Apparent Opening Size of a Geotextile
ASTM D4833	Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
ASTM D5101	Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio
ASTM D5567 Systems	Hydraulic Conductivity Ratio Testing of Soil/Geotextile
ASTM D6684-04	Standard Specification for Materials and Manufacture Articulating Concrete Block (ACB) Revetment Systems
AASHTO T88	Determining the Grain–size Distribution of Soil
AASHTO M288-96	-Standard Specification for Geotextiles
FHWA-RD-89-199	November 1989 Standard Testing for Hydraulic Stability of Concrete Revetment System During Overtopping Flow
~~FHWA-RD-88-181~~	~~Minimizing Embankment Damage During Overtopping Flow (Replace by FHWA-RD-89-199 in November 1989)~~

Quality Control. Units shall be sampled and tested in accordance with ASTM D 6684-04, Standard Specification for Materials and Manufacture of Articulating Concrete Block (ACB) Revetment Systems.

All units shall be sound and free of defects that would interfere with either the proper placement of the unit or impair the performance of the system. Surface cracks incidental to the usual method of manufacture, or surface chipping resulting from the customary methods of handling in shipment and delivery, shall not be deemed grounds for rejection. Chipping resulting in a weight loss exceeding 10 percent of the average weight of a concrete unit shall be deemed grounds for rejection. Blocks rejected prior to delivery from the point of manufacture or at the jobsite shall be repaired with structural grout or replaced at the expense of the Contractor. The Department or their authorized representative shall be accorded proper access to facilities to inspect and sample the units at the place of manufacture from lots ready for delivery.

Field installation procedures shall comply with the procedures utilized during the hydraulic testing procedures of the recommended system. All system restraints and ancillary components shall be employed as they were during testing. For example, if the hydraulic testing installations utilize a drainage layer, then the field installation must utilize a drainage layer; and installation without the drainage layer would not be permitted. In bid preparation, the Contractor must verify with manufacturers whether their proposed systems were hydraulically tested using a drainage layer. If the Contractor elects to use such a system, the required adjustment of the subgrade and all other costs of supplying and installing the drainage layer shall be born exclusively by the Contractor.

cleaned up a bit
July 18, 2013

The theoretical force-balance equation used for performance extrapolation tends for conservative performance values of thicker concrete units based on actual hydraulic testing of thinner units. When establishing performance values of thinner units based on actual hydraulic testing of thicker units, there is a tendency to overestimate the hydraulic performance values of the thinner units. Therefore, all performance extrapolation must be based on actual hydraulic testing of a thinner unit then relating the values to the thicker units in the same family of blocks.

Additional testing, if required, for alternate designs shall be the responsibility of the Contractor.

Hydraulic Testing, Calculations and Submittals. The Contractor shall submit to the Resident all manufacturers' hydraulic testing and calculations in support of the proposed articulated concrete block system and geotextile filter fabric. All calculations submitted must be consistent with the hydraulic details found in the section and stamped by a Professional Engineer licensed in the State of Maine.

The Contractor shall furnish the manufacturer's Certificates of Compliance for Precast Block Mat, revetment cable, and any revetment cable fittings and connectors as specified in this Special Provision. The Contractor shall also furnish the manufacturer's specifications, literature, shop drawings for the layout mats, and any recommendations, if applicable, that are specifically related to the Project. The Contractor shall also submit the proposed method for anchoring the Precast Block Mat, both to the embankments and the streambed/abutments.

Alternative materials may be considered. Such materials must be approved in writing by the Resident. Submittal packages must include, as a minimum, the following:

1. Full-scale laboratory testing and associated engineered calculations quantifying the hydraulic capacity of the proposed Precast Block Mat system in similar conditions to the specific project. Submitted calculations must be PE stamped by a duly licensed Engineer registered to practice in the State of Maine.

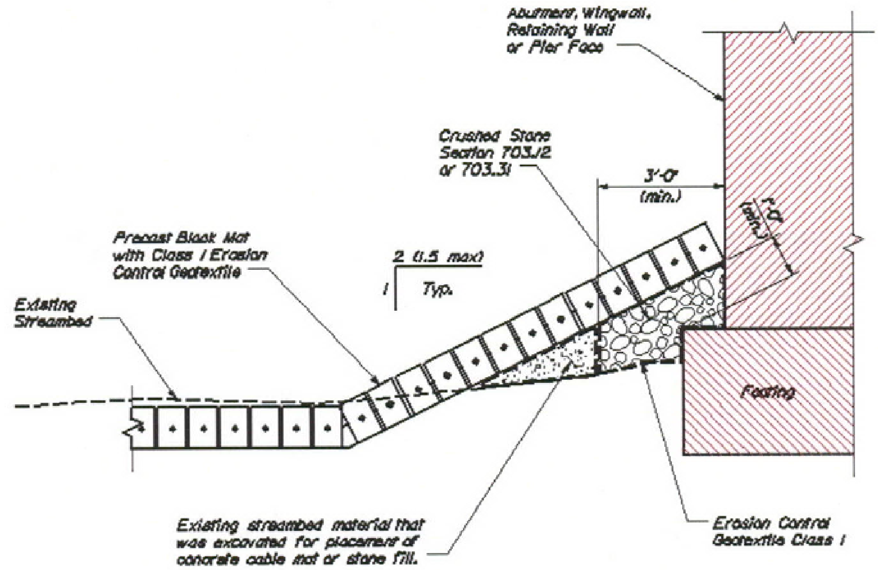
2. A list of five comparable projects, in terms of size and applications, in the United States, where the results of the specific alternate revetment system used can be verified after a minimum of five (5) years of service life.

Method of Measurement. The Precast Block Mat will be measured for payment by the area of articulating block mat system in square feet, accepted and in place.

Basis of Payment. The accepted quantity of Precast Block Mat shall be paid for at the contract unit price. Such payment being full compensation for all labor, materials, equipment, Quality Control, submittals, testing and incidentals necessary to complete the work as specified including, but not limited to, ground preparation, Precast Block Mats, geotextile, anchors, clamps, grouting, grout bags, stone fill, and backfill.

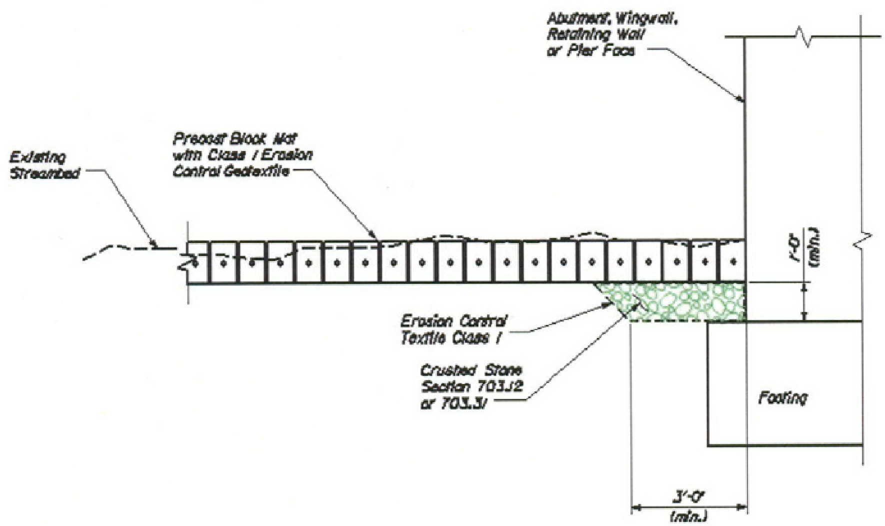
Payment will be made under:

Pay Item	Pay Unit
502.83 Precast Block Mat	Square Foot



PRECAST BLOCK MAT DETAIL ~ OPTION I

Typ. section at face of Abutment, Wingwall, Retaining Wall or Pier when the streambed is at or below the top of the footing



PRECAST BLOCK MAT DETAIL ~ OPTION 2

Typ. section at face of Abutment, Wingwall, Retaining Wall or Pier when the streambed is above the top of the footing