Countermeasures Analysis for an Existing Scour Critical Bridge (Bridge # 1200) Over Río Grande de Loíza at Road PR-3 Carolina, Puerto Rico

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Abstract *— Scour is the most common reason for the failure of highway bridges in the United States. In Puerto Rico, there are more than 350 scour critical bridges. Bridge 1200 has been classified as a scour critical bridge by the Puerto Rico Highway and Transportation Authority (PRHTA). Scour along the abutments and piers supports is the key parameter to calculate the appropriate protection for the bridge piers and abutments. Models used for analysis were: ArcMap (GIS), HEC-HMS, HEC-RAS 4.1.0, HEC-18 and HEC-23. The investigation has shown contraction scour depths up to 1.5m and pier scour depths up to 5.27 m for the 500yr event. Abutment scour depths up to 2.32 m were found for pressure flow condition. Due to its proven performance and availability, rock riprap was recommended as a scour countermeasure. Sizes up to 9in (60 pounds) for abutments and 24 in (1/2 ton.) were calculated.*

Key Terms *— HEC-HMS and HEC-RAS Analysis, Scour Countermeasures, Scour Critical Bridge, Total Scour.*

INTRODUCTION

Bridge (BR) 1200 is an existing PRHTA's bridge over Rio Grande de Loíza constructed in 1969 and reconstructed in 1984 (see Figure 1).

The Bridge is located in the Road PR-3 in the town of Carolina, Puerto Rico. Its average daily traffic is 126,000 vehicles from which 4% are trucks. The bridge has eight spans with an average mean length of 35 meters (7 piers and 2 abutments), a total length of 239.8 m and a deck width of 30.6 m.

Río Grande de Loíza's watershed is the largest in Puerto Rico and its drainage basin includes the towns of San Lorenzo, Juncos, Las Piedras, Gurabo, Caguas, Aguas Buenas, Trujillo Alto, Loíza, and Carolina. The main water bodies of the watershed considered in the hydrology study as part of the analysis are: Río Cayaguas, Río Emajagua, Río Canas, Río Turabo, Río Caguitas, Río Gurabo, Río Blanco, Quebrada Grande, Quebrada Maracuto, Río Canovanillas, Río Canóvanas, Río Grande de Loíza, Río Valenciano and Río Bairoa (see Figure 6).

The hydraulic analysis was made by using the mathematical model HEC-RAS (River Analysis System) version 4.1.0 developed by USCOE.

Figure 1
BR-1200 (1969)

HEC-RAS is an integrated system of software, designed for interactive use in multi-tasking environment [1]. After performing the water surface profile calculations for the design events of 100- and 500- year flood, the bridge scour was evaluated. Also, the scour was calculated under pressure flow. This occurs when the flow comes into contact with the low chord of the bridge [2]. Calculation of basin area and parameters, 24hrs. hydrographs, WSE, distribution of channel velocities, scour effects, type and size of countermeasure and the collection of meteorological data are some of the tasks performed in this work.

JUSTIFICATION

Under the 2005 National Bridge Inspection Standards (NBIS) bridge owners, like PRHTA, are required to develop a Plan of Action (POA's) for scour critical bridges and monitor those bridges in accordance with the plan [3]. A scour POA is a comprehensive document developed for each scour critical bridge which provides a single source of information pertaining to scour inspection, flood monitoring, and a schedule of the countermeasures that have been recommended by bridge inspectors to protect a bridge from scour and stream stability problems [4].

The POA establishes a systematic process of monitoring and closing bridges to ensure public safety during a significant flood event and criteria for inspection and re-opening after a flood event. It also assists bridge owners to program and prioritize the installation of scour countermeasures to protect scour critical bridges from flood damage [4].

As part of the POA the owner is requested to access the scour condition of the bridge by means of visual inspection or by the necessary HH and Scour studies.

This study is not only a federal requirement; it is a safety concern for the people of Puerto Rico that uses the PR-3. Our commitment with the island and our desire to contribute to the wellness and safety of our people encourage us to conduct this study.

LITERATURE REVIEW

There are four main processes that must be studied in depth to access the proper solution of the scour problems of Bridge 1200. These are; Hydrology, Hydraulics, Scour and Scour Countermeasures.

Hydrology is the science that deals with the occurrence and distribution of naturally occurring water on, around, and under the Earth's surface [5] (see Figure 2).

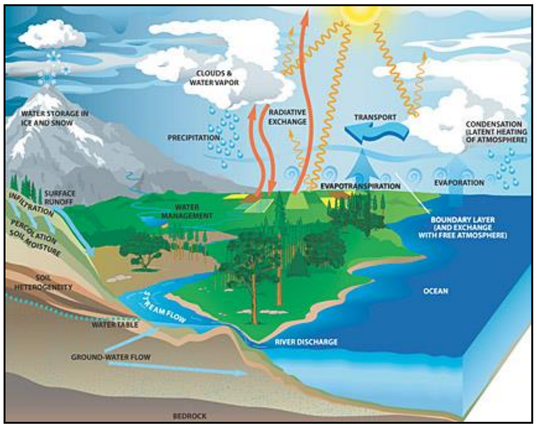


Figure 2
Hydrologic Cycle

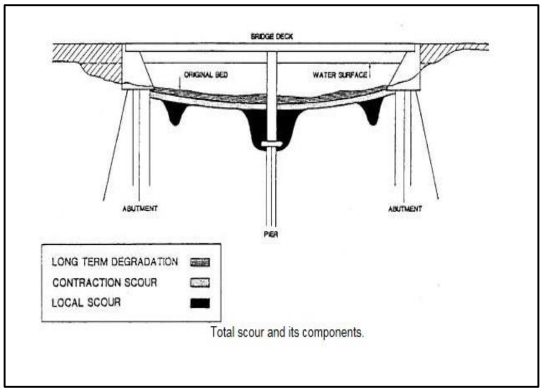
Hydraulics of rivers, often called free surface hydraulics, is the branch of hydraulics dealing with free surface flow, such as those occurring in rivers, canals, lakes, estuaries and seas. Fluid mechanics provides the theoretical foundation for hydraulics, which focuses on the engineering uses of fluid properties [6]. As with the hydrology, the advances in computer methods and the extensive collection of empirical data have allowed the development and application of simulation models.

Important data like the water surface elevations, the distribution of channel velocities associated with the 24 hours storm for the recurrences of 100 and 500 years and the associated pressure flow parameters will be supplied by the hydraulic analysis performed with the hydraulic model HEC-RAS.

Scours is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges [7].

Total scour in a bridge is composed of; long term degradation, local scour and (general) contraction scour (See Figures 3) [7].

Long term degradation involves the lowering or scouring of the streambed over relatively long reaches due to a deficit in sediment supply from upstream [7].



Total scour and its components.

Figure 3
Bridge Total Scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by obstructions to the flow (See Figure 4 and Figure 5) [7].

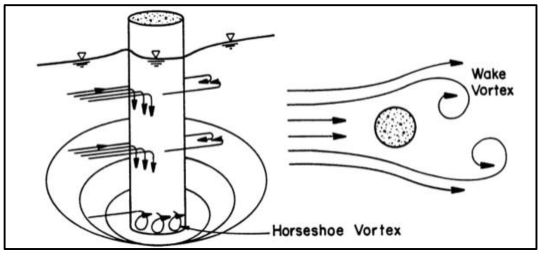


Figure 4
Pier Scour

Contraction scour is a lowering of the streambed across the stream or waterway bed at the bridge caused by the contraction (or constriction) of the flow, which results in removal of material from the bed across all or most of the channel width. Occurs in the vicinity of the constriction or bridge, may be cyclic and/or related to the passing of a flood [7].

Scour Countermeasures are defined as measures incorporated into a highway-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream instability and bridge scour problems [7]. Riprap is the most widely used scour countermeasure in the United States. Its effectiveness has been well established where it is of adequate size, of suitable size gradation, and properly installed [8]**.** Riprap consists of a layer or facing of rock, dumped or hand-placed on channel and structure boundaries to limit the effects of erosion. It is the most common type of countermeasure due to its general availability, ease of installation and relatively low cost [8].

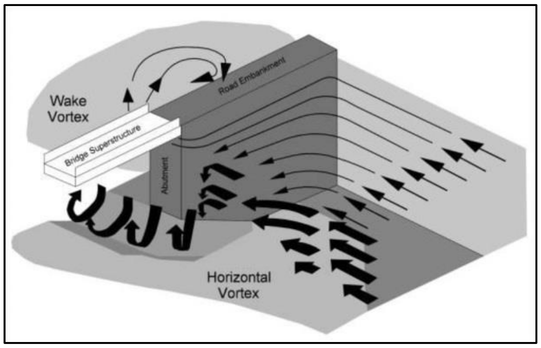


Figure 5
Abutment Scour

Riprap particle erosion is primarily limited by sizing the riprap to withstand hydraulic and turbulence forces, but is also be affected by riprap slope, impact and abrasion, ice, waves and vandalism [7].

METHODOLOGY

The following information explains the methodology used in this project.

Hydrologic Calculations

Watershed delineation is the first step in the hydrologic analysis. The Río Grande de Loiza watershed is the largest in Puerto Rico and its drainage basin includes several towns and water bodies as described in our introduction. The drainage area of the Río Grande de Loíza at highway PR-3 bridge in Carolina is 627.3 square kilometers (see Figure 6).

Lago Loíza had an original storage capacity of 26.80 million m^3 of water at an elevation of 41.14 m above mean sea level for the Sergio Cuevas Filtration Plant, serving the San Juan metropolitan area.

The impoundment at Loíza Dam was built for water supply and hydroelectric generation, not as a flood protection measure. The reducing effect of the reservoir on flood flows through the dam is relatively small. Approximately 1,360 hectare-meters are available for flood storage. This represents approximately 0.03 meter of runoff from the contributing drainage area of 536 square kilometers [9].

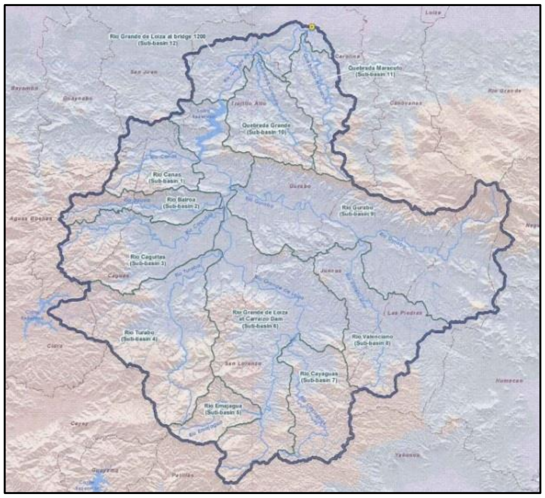


Figure 6
BR-1200 Watershed

The watershed tributary to the study reach was divided into twelve sub-basins, delimited using the USGS topographic quadrangle and GIS. The subbasin drainage areas are shown in Table 1.

Table 1
Sub-Basins Drainage Areas

Sub-Basins	**Areas (km^2)**
QuebradaMaracuto	26.70
Quebrada Grande	34.03
R. Grande de Loiza (at CarraizoDam)	131.70
Rio Canas	27.19
Rio Gurabo	131.57
Rio Bairoa	20.25
Rio Caguitas	48.64
Rio Valenciano	49.37
Rio Turabo	76.92
Rio Cayaguas	36.21
Rio Emajagua	15.98
Rio Grande de Loiza at mouth	28.96

The NRCS method of runoff estimation involves the computation of a runoff curve number (CN). This number corresponds to soil-cover and land use conditions of the area. The major factors in determining CN are the hydrologic soil group, land cover type, treatment, hydrologic condition, and antecedent moisture condition. Values of CN were obtained from tables prepared by the NRCS and were weighted according to the soil type and land use area percentage on each sub-basin. Soil characteristics of the study area were identified by means of soil maps [10]. The slopes and the calculated curve numbers for each sub-basin are presented in Table-2.

Table 2
CN and Slopes for Sub-Basins

Sub-Basins	**CN**	**Slope%**
QuebradaMaracuto	81.30	29.14
Quebrada Grande	81.97	31.53
R. Grande de Loiza (at CarraizoDam)	80.73	26.51
Rio Canas	77.20	27.86
Rio Gurabo	79.17	23.67
Rio Bairoa	80.47	26.63
Rio Caguitas	82.51	23.56
Rio Valenciano	79.26	19.84
Rio Turabo	76.88	31.52
Rio Cayaguas	72.87	29.35
Rio Emajagua	73.70	37.51
Rio Grande de Loiza at BR-1200	84.80	18.91

The time of concentration (T) is the time it takes for runoff to travel from the hydraulically most distant point of the basin to a point of interest within the basin. Water moves through a basin as sheet flow, shallow concentrated flow, open channel flow, or some combination of these [11].

The lag time is an estimation of the time between the centroid of precipitation mass and the peak flow of the discharge hydrograph and it is commonly estimated as the 60% of the time of concentration [11].

Lag times were estimated using the SCS method as defined by (1).

$$\frac{L^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} * 60 \tag{1}$$

Where:

t_{Lag} = lag time (minutes)

L = hydraulic length (ft)

$S = 1000/CN - 10$

Y = average basin slope (ft/ft)

Time concentration (Tc) is defined as time lag (T_L) x 1.67 [11]. Table 3 shows T_C and T_L for the different sub-basins.

Table 3
Time of Concentration (T_C) and Time Lag (T_L)

Sub-Basins	**Longest Flow Path (ft)**	**T_C (min)**	**T_L (min)**
QuebradaMaracuto	11.3	119.25	71.55
Quebrada Grande	7.99	88.61	53.16
R. G. de Loiza (atCarraizoDam)	27.17	243.08	145.85
Rio Canas	6.42	78.54	47.12
Rio Gurabo	20.79	206.62	123.97
Rio Bairoa	10.4	115.85	69.51
Rio Caguitas	12.7	141.64	85.0
Rio Valenciano	10.4	129.78	77.87
Rio Turabo	13.16	130.16	78.09
Rio Cayaguas	12.04	124.95	74.97
Rio Emajagua	5.53	62.45	37.47
R. G. de Loiza at BR-1200	8.23	110.41	66.24

The peak discharges for 100 and 500 year's recurrence storms were obtained using the HEC-HMS software. The schematic diagram for the hydrologic model is shown in Figure 7. For both simulations (100 and 500 years) assumptions, based on available data and engineering judgment, were incorporated in the HEC-HMS model.

Hydraulic Calculations

The software HEC-RAS from the USACE was used in the hydraulic analysis. The model accepts changes in the geometry of the water course, bed and overbank friction coefficients and shapes of hydraulic structures. The friction's coefficients (Manning's Coefficient) used in the modeling were obtained from visual inspection of the stream bed and channel banks and compared with the values provided by FIS 2009, rev. 2012.

The reach cross sections, peak flows (100yr, 500yr and pressure flow), boundary conditions and estimated roughness coefficients (Manning's n) were some of the data used as input of the hydraulic model. The water surface elevation (WSE) and flow velocities for the 100 and 500 year return period and the pressure flow were computed. Cross sections were provided by PRHTA.

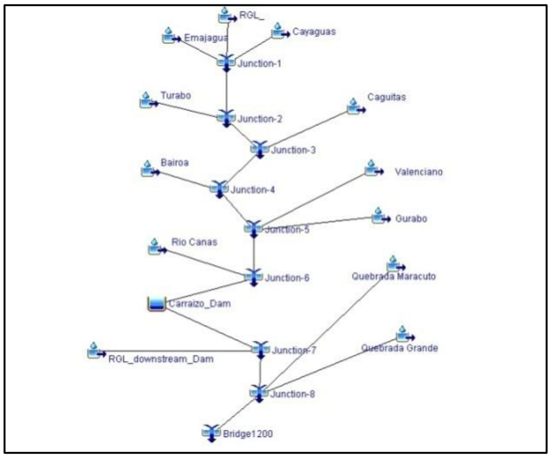


Figure 7
HEC-HMS Schematic Diagram

FEMA flood profiles of Río Grande de Loiza reach 1 for 1.0 and 0.2 percent annual chance floods [9] were used to calibrate the model for the 100 yrs. and 500 yrs. floods respectively.

Scour Calculations

Bridge 1200 scour was calculated using a comprehensive analysis of all the factors that affect it and the equations that apply as established in the scour guidelines of HEC-18 [7].

HEC-RAS model does not have the capability to perform long term aggradation or degradation calculations. Information about long term aggradation or degradation can be obtained from historic and current streambed elevations. Historic inspection records of bridge BR-1200 [11] were accessed to estimate this component of scour.

Contraction Scour can occurs in two forms depending on how much bed material is transported upstream of the bridge contraction reach. Live-bed contraction scour occurs when there is transport of bed material in the approach reach and Clear-water contraction scour when there is not such transport. Bridge 1200's contraction scour analysis resulted in the use of both equations. Left overbank and channel scour were calculated as live bed scour while right over bank scour result clear water scour.

HEC-RAS model does not have the capability to perform scour calculation in complex pier foundations. Complex pier scour is computed if the foundation of the pier is wider than the pier and it is exposed to the flow. Piers 3, 4, 5 and 6 of Bridge 1200 were calculated using this method. The scour of each component of the pier was added as shown in Figure 8.

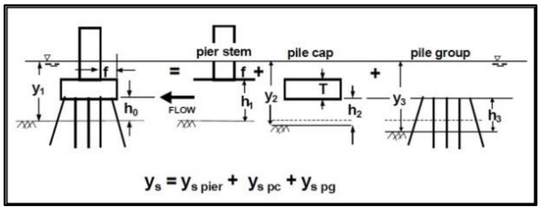


Figure 8
Complex Pier Scour Components

The scour components were computed from the basic pier scour equation using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities, and height adjustments for the pier stem and pile group [7]. Then components were added as shown by equation (2).

$$y_s = y_{SPier} + y_{S\,PC} + y_{S\,PG} \qquad (2)$$

Where scour components are;
$y_{S\,Pier}$ = For the pier stem (ft or m)
$y_{S\,PC}$ = For the pier cap or footing (ft or m)
$y_{S\,PG}$ = For exposed piles (ft or m)

Two equations are recommended for the computation of live-bed abutment scour: the HIRE equation and the Froehlich equation. HIRE equation must be used when the wetted embankment length divided by the approach flow depth is greater than 25. When this ratio is less than or equal to 25 Froehlich's should be used. [7]. Abutments scour computations for Bridge 1200's were automatically generated by HEC-RAS program. For both abutments the model determined to use the Froehlich equation. The left and right abutments of the Bridge 1200 would suffer scour under the 500 year flood event and the pressure flow conditions but not under the 100 year condition.

D_{50} and other information about the bed material for scour calculation were obtained from a soil study, provided by PRHTA [12].

Scour Countermeasures

Scour countermeasures are features generally incorporated after the initial construction of a bridge to make it less vulnerable to damage or failure from scour. Literature describes several scour counter measures [13]. Riprap, Gabions and Toscanes were evaluated as possible scour's countermeasures for the Bridge 1200. However, due to its performance and availability, rock riprap was selected as scour countermeasure for the piers and abutments of Bridge 1200.

Equation (3) was used for the sizing of the rock riprap at vertical wall abutments of the Bridge 1200 (Froude Numbers ≤ 0.80) and equation (4) was used for the sizing of the rock riprap in the piers. The values used to determine the median stone diameter in equations (3) were obtained from the hydraulic results at the approach section of the bridges.

$$D_{50} = \frac{yK}{(S_S-1)}\left[\frac{V^2}{gy}\right] \qquad (3)$$

Where:
D_{50} = Median stone diameter (m)
V = Characteristic average velocity in the contracted section (m/s)
S = Specific gravity of rock riprap (normally 2.65)
g = Gravitational acceleration (9.81 m/s^2)
y = Depth of flow in the contracted bridge opening (m)
K = 1.02 for a vertical wall abutment

$$D_{50} = \frac{0.692(KV)^2}{(S_s-1)2g} \qquad (4)$$

Where:
D_{50} = Median stone diameter (m)
V = Velocity on pier (m/s)
S_S = Specific gravity of riprap (normally 2.65)
g = Gravitational acceleration (9.81 m/s)
K = 1.5 for round nose pier 2

RESULTS

A summary of the results from the different models, equations and methodologies used in this project are shown in the following tables and figures.

Hydrologic Results

Peak discharge estimates were obtained using the HEC-HMS model (see Table 4). Figure 9, shows an example of the hydrograph generated by the model for the 500 yrs. storm. To ensure that the 100 and 500 years peak flow estimate by HEC-HMS represents the hydrologic existing conditions at the Bridge 1200 a comparison of the estimates with available historic data and/or studies was conducted.

Table 4
Results HMS Simulation

Sub-Basins	Areas	Discharge (cms)	
	(km^2)	100 yrs	500yrs
RGL	50.85	1,188.63	1840.13
Cayaguas	13.98	425.73	629.98
Emajagua	6.17	230.07	319.99
Junction-1	71.00	1,609.28	2,506.53
Turabo	29.7	929.04	1,323.27
Junction-2	100.70	2,491.69	3,789.44
Caguitas	18.78	570.32	804.56
Junction-3	119.48	3,059.80	4,592.97
Bairoa	7.82	250.69	353.58
Junction-4	127.30	3,300.62	4,936.18
Gurabo	50.86	1,263.48	1,923.87
Valenciano	19.06	569.27	833.40
Junction-5	197.22	5,036.31	7,583.35
Canas	10.5	348.29	514.84
Junction-6	207.72	5,319.73	7,998.68
CarraizoDam	207.72	5,577.05	8,597.04
RGL upstream	11.18	357.25	511.11
Junction-7	218.9	5,917.82	9,106.31
Quebrada Grande	13.14	453.97	629.64
QuebradaMaracuto	10.31	322.69	469.66
Junction-8	242.35	7,031.86	10,179.04
Bridge 1200	242.35	7,031.86	10,179.04

The peak discharges obtained by HEC-HMS were compared with those in a FEMA previous study of the basin [9] and with the estimates resulting from a frequency analysis of USGS Station 50059050 [14]. Tables 5 and 6 show the comparison for the 100 and the 500 years events. Table 5 also contain the location of the three peak flow values of the FIS (2009, revised 2012) used to calibrate the peak flow values in HMS for the 100 and 500 years events.

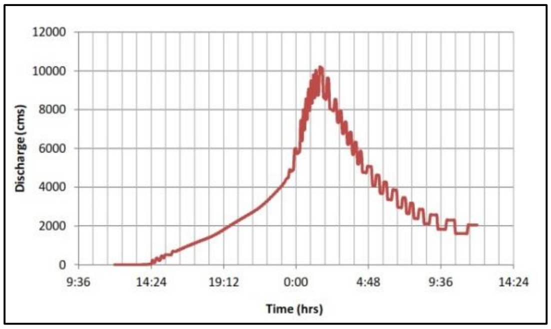


Figure 9
500Yrs. Discharge Hydrograph

Table 5
Peak Discharges 100 Yrs. Floods Comparison

Location	HEC-HMS (cms)	FEMA (cms)	Bulletin 17B (cms)
US Conf. with Bairoa R.	3,059.80	3,522	-----
US Conf. with Gurabo R.	3,300.62	3,712	-----
USGS Station 50059050	5,911.24	6,004	4,865.5

The 100 and 500 year peak discharges values, obtained from the HEC-HMS, and shown in the last row of Table 4, were chosen to be used in the hydraulic model. They are the largest, between the three sets of values (see Table 5 and 6) and represent the exiting conditions of the basins.

Table 6
Peak Discharges 500 Yrs. Floods Comparison

Location	HEC-HMS (cms)	FEMA (cms)	Bulletin 17B (cms)
US Conf. with Bairoa R.	4,592.97	4,936	-----
US Conf. with Gurabo R.	4936.18	5,216	-----
USGS Station 50059050	8597.34	8,913	5,308

Hydraulic Results

HEC-RAS results are used as an input in the scour and countermeasures equations. Table 7 shows some results from the hydraulic model.

Table 7
Hec - Ras Hydraulic Parameters

Location	100Yrs. Flow	500Yrs. Flow	Pressure Flow
WSE (m)	13.13	13.96	15.46
Vel. in Left Abut. (m/s)	0.49	0.85	0.91
Vel. in Right Abut. (m/s)	0.00	0.40	0.43
Vel. Pier 1 (m/s)	0.79	1.08	1.06
Vel. Pier 2 (m/s)	0.90	1.18	1.16
Vel. Pier 3 (m/s)	0.92	1.21	1.18
Vel. Pier 4 (m/s)	1.88	2.48	2.43
Vel. Pier 5 (m/s)	2.01	2.67	2.60
Vel. Pier 6 (m/s)	2.04	2.69	2.62
Vel. Pier 7 (m/s)	1.81	2.38	2.33

Figure 10 shows the water surface elevation (WSE) profiles for the 100 years, 500 years and Pressure Flow events at Bridge 1200, generated by the hydraulic model HEC-RAS.

Figure 11 shows a schematic representation of the flow velocity distribution tubes in the upstream, inside, cross section of the bridge 1200, also generated by the hydraulic model HEC-RAS.

Scour Results

Hydraulic study results generated by HEC-RAS were used as input for the contraction, pier and abutments scour analysis.

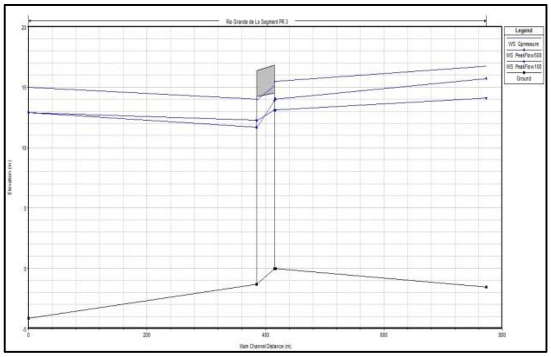


Figure 10
Bridge 1200 - HEC-RAS Water Surface Elevations Profiles

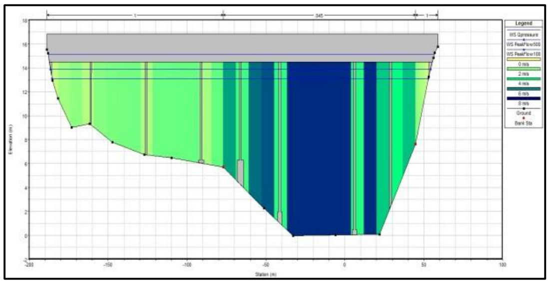


Figure 11
Bridge 1200 - HEC RAS Inside Upstream Cross Section Flow Velocity Distribution

Table 8 shows the contraction scour for 100 yrs, 500 yrs and Pressure Flow.

Table 8
Contraction Scour for 100 yrs., 500 yrs. and Pressure Flow

Flows	Depth of Scour (m)		
	Left	Channel	Right
Q peak 100	1.48	0.82	0.00
Q peak 500	1.50	0.70	1.38
Q Pressure	0	0	0

Table 9 shows the piers affected by the contraction scour.

Table 9
Computed Contraction Scour for the Piers

Pier ID	Qp 100Yrs.	Qp 500Yrs.	Qpressure
Pier 1 (m)	1.48	1.5	0.0
Pier 2 (m)	1.48	1.5	0.0
Pier 3 (m)	0.82	0.7	0.0
Pier 4 (m)	0.82	0.7	0.0
Pier 5 (m)	0.82	0.7	0.0
Pier 6 (m)	0.82	0.7	0.0
Pier 7 (m)	0	1.38	0.0

Table 10 shows the local pier scour results to be expected at bridge piers (looking downstream) for the 100 and 500 year flood event and pressure flow.

Table 10
Local Pier Scour Results

Pier ID	Qp 100Yrs.	Qp 500Yrs.	Qpressure
Pier 1 (m)	2.99	3.25	3.67
Pier 2 (m)	3.40	3.77	3.46
Pier 3 (m)	1.39	1.57	1.41
Pier 4 (m)	2.19	2.35	2.79

Pier 5 (m)	2.20	2.42	2.31
Pier 6 (m)	1.91	2.02	1.92
Pier 7 (m)	2.02	2.36	2.27

Table 11 shows the total pier scour results and Table 12 shows the abutment scour for the analyzed flows.

Table 11
Total Pier Scour Results

Pier ID	Qp 100Yrs.	Qp 500Yrs.	Qpressure
Pier 1 (m)	4.47	4.75	3.67
Pier 2 (m)	4.88	5.27	3.46
Pier 3 (m)	2.21	2.27	1.41
Pier 4 (m)	3.01	3.05	2.79
Pier 5 (m)	3.02	3.12	2.31
Pier 6 (m)	2.73	2.72	1.92
Pier 7 (m)	2.02	3.74	2.27

Table 12
Abutments Scour Results

Flows	Depth of Scour (m)	
	Left	Right
Qp 100Yrs.	0.00	0.00
Qp 500Yrs.	2.07	0.90
Qpressure	2.29	2.32

The total scour for a bridge is a combination of the contraction scour, pier scour and abutment scour. Figure 12 is a Hec Ras schematic diagram of the scour in Bridge 1200 for the 500yr condition. The scour of piers 3, 4, 5 and 6 were calculated by the complex pier scour method, not included in the Hec Ras model.

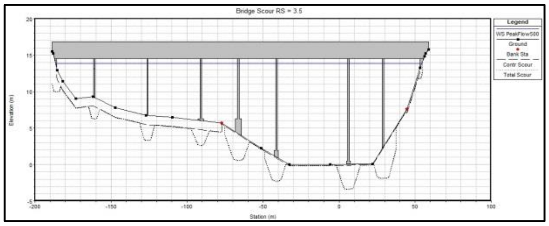


Figure 12
500 yrs. HecRas Bridge Scour Diagram

Countermeasure Results

Table 13 and Table 14 show the summary of the riprap countermeasure for piers and abutments.

Table 13
Nominal Riprap Class for Abutments

Abutments	Median Particle Size	Size (in)	Mat Width (m)	Thickness (m)	Weight (pounds)
Left	I	6	12.26	0.07	20
Right	II	9	15.04	0.24	60

Table 14
Nominal Riprap Class for Piers

Pier ID	Median Particle Size	Size (in)	Mat Width (m)	Thickness (m)	Weight
Pier 1	II	9	1.80	2.25	60 pnds
Pier 2	II	9	1.80	2.25	60 pnds
Pier 3	II	9	1.80	2.25	60 pnds
Pier 4	VI	21	1.80	3.66	3/8 ton
Pier 5	VII	24	1.80	4.36	1/2 ton
Pier 6	VII	24	1.80	4.50	1/2 ton
Pier 7	V	18	1.80	3.57	1/4 ton

Figure 13 shows a typical section for riprap around piles.

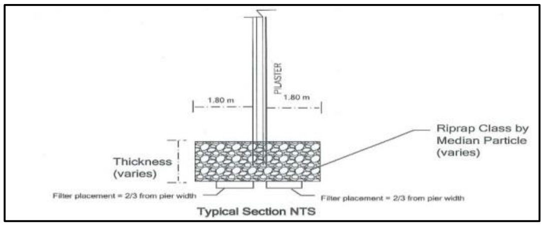


Figure 13
Typical Section for Riprap around Piles

CONCLUSIONS AND RECOMMENDATIONS

Total scour values piers and abutments were critical for the 500 yrs. and pressure flow scenarios. The higher scour value shifts between these two conditions for the different piers but stay at the pressure flow condition for the abutments. Worst case scenario was assumed for the countermeasure analysis and design.

The designed countermeasures will not change significantly the water levels at Bridge 1200 as shown in Table 15.

Table 15
Original and Modified WSE at Bridge 1200

Flow	Original	Modified	Difference
Q100	15.15	15.08	- 0.08
Q500	13.87	14.05	+ 0.18
Pressure Flow	13.12	13.05	- 0.07

Velocity values at abutments will decrease after the placement of the countermeasures, as can be observed in Table 16, except for the left abutment under the pressure flow condition. The velocity at the piers remains the same after the riprap placement.

Table 16
Original and Modified Velocity (m/s) at Abutments

Flow	Original 500 yrs.	Modified 500yrs.	Original Pressure Flow	Modified Pressure Flow
Left	0.90	0.84	0.84	0.93
Right	1.69	1.50	1.69	1.41

Tables 17 and 18, respectively, show the simulated contraction and piers scour values, before and after the implementation of the countermeasures.

Table 17
Original and Modified Calculated Contraction Scour (m) for Qpeak 500 yrs.

500 yrs. Flow	Left	Channel	Right
Original	1.50	0.70	1.38
Modified	1.05	0.03	0.64

Table 18
Original and Modified Calculated Total Pier Scour (m)

ID	Original	Modified
Pier 1	4.75	1.84
Pier 2	5.27	2.14
Pier 3	2.27	0.88
Pier 4	3.05	1.31
Pier 5	3.12	1.51
Pier 6	2.72	1.60
Pier 7	3.74	3.74

Abutments scour turns to be the same before and after the placement of the riprap revetment. The contraction scour will decrease at right, center and left channel. The total pile scour will decrease for all of the piers, except for pier #7, which scour remain the same. Some recommendations for the riprap placement and treatment can be found in the HEC-23 [13].

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