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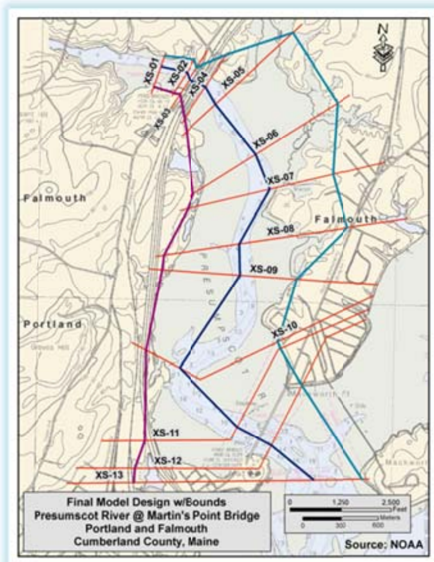
MaineDOT

Transportation Research Division



Technical Report 14-05

Cost-Efficient and Storm Surge-sensitive Bridge Design for Coastal Maine

Final Report – August 2013

Technical Report Documentation Page

1. Report No. ME 14-05	2.	3. Recipient's Accession No.
4. Title and Subtitle Cost-Efficient and Storm Surge-sensitive Bridge Design for Coastal Maine		5. Report Date August 2013
		6.
7. Author(s) Ellen Douglas, Paul Kirshen, and Sam Merrill		8. Performing Organization Report No.
9. Performing Organization Name and Address New England Environmental Finance Center (EFC) Muskie School of Public Service University of Southern Maine		10. Project/Task/Work Unit No.
		11. Contract © or Grant (G) No.
12. Sponsoring Organization Name and Address Maine Department of Transportation 16 State House Station Augusta, Maine 04333		13. Type of Report and Period Covered
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract (Limit 200 words)

Climatic variation felt through changing weather patterns is having increasingly acute effects on Maine's transportation infrastructure. Acute risk occurs as a result of events, such as storms and flooding, while chronic risk surrounds longer range changes due to climate over time. It is acute risk that results in an increased need for disaster designation and response, increased risk of collateral property damage, and threats to safety of the traveling public. This type of risk can be mitigated by early preparation. Because of uncertainty about the future of climate and weather variability, chronic risk is much harder to gauge. This uncertainty can create paralysis in an agency charged with making and justifying long-term, fiscally-responsible decisions around the safety and efficiency of public travel. But long lifecycles of most transportation infrastructure demand adaptation via early preparation to protect significant taxpayer investments into the reasonably foreseeable future. This research effort builds on cost modeling developed to estimate fiscal impacts of related state legislation and develops an approach to estimate acute and chronic cost/risk tradeoffs for a subset of transportation project needs along the Maine coast.

The general goal is to address cost and risk issues associated with projected sea level rise (SLR) and threats to state infrastructure elements that are also designed to pass freshwater streams and rivers. More specifically, this project aims to help develop storm surge-sensitive design standards and approaches for large, tidally influenced transportation structures along the Maine coast. For each of the two major tidal regimes along the coast, the project aims to 1) obtain and analyze surge frequency from an existing tide gauge; 2) obtain and analyze flood flows from the corresponding stream flow gauge at the site (if none available, use regional hydrologic analysis to develop) and develop a joint probability distribution with coastal storm surges; 3) focus on one type of structure (bridges); and 4) identify installation costs versus damage risk-tradeoffs under different SLR, storm surge, and river flood scenarios extending out over the useful life of the asset.

17. Document Analysis/Descriptors Climate, sea level rise, storm surge, bridge hydrology		18. Availability Statement	
19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages 35	22. Price

<u>Cost-Efficient and Storm Surge-sensitive Bridge Design for Coastal Maine</u>

Ellen Douglas, Paul Kirshen, and Sam Merrill, through the New England Environmental Finance Center

August 31, 2013

1. Introduction

In October 2009, MaineDOT prepared a synthesis paper entitled "Climate Change Adaptation and Transportation in Maine". This document served to prepare MaineDOT to respond to the recommendations presented in LD 460, Resolve to Evaluate Climate Change Adaptation Options for the State, passed during the First Regular Session of the 124th Maine State Legislature, and the suggestion that adaptation measures be identified that would likely lessen catastrophic effects of documented changing weather patterns.

Climatic variation felt through changing weather patterns is having increasingly acute effects on Maine's transportation infrastructure. Acute risk occurs as a result of events, such as storms and flooding, while chronic risk surrounds longer range changes due to climate over time. It is acute risk that results in an increased need for disaster designation and response, increased risk of collateral property damage, and threats to safety of the traveling public. This type of risk can be mitigated by early preparation. Because of uncertainty about the future of climate and weather variability, chronic risk is much harder to gauge. This uncertainty can create paralysis in an agency charged with making and justifying long-term, fiscally-responsible decisions around the safety and efficiency of public travel. But long lifecycles of most transportation infrastructure demand adaptation via early preparation to protect significant taxpayer investments into the reasonably foreseeable future. This research effort developed by the New England Environmental Finance Center (EFC), housed at the Muskie School of Public Service, University of Southern Maine, builds on cost modeling developed to estimate fiscal impacts of LD 1725 and develops an approach to estimate acute and chronic cost/risk tradeoffs for a subset of transportation project needs along the Maine coast.

1.1 Project Objectives

The general goal is to address cost and risk issues associated with projected sea level rise (SLR) and threats to state infrastructure elements that are also designed to pass freshwater streams and rivers. More specifically, this project aims to help develop storm surge-sensitive design standards and approaches for large, tidally influenced transportation structures along the Maine coast. For each of the two major tidal regimes along the coast, the project aims to 1) obtain and analyze surge frequency from an existing tide gauge; 2) obtain and analyze flood flows from the corresponding stream flow gauge at the site (if none available, use regional hydrologic analysis to develop) and develop a joint probability distribution with coastal storm surges; 3) focus on one type of structure (bridges); and 4) identify installation costs versus damage risk-tradeoffs under different SLR, storm surge, and river flood scenarios extending out over the useful life of the asset.

Because design standards to achieve storm surge and freshwater flooding sensitivity and optimum risk reduction are sensitive to a wide range of locally idiosyncratic hydraulic and hydrologic variables, and for most locations these data do not exist, EFC was limited in its ability to guide design standard decisions. To address this, EFC analyzed the results from the two tidal regime examples to determine cost effective design processes. This resulted in: (1) recommendations for a data-gathering process (including which data are necessary and how to obtain them) so that once MDOT has the appropriate local hydrological and coastal data for any location, MDOT can make a few calculations to identify upfront costs versus risk tradeoffs under different sea level rise and storm surge scenarios extending out over the useful life of the asset, in a manner tailored to each location; and (2) details on how to incorporate in the evaluation process both the probabilities of high volume freshwater flow and the probabilities of different storm surge frequencies (20%, 5%, 1%, and 0.5 % likely storm events), through the expected life of the infrastructure, under low and high SLR conditions.

1.2 Project Tasks

Tasks from the original project workplan are as follows.

Task 1. Meet with ME DOT about design procedures and case study sites. The project team will meet with ME DOT officials to discuss their present design procedures relating to coastal freshwater and saltwater discharges, and then select two ME DOT structures (e.g., bridge or culvert) to use in the case studies. Each site will be representative of one of the two tidal regimes in ME and near reliable tide and streamflow gauges with reasonable time series of data.

Task 2. Develop elevation and velocity frequencies at Case Study Sites. We anticipate that the key design variables will be the velocity and discharge of water at each site and that they are uniquely related at the site. For each set, we will develop frequency curves of their occurrence using (1) a hydraulic model such as HEC RAS to determine the flow elevations and velocities at a site given the historic values of tidal, storm surge, and freshwater discharge conditions and then (2) extreme value analysis. The choice of including the tidal height probability in the design or assuming all events occur at some condition such as Mean Higher High water (MHHW) will be determined based upon discussions with ME DOT.

Task 3 Identify installation costs versus damage risk tradeoffs under different sea level rise, storm surge, and river flood scenarios extending out over the useful life of the asset. Using flood damage data, alternative structural designs and cost data to be provided by ME DOT, the team for each alternative structural design and each climate change scenario of sea level rise and discharge will determine the expected values of the costs and damages over time as was done in Kirshen et al (2012).

Task 4. Provide 1) graphic and numeric descriptions of avoided cost implications of different design standards for the modeled infrastructure, under different flooding, storm surge and sea level rise scenarios, and 2) a step-by-step method for MDOT to collect hydrologic and coastal data and model cost/risk tradeoffs for transportation project needs along the Maine coast.

Task 1 was accomplished when the project team met with ME DOT personnel on March 25, 2011 to decide on the appropriate structure to evaluate and the two case studies to use for the analysis. We selected two sites: the Rt. 1 bridge in Machias, ME and the Martins Point Bridge linking Portland and Falmouth, ME. The research results are first presented by providing the methodology and output for Tasks 2 and 3 for each of the sites and then Task 4 is discussed in the final sections of the report. The present design procedure of ME DOT is event-based. For example, Section 2.3.10.2, B requires that the minimum design freeboard in tidal areas is two feet above the Q10 river discharge based upon mean high water (ME DOT, 2003). There are other more complex procedures for selecting design scour conditions. In this report, we demonstrate an alternative risk-based approach for bridge design considerations.

2. Case Study: Little Machias Bridge

2.1 Tide gage data

Water levels measured at the Eastport tidal gage (NOAA ID 8410140) were used to represent tide and storm surge occurrences for the Little Machias bridge http://tidesandcurrents.noaa.gov/ data_menu.shtml?stn=8410140%20Eastport,%20ME&type=Tide%20Data). Water level anomalies, representing storm surge, were computed by removing tidal influence and historical sea level rise following Kirshen et al. (2008). A frequency analysis was performed on the water level anomalies; results are shown in Table 2.1.

Table 2.1: Water level anomaly heights at selected frequencies estimated from Eastport tide gage.

Return Period (yr)	Water level anomaly (ft)
2	2.57
5	3.35
10	4.01
20	4.76
50	5.95
100	7.04
200	8.32
500	10.40

To estimate the difference between observed water levels at the tide gage in Eastport and at the Little Machias bridge, MaineDOT installed a pressure transducer for approximately 3 weeks (July 12 through August 10, 2011). A comparison of maximum water level heights between the Eastport gage and the Little Machias bridge is shown in Figure 2.1. Maximum tide elevations were lower at the Little Machias bridge than at the Eastport gage by a median difference of -1.91 ft. The median difference was used to lower the Eastport gage elevations used in our analysis. Mean higher high water (MHHW) at the Eastport gage was reported to be 9.35 ft NAVD (http://tidesandcurrents.noaa.gov/data_menu.shtml? stn=8410140%20Eastport%20ME &type=Datums). Subtracting the median difference between observed water level maxima between the Eastport gage and the Little Machias bridge, we estimated MHHW at the Little Machias bridge as 7.44 ft NAVD. To simulate the impact of storm surge on the Machias bridge, we added the water level anomaly heights (see Table 2.1) to Machias MHHW (7.44 ft NAVD). For instance, the elevation of the 100-year storm surge at high tide was estimated to be 14.48 ft NAVD (7.04 ft + 7.44 ft NAVD).

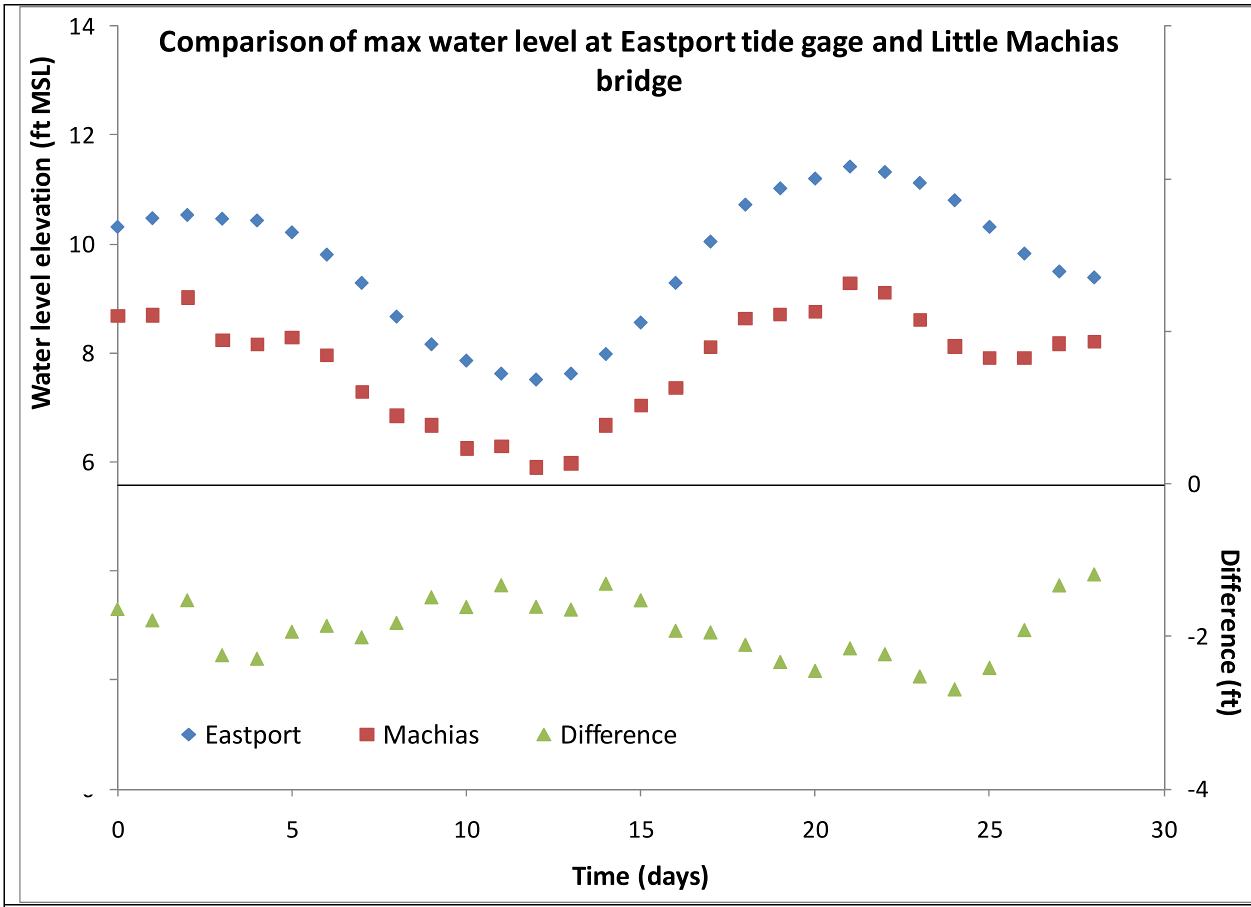


Figure 2.1: Comparison of the maximum water level heights measured at the Eastport gage (blue) and near the Little Machias bridge (red). Differences in elevation (green) ranged from approximately -1 to -3 feet with a median of -1.91 ft between the two elevations.

2.2 Streamflow data

The Middle River, which flows under the Little Machias bridge, is a small ungaged tributary of the Machias River. The watershed area was estimated to be 13.2 mi^2 In ArcMap using DEM elevation data from the Maine Office of GIS. (see Figure 2.2). A regression equation (Hodgkins, 1999) was used to estimate flood flows at various frequencies (see Table 2.2).

Table 2.2: Flood frequency estimated for the ungaged Middle River

Return Period (year)	Peak Flow Estimate			
	Lower	Q_T (m^3/s)	Upper	Q_T (ft^3/s)
1.1		4.29		152
2	6.01	8.41	11.76	297
5	9.12	12.80	17.95	452
10	11.28	15.99	22.68	565
25	14.09	20.26	29.14	715
50	16.20	23.57	34.31	832
100	18.42	27.14	39.98	958
500	23.53	35.79	54.45	1264

2.3 Joint probability assessment

Flooding at the Little Machias bridge can result from either excessive storm surge height (hs) or river flood height (hr). We evaluated the likelihood of coincident storm surge and river flood. We found that only four of the twelve floods of 20-year frequency or less coincided with storm surges. These surge heights ranged from 1.0 and 1.4 feet, much lower than the 2-year storm surge height of 2.57 feet (Table 2.1). We found that there were no floods with frequency of 50-yr or larger that coincided with storm surge. Based on this analysis, we established that storm surge and river flooding were essentially independent events, which made the evaluation of joint probabilities much simpler.

In order to estimate the joint probability of flooding due to storm surge heights (hs) and river flooding heights (hr) at the Machias bridge, we estimated the composite probability function of flood heights following Vogel and Stedinger (1984). The composite probability function was estimated from the distribution of the maximum heights, hm, observed at the location of interest where hm = max(hr, hs). First, a probability distribution was estimated for each height series, hr and hs. The flood flows in Table 2.2 were input into the HEC-RAS model (described in Section 2.4 below) to estimate their associated river flood stage heights (hr, in ft

NAVD). These heights were found to follow a ln-normal probability distribution. The storm surge heights (represented by water level anomalies) were found to follow a Generalized

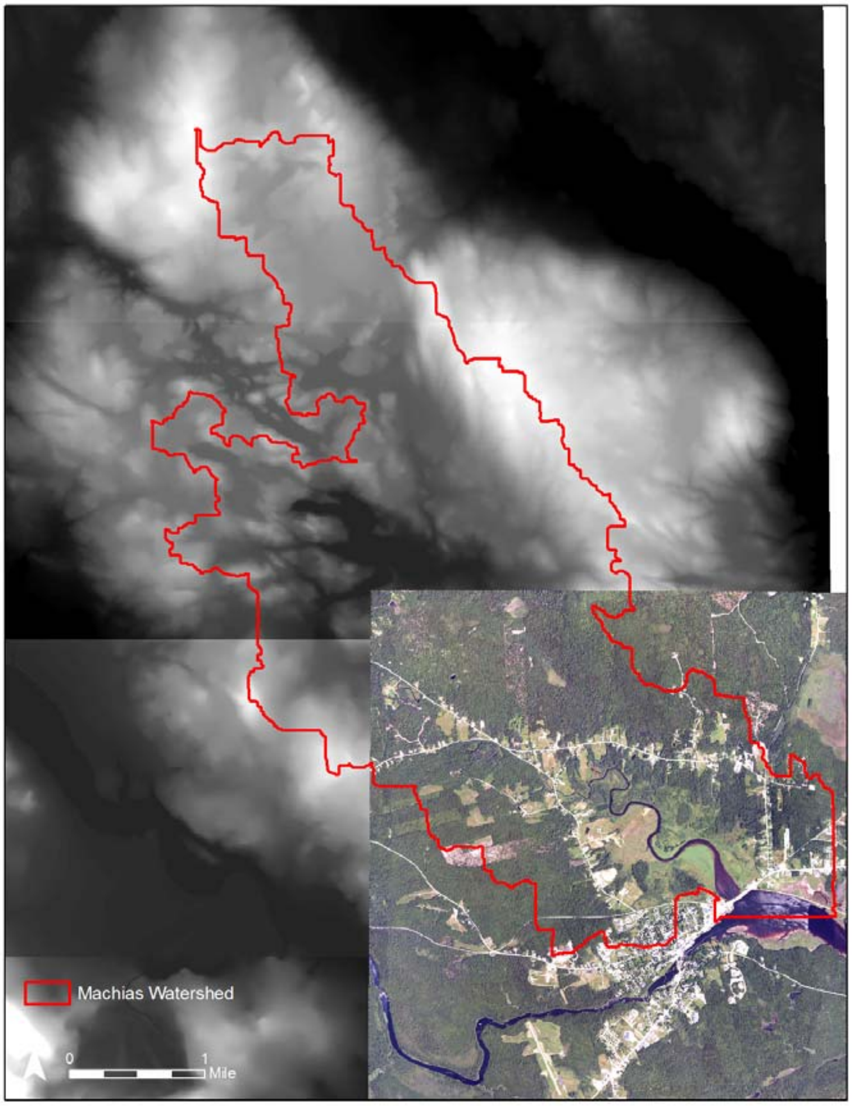


Figure 2.2: Middle River watershed delineation using GIS analysis.

Extreme Value (GEV) distribution. Parameters for these probability distribution functions (pdfs) were estimated from the river stage and surge height data and the fitted pdfs were used to randomly generate a coincident series of 1,000 values for hr and hs, from which the composite distribution series was computed as hm_i = max (hr_i, hs_i), where i = 1…1000. For all i, the storm surge elevations (hs) exceeded the flood stage elevations (hr) as shown in Figure 2.3, hence hm followed the same GEV probability function as hs (Table 2.1) . The probability of flood heights at the Little Machias bridge can be estimated as:

$$P(h_m \le h) = \exp\left\{-\left[1-\kappa\left(\frac{h-\xi}{\alpha}\right)\right]^{1/\kappa}\right\} \quad (1)$$

The exceedance probability for hm is:

$P(h_m > h) = 1 - P(h_m \le h)$

where h is the flood height of interest.

Exceedance values were derived from this equation for the analysis in Section 2.5.

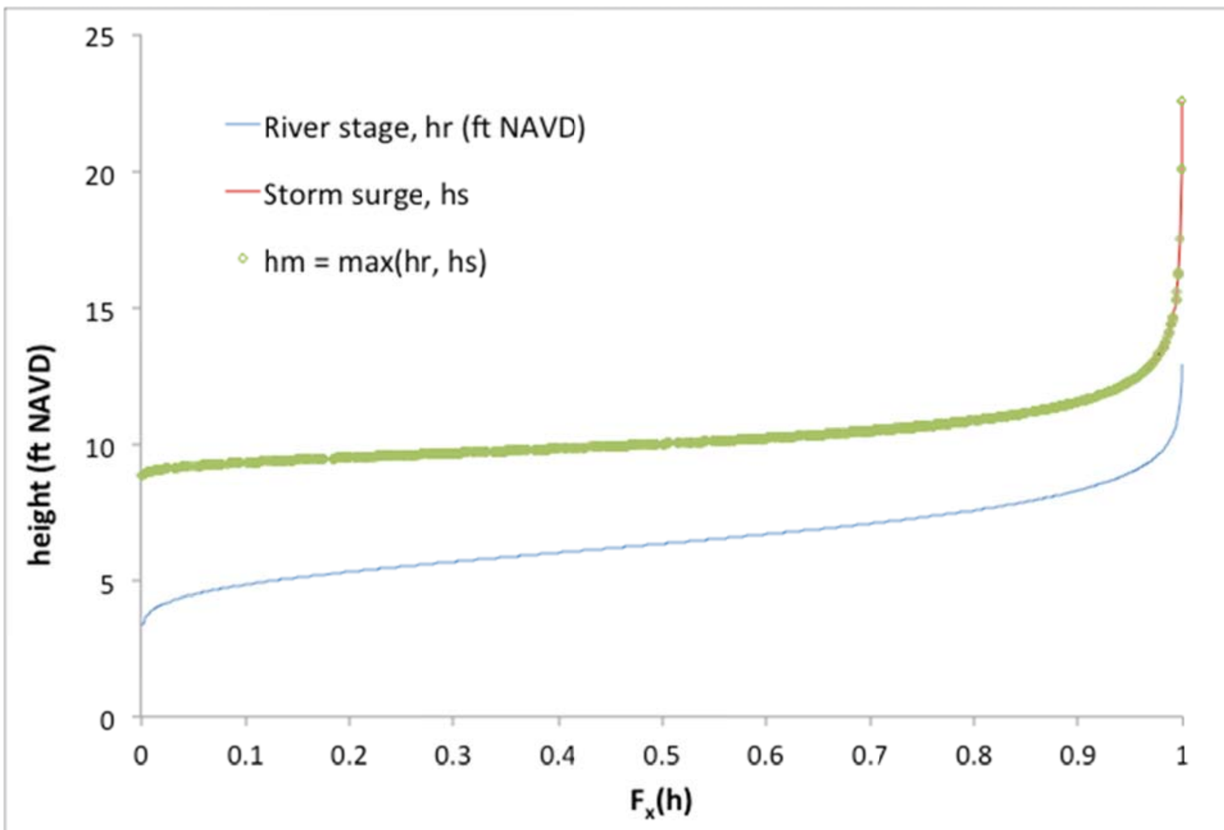


Figure 2.3: Comparison of flood stage height, storm surge height, and maximum height distributions.

2.4 Simulation of river flow and bridge alternatives

In order to simulate the combined effects of river flooding and storm surge on flood elevations at the bridges, a computer model was developed for each site. HEC-RAS, a river model developed by the Army Corps of Engineers and publically available (http://www.hec.usace.army.mil/software/hec-ras/hecras-download.html) was used. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The required input data includes channel geometry, river discharge and boundary conditions. The model outputs discharge, velocity and water surface elevations at selected cross-sections. For our analysis of existing conditions, we used the 100-year peak discharge provided by MaineDOT (see Table 2.2). For future conditions, this discharge was increased by 10%. In the absence of storm surge or flooding in the Machias River, the downstream boundary condition was set at normal depth, using an estimated slope of 0.0008 based on cross-section information. For storm surge and downstream flooding conditions, the downstream boundary condition was set to equal the storm surge height plus MHHW (7.44 ft NAVD as described in Section 2.1). Bridge geometry was provided by the MaineDOT. Channel geometry and model development are described below.

2.4.1 Machias Bridge HEC-RAS model

Surveyed elevation contour data (in CAD format) and river channel cross-sections were provided by MaineDOT. These data was plotted in ArcMap and an aerial photograph was georeferenced to provide context for the project area. Transects were created on either side of the bridge, and existing channel cross-sections were extended into the floodplain to enable modeling of higher flow regimes. The extended transects were then split at slope breaks to calculate both the length of each segment of the transect and to extract the elevation from the contour data at each break point. The distance between transects was determined by sketching a longitudinal profile along the stream channel, then calculating length in ArcMap. These values were used as inputs for the downstream reach lengths in HEC-RAS. Figure 2.4 depicts elevation contours,

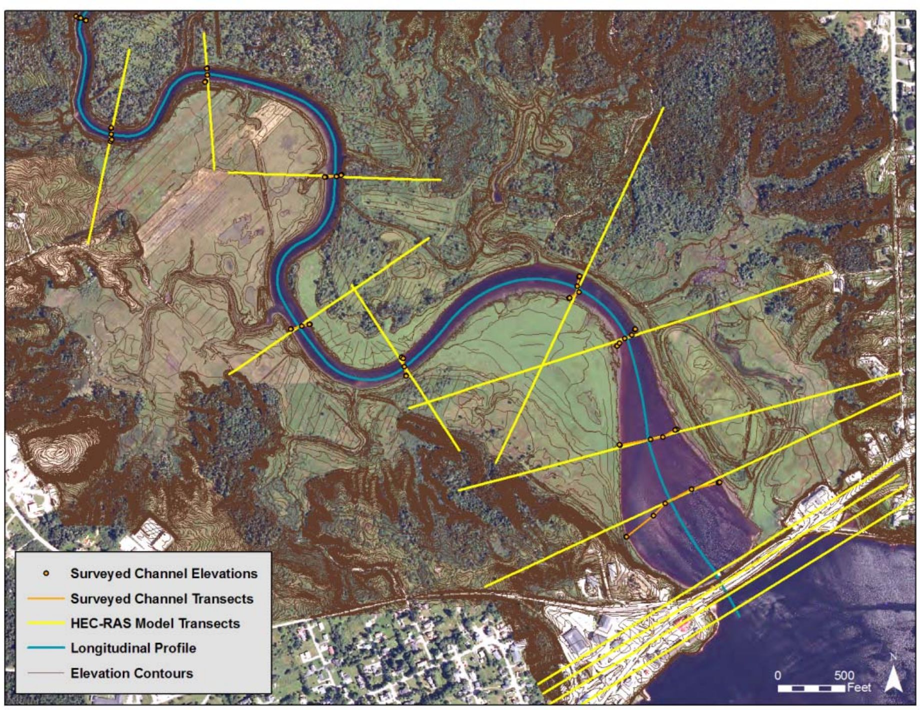


Figure 2.4: Locations of Little Machias Bridge Cross-Sections for establishing river channel geometry.

surveyed channel cross-section locations, as well as the extended transects. Additional interpolated cross-sections were added

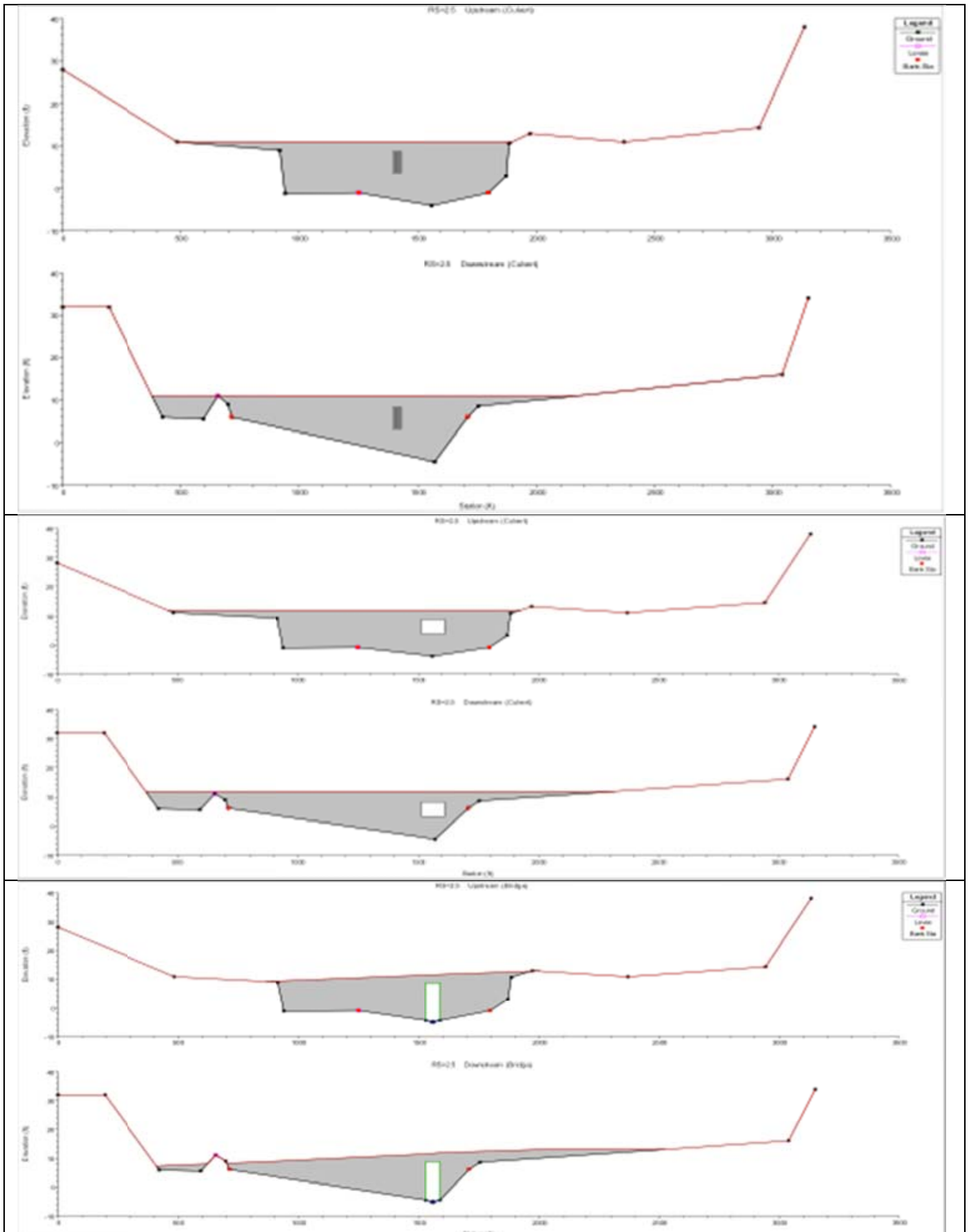


Figure 2.5: Upstream (top) and downstream (bottom) bridge representations for a) existing 4-opening bridge; b) box culvert and c) span.

in HEC-RAS, with a maximum separation of 100 feet. The bridge deck height and width were measured from the elevation contours, and were then augmented with bridge geometry information provided by MaineDOT. Figure 2.5 a-c represent the three bridge scenarios: existing, box culvert and span. Exiting bridge geometry was taken from a construction drawing, box culvert (100 ft width, 5 ft height) and span geometries were provided by Michael Wright of MaineDOT. It is important to note that the existing bridge was simulated assuming that the tide gates were always open, as tide gates are not a bridge option in HEC-RAS. No scour analysis was carried out because the bridge alternatives here are well protected against scour.

2.5 Cost Analysis

A climate change adaptation least cost analysis was carried out for each sea level rise (SLR) scenario and each alternative bridge design using the expected value analysis method of Kirshen et al (2012). It assumes that over the study period every time the bridge is damaged by flooding, it is repaired within that year back to the present design. Therefore the depth damage function remains the same for each year. However, due to SLR, the exceedance probabilities of the damages change each year. Therefore the analysis is applied for several scenarios of SLR. For each SLR scenario, the method determines the expected discounted value of damage for the present (assumed to be 2010) and specified years in the future – here assumed to be 2030 and 2050. The area under the curve of expected discounted value damage versus time is equal to the total expected discounted value over the time period. To this latter cost can be added the cost of the bridge and any maintenance costs to determine the total expected value discounted cost of the bridge including bridge construction and damage costs over the planning period. Maintenance costs can also be added. Assuming each bridge supplies the same level of service and cost is the only metric, then the less total cost bridge is to be recommended. If damages can be assigned to the services lost because of bridge flooding and consequent loss of services and these vary by bridge design, then it is possible to assign a benefit to damages avoided for each bridge design for each scenario and conduct a benefit-cost analysis. Here only a less cost analysis is applied.

The method is applied to the box culvert replacement design with results also being presented for the single span design.

2.5.1 Develop a depth – damage relationship.

MaineDOT provided ranges of costs to repair each of the two bridge alternative designs as a function of the ranges of elevations of flooding (see Appendix 1). It was assumed that within a range of flood elevations, the damage cost was the average of the range of costs. Therefore the depth- damage functions for the box culvert and single span alternatives are in Table 2.3.

Table 2.3 Depth-Damage curve for box culvert and single span alternatives

Range of Elevation (feet NAVD, includes wave action)	Damage Costs (2010 $1000) for 4 - 6 'span by 6' rise butted precast concrete box culverts with flapper gates	Damage Costs (2010 $1000) for single span bridge with integral abutments supported on piles driven to bedrock
11 -12	25	25
12-13	38	38
14-15	125	125
15	250	250
>15	1000	1750

2.5.2. Determine elevation - probability exceedance relation for present and climate change scenarios.

The previous analysis in Section 2.3 determined that flooding at the bridge is due to the elevation of the storm surge. Therefore, for high and low SLR scenarios, we determined elevation-exceedance curves. Table 2.4 shows the exceedance probabilities for various surge heights (column 2), which we assume to remain stationary. While more heights were available, the table only has 4 values, judged suitable to capture the variability. According to Vermeer and Rahmstorf (2009), the amount of global SLR expected for high and low scenarios is given in Table 2.5. Local subsistence in this area is negligible (estimated to be approximately 4 cm over 100 years). We assumed wave action in this area is 1.0 feet and that all surges occurred at MHHW, which is 7.44 feet NAVD. Therefore, flood probability exceedances for various flood elevations for each SLR scenario can be determined by adding the MHHW elevation to the surge height and then to the wave height and the amount of SLR for that scenario. This is in Table 2.4 for the low SLR for 2030 (column 7).

Table 2.4 Surge height exceedances for the low SLR for 2030

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Return Period (years)	Probability of Exceedance	Surge Height (ft)	MHHW (ft NAVD)	Wave Height (ft)	2030 Low SLR Scenario (ft)	Elevation for Low SLR, 2030, (ft NAVD)	Box Culvert Bridge Damage ($1000)
5	0.2	3.35	7.44	1.0	0.5	12.29	38
20	0.05	4.76	7.44	1.0	0.5	13.7	125
100	0.01	7.04	7.44	1.0	0.5	15.98	1000
200	0.005	8.32	7.44	1.0	0.5	17.26	1000

Table 2.5. SLR scenarios for Vermeer and Rahmstorf (2009)

Year	Low (ft)	High (ft)
2030	0.5	0.9
2050	1.0	1.7

2.5.3. Determine the damage- probability exceedance relation for present and climate change scenarios.

This is done by assigning a bridge damage cost to each elevation. The result for the box culvert design is in Table 2.4, column 8. The damage is $1,000K for the 0.01 and 0.005 exceedances because above 15 ft NAVD of flooding, the damage is $ 1million according to Appendix 1.

2.5.4 Determine the expected value of the damage for the present and each year in the scenarios.

This is the area under the curve resulting from Table 2.4 if the costs were plotted versus the exceedances. The area can be estimated by many methods. Here, the area was estimated by multiplying the average damage between two exceedance values by difference of the exceedance values. This was done for the years of 2010, 2030, and 2050. The value for each year was then be discounted by 3.5 percent. Table 2.6 shows the damage exceedance data for 2010 and Figure 2.6 shows how the area under the curve was estimated. Table 2.7 shows the discounted damage for each year for each scenario for each bridge alternative with the curve area determined in this manner.

Table 2.6 Damage Exceedance for Box Culvert Design in 2010

Return Period (years)	Exceedance Probability	Damage ($1000)
5	0.2	25
20	0.05	38
100	0.01	125
200	0.005	1000

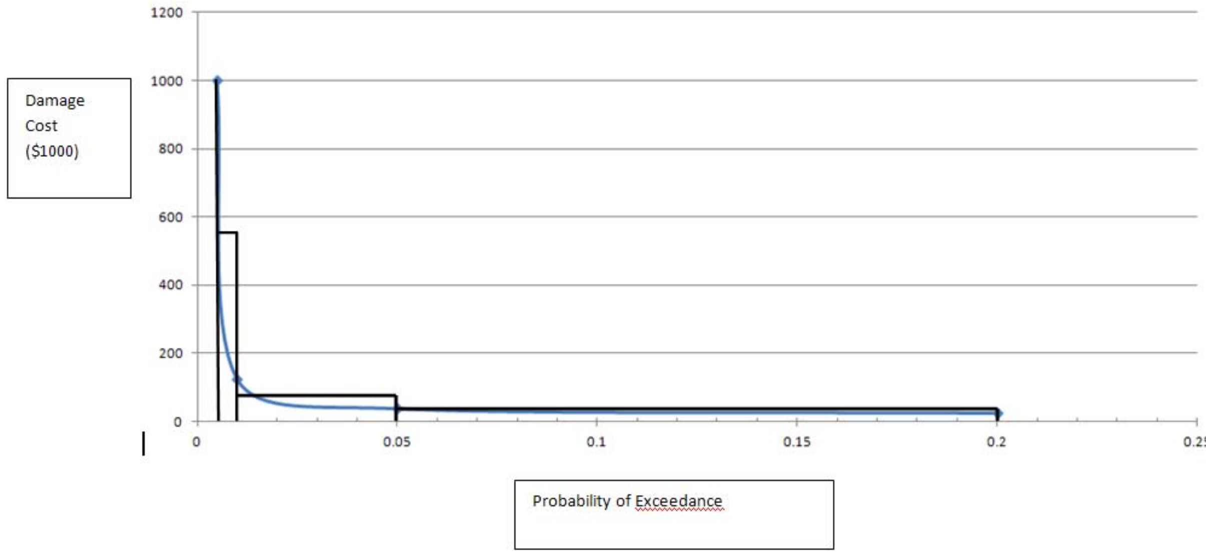


Figure 2.6 Estimating area under damage exceedance curve for box culvert for 2010

2.5.5. Determine the total expected discounted value of the damages for each scenario over all years.

The results from Step (4) can be plotted as expected value discounted damages for the planning years versus year. The area under this curve is the total expected discounted value of the damages for each scenario over all years. The results for the box culvert are in Table 2.7 as are results for the Single Span. Costs are the same for both SLR scenarios for each bridge for each time period because the scenario elevation changes are not large over time or scenario and the costs are given for ranges of a foot or more. The Single Span cost is higher than the Box Culvert cost because of its significantly larger damage costs over 15 feet NAVD flooding.

Table 2.7 Expected Discounted Value of Damages

Year	Low SLR, Box Culvert, Discounted Damages ($1000)	High SLR, Box Culvert, Discounted Damages ($1000)	Low SLR, Single Span, Discounted Damages ($1000)	High SLR, Single Span, Discounted Damages ($1000)
2010	10.8	10.8	12.7	12.7
2030	20.0	20.0	29.4	29.4
2050	10.02	10.02	14.8	14.8
Total Expected Value Discounted Damages	607.3	607.3	862.8	862.8

2.5.6. Add in the costs of the bridges and compare.

As can be seen in Table 2.8, the Box Culvert is the least total cost design. If the more expensive Single Span bridge received less damage from flooding, then that may have been the least cost bridge.

Table 2.8 Total Discounted Value of Damages and Costs

Bridge Design	Expected Discounted Value Damages ($1000)	Bridge Cost ($1000)	Total Costs ($1000)
Box Culvert	607.3	2,200	2,807.3
Single Span	862.8	2,400	3,262.8

3. Case Study: Martin's Point Bridge

3.1 Tide gage data

Historical hourly water-level data from the tide gauge in Portland (ID 8418150) were obtained from the Tides and Current web site of the National Oceanic and Atmospheric Administration / National Ocean Service (NOAA/NOS) Center for Operational Oceanographic Products and Services (CO-OPS). Upon initial processing of the NOAA/NOS dataset, data gaps and unexplained fluctuations were found in the data record, therefore it was determined that the data quality was not sufficient to continue the processing at this time. Instead, we obtained the Research Quality Dataset (RQDS) for the Portland tide gauge from the University of Hawaii Sea Level Center/National Oceanographic Data Center, Joint archive for sea level (http://ilikai.soest.hawaii.edu/uhslc/htmld/d0252A.html). Water level anomalies, representing storm surge, were computed by removing tidal influence and historical sea level rise following Kirshen et al. (2008). A comparison of annual maximum WLA from the raw tide gauge dataset and the RQDS is shown in Figure 3.1. A frequency analysis was performed on the WLA

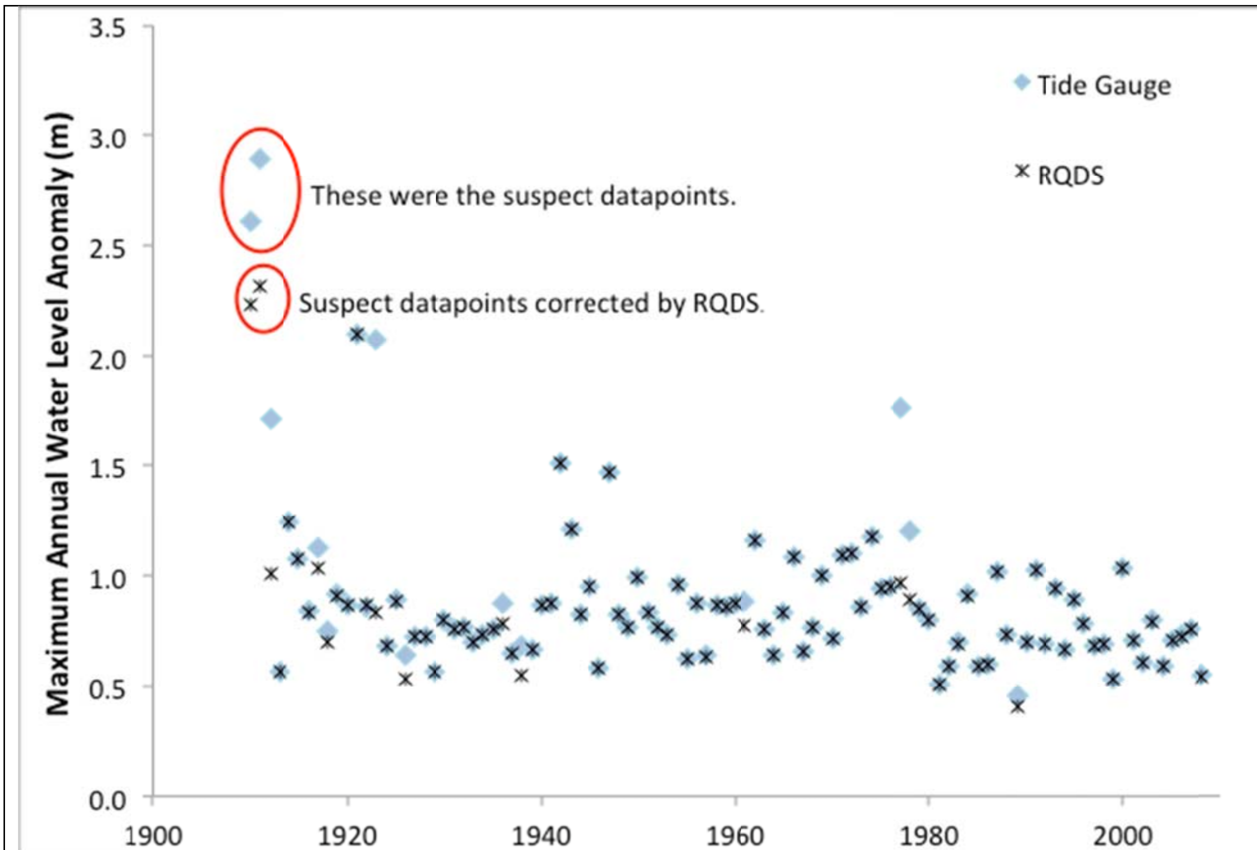


Figure 3.1: Comparison of raw Portland, ME tide gauge data with corrected research quality dataset (RQDS) from the University of Hawaii Sea Level Center.

Table 3.1: Results of frequency analysis for Portland, ME RQDS maximum annual water level anomalies.		
Exceedance probability	**Water level anomaly quantiles (m)**	**Water level anomaly quantiles (ft)**
0.5	0.78	2.57
0.2	1.01	3.32
0.1	1.19	3.92
0.05	1.40	4.58
0.02	1.71	5.60
0.01	**1.98**	6.50
0.005	2.30	7.54
0.002	2.79	9.15

extracted from the raw tide gage data and from the RQDS, both with and without the 1910 and 1911 high datapoints; based on this comparison, we decided to retain the results from the RQDS with the 1910 and 1911 datapoint. These results are shown in Table 3.1.

3.2 Simulations of Presumpscot River flow and Martin's Point bridge

3.2.1 Martin's Point Bridge

Limited elevation data was provided in CAD format by MaineDOT for the area immediately surrounding the Martin's Point bridge. The bridge geometry and cross-sections on either side of the bridge were extracted from these contours in ArcMap by sketching transects and measuring the distance between slope breaks. Figure 3.2 depicts the elevation contours that were provided, along with the transects on either side of the bridge. Measurements of the bridge piers and the wider span towards the center of the bridge were provided by MaineDOT and incorporated into the HEC-RAS model. In order to produce enough cross-sections to run the HEC-RAS model, NOAA bathymetric maps were used in conjunction with topographic data upstream of the bridge to produce profiles of the channel and floodplain (see Figure 3.3). Distances between cross-sections were measured along the longitudinal profile in ArcMap, which were subsequently imported into HEC-RAS. In HEC-RAS, more cross-sections were interpolated at 100 foot intervals.

Figure 3.2: Areal view, transects and elevation contours for the Martins Point Bridge

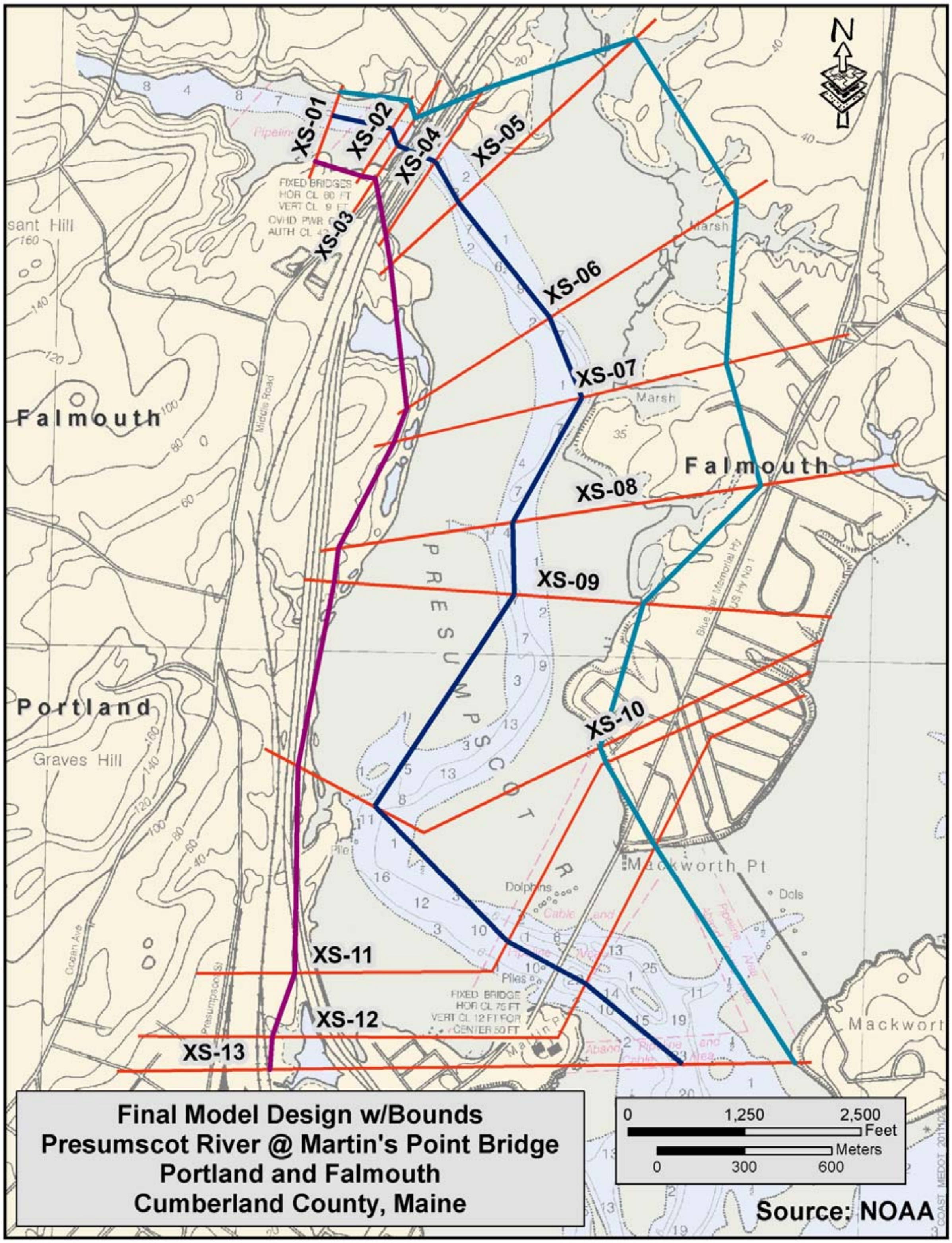


Figure 3.3: Model cross-section locations overlain on a bathymetry map of the Presumpscot River estuary.

3.3 Streamflow data

There are three relevant U.S. Geological Survey locations at which streamflow were available for the Presumpscot River: the outlet of Sebago Lake (USGS 01064000; period of record 1901 to 2000), Westbrook, ME (USGS 01064118; period of record 1975 to 1995) and near West Falmouth, ME (01064140; period of record 1975 to 1984). Unfortunately, the location with the longest streamflow record (hence most appropriate for flood frequency analysis) was not appropriate for evaluating floods at the Martin's Point bridge because of flow regulation. In order to extend the annual maximum streamflow (flood flow) record at the location closest to the bridge (West Falmouth ME), a linear regression analysis was performed between the Westbrook and West Falmouth annual maximum data; the resulting regression model was then used to extend the flood flows at West Falmouth to 1995. A frequency analysis was performed on the extended West Falmouth flood flows (see Table 3.2) and the 100-year and 500-year flood flows (Q100 and Q500) were estimated to be approximately 21,900 and 29,100 cfs, about 40 to 50% higher than the FEMA FIS Q100 and Q500 estimates for the lower Presumpscot River. This discrepancy was most likely a result of the short streamflow record (21 years) available for this analysis. However, the frequency analysis was useful in identifying a range of expected flood flows for simulation purposes.

Table 3.2: Flood flow quantiles estimated from the extended annual maximum streamflow record at West Falmouth, ME and compared to FEMA FIS study results. Right column tabulates the modeled river stage just upstream of the Martin's Point bridge assuming flood flows occur at MHHW.

Exceedance Probability	Flood quantiles (cfs)	FEMA FIS quantiles (cfs)	Water surface elevation (ft NAVD)
0.5	6584		4.62
0.1	12757		4.69
0.02	19003		4.77
0.01	21874	15300	4.81
0.002	29081	19700	4.89

A rating curve for the river cross-section just upstream of the Martin's Point bridge (XS-sec 11 in Figure 3.4) was developed using the HEC-RAS model run with an incremental series of flood flows across this range. To be consistent with the storm surge analysis, it was assumed that these flood flows would occur at mean higher high water (MHHW), so the downstream boundary condition in the model was set at MHHW (4.65 ft NAVD). The rating curve (Figure 3.4) was then used to estimate the river stage elevation at this cross-section (right column in Table 3.2).

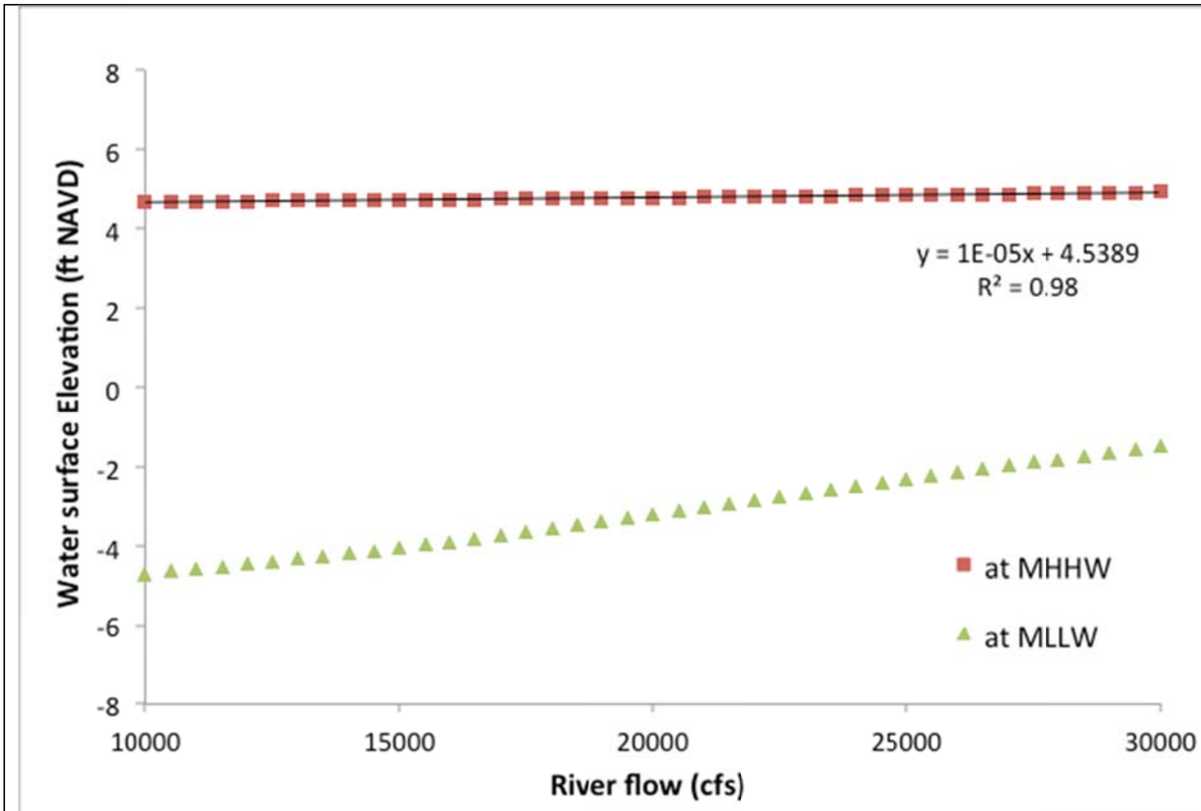


Figure 3.4: Rating curves for model XS-11 just upstream of Martin's Point bridge, assuming both high tide (MHHW) and low tide (MLLW) boundary conditions.

3.4 Joint probability of flooding analysis

We evaluated the possibility of flooding at the Martin's Point bridge resulting from either excessive storm surge height (hs) or river flood height (hr). We evaluated the coincidence of storm surge and river flood events at this location over the relatively short timeframe of the streamflow record (1975 to 1995). As with a similar comparison for the Little Machias bridge analysis (section 2.3), we found that annual maximum flood events did not coincide with annual maximum storm surge events, hence we assumed that storm surge and river flooding at the Martin's Point bridge were also independent events.

In order to estimate the joint probability of flooding due to storm surge heights (hs) and river flooding heights (hr) at the Martin's Point bridge, we estimated the composite probability function of flood heights following Vogel and Stedinger (1984). The composite probability function was estimated from the distribution of the maximum heights, hm, observed at the location of interest where hm = max(hr, hs). First, a probability distribution was estimated for each height series, hr and hs. The flood flows in Table 3.2 were input into the HEC-RAS model (described previously) to estimate their associated river flood stage heights (hr, in ft NAVD).

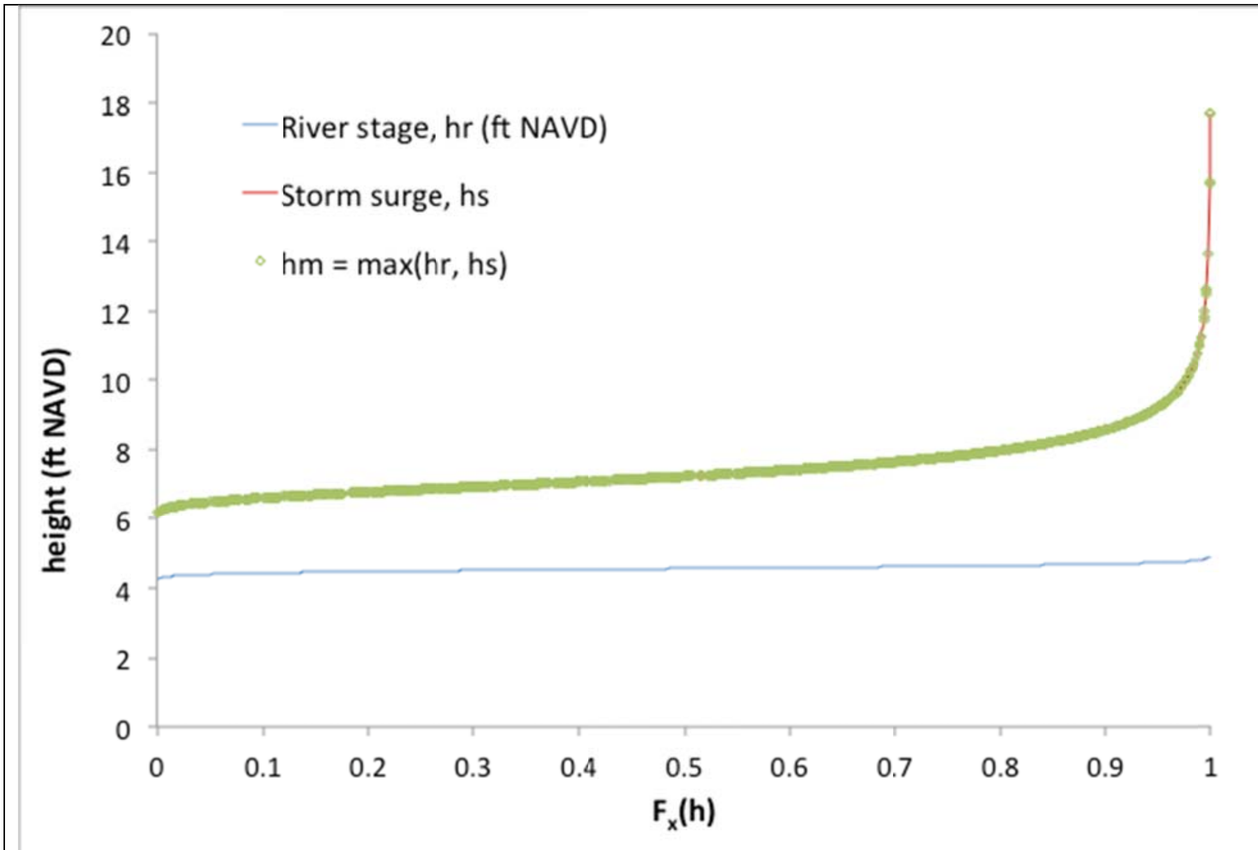


Figure 3.5: Comparison of flood stage height, storm surge height, and maximum height distributions.

These heights were found to follow a ln-normal probability distribution. The storm surge heights (water level anomalies) were found to follow a Generalized Extreme Value (GEV) distribution. The probability functions were used to randomly generate a coincident series of 1,000 values for hr and hs, from which the composite distribution series hm was computed. In all cases, the storm surge elevations (hs) exceeded the river flood stage elevations (hr, shown inFigure 2.5), hence the GEV probability function for storm surge heights can be used to estimate the probability of flooding at Martin's Point bridge. The probability of storm surge heights at the Martin's Point bridge can be estimated as in equation 1, where h is the surge height of interest, and for Martin's Point, ξ = 0.722; α = 0.17 and κ = -0.20. The exceedance probability for hm, $P\left(h_m > h\right) = 1 - P(h_m \leq h)$. The flooding elevation can be estimated by adding MHHW and wave height to the storm surge heights estimated by (1).

3.5 Potential for scour at the Martin's Point bridge.

Using the HEC-RAS model with three different downstream boundary conditions (normal depth, MLLW and MHHW), we evaluated the average flow velocity at the river cross-section just upstream of the Martin's Point bridge for Q50, Q100 and Q500 using both the flood flows estimated in this report (Table 3.2) and by FEMA (Q50 not reported by FEMA). Because the Presumpscot River empties into a tidal estuary, normal depth is an expected flow condition at the bridge and was included for comparison purposes only. Table 3.3 shows the results of this analysis. We did not perform a HEC-18 scour analysis but note that the velocities at MLLW could warrant a more detailed scour analysis.

Table 3.3: Average river velocities for specified downstream boundary conditions.

Flood Discharge (cfs)	Velocity upstream of bridge with specified boundary condition (fps)		
	Normal depth	*MLLW*	*MHHW*
15,300[a]	2.1	2.1	0.54
19,700[b]	1.7	2.3	0.70
19,000[c]	1.7	2.3	0.66
21,900[d]	1.7	2.4	0.77
29,100[e]	1.7	2.4	1.0

Notes: [a]FEMA Q100
[b]FEMA Q500
[c]Q50 from frequency analysis described in this report (Table 3.2).
[d]Q100 from frequency analysis described in this report (Table 3.2).
[e]Q500 from frequency analysis described in this report (Table 3.2).

3.6 Cost Analysis

A climate change adaptation least cost analysis was carried out for each sea level rise (SLR) scenario and each alternative bridge design using the expected value analysis method of Kirshen et al (2012). It assumes that over the study period every time the bridge is damaged by flooding, it is repaired within that year back to the present design. Therefore the depth damage function remains the same for each year. However, due to SLR, the exceedance probabilities of the damages change each year. Therefore the analysis is applied for several scenarios of SLR. For each SLR scenario, the method determines the expected discounted value of damage for the present (assumed to be 2013) and specified years in the future – here assumed to be 2033 and 2063. The area under the curve of expected discounted value damage versus time is equal to the total expected discounted value over the time period. To this latter cost can be added the cost of the bridge and any maintenance costs to determine the total expected value discounted cost of the bridge including bridge construction and damage costs over the planning period. Maintenance costs can also be added. Assuming each bridge supplies the same level of service and cost is the only metric, then the least total cost bridge is to be recommended. If damages can be assigned to the services lost because of bridge flooding and consequent loss of services and these vary by bridge design, then it is possible to assign a benefit to damages avoided for each bridge design for each scenario and conduct a benefit-cost analysis. Here only a least cost analysis is applied.

3.6.1 Develop a depth – damage relationship.

As given in Table 3.7 e, the maximum elevation expected at the bridge under the high climate change scenario by 2063 is 16.7 feet NAVD or approximately 17 feet NAVD. At water elevation 17 feet NAVD the bridge deck of the Martin's Point Bridge as currently designed and is being built would still be above water. However, some of the beams would be impacted because the bottom of the beams span from 24 feet NAVD on the Portland side to 12 feet NAVD on the Falmouth side. In addition, the causeway approaching the bridge from Falmouth has a low elevation of 12 feet NAVD. VHB provided the information in Table 3.4 on repair or replacement costs for the bridge and the roadway as a function of elevation. While this would not be the case for the roadway, the bridge could withstand multiple events with partial damage and subsequent repair before a total rebuilding or replacement would be necessary. VHB estimated that the first time the bridge beams were flooded, the repair cost would be zero, but would gradually increase after each flooding. This increase is assumed to be a linear increase of by ¼ of the total repair costs up to the fifth flooding event. Since this is very difficult to model, we calculated the average cost of repair for each event for each elevation. Thus, for example, the total bridge repair cost for a 17 feet NAVD surge and wave elevation is $16 million (2013 dollars). The total cost for 5 events is the sum of 0, 16/4, 2*16/4, 3*16/4, 4*16/4 or $40M with an average cost of $8M per event. Table 3.5 contains the average repair/replacement costs for the bridge and roadway as a function of elevation. An alternative repair/replacement procedure as reported by VHB is to replace the portions of the bride and roadway below 17 feet NAVD with a new bridge and causeway connecting to the higher portions of the bridge at the cost of $15 million.

Table 3.4 Maximum repair or replacement costs for the Martin's Point bridge and the roadway as a function of elevation.

	Surge and Wave Elevation (ft NAVD)				
	9	*11*	*13*	*15*	*17*
Bridge	$0	$0	$2 M	$8 M	$16 M
Roadway	$0	$0	$1 M	$2 M	$4 M
Bridge	$0	$0	$2 M	$8 M	$16 M
Roadway	$0	$0	$1 M	$2 M	$4 M
Bridge	$0	$0	$2 M	$8 M	$16 M
Roadway	$0	$0	$1 M	$2 M	$4 M
Bridge	$0	$0	$2 M	$8 M	$16 M
Roadway	$0	$0	$1 M	$2 M	$4 M
Bridge	$0	$0	$2 M	$8 M	$16 M
Roadway	$0	$0	$1 M	$2 M	$4 M

Table 3.5 Average Repair Costs for Each Event for the Bridge and Roadway

Surge and Wave Elevation (ft NAVD)	Cost of Total Rebuild ($M)	Average Cost over 5 Events ($M)	Cost of Roadway Repair ($M)	Total Cost ($M)
13	2	1	1	2
15	8	4	2	6
17	16	8	4	12

3.6.2 Determine elevation - probability exceedance relation for present and climate change scenarios.

The hydrology/hydraulic analysis determined that flooding at the bridge is due to the elevation of the storm surge (see Section 3.4). Therefore, for high and low SLR scenarios, we determined elevation-exceedance curves. Table 3.1 shows the exceedance probabilities for various surge heights, which we assume to remain stationary. According to Figure 6 of Vermeer and Rahmstorf (2009), the amount of global SLR expected for high and low scenarios is given in Table 3.6. These scenarios are consistent with the recently SLR scenarios prepared by NOAA for the 2013 US National Climate Assessment (Parrish et al 2012). Local subsistence in this area is negligible (estimated to be approximately 0.07 feet over 100 years). We assumed wave action in this area is 2.0 feet and that all surges occurred at MHHW, which is 4.65 feet NAVD. Therefore, flood probability exceedances for various flood elevations for each SLR scenario can be determined by adding the MHHW elevation to the surge height and then to the wave height and the amount of SLR for that scenario. This is in Tables 3.7 a-e for the present (2013) and the low and high SLR scenarios for 2033 and 2063 (column 7). While more surge heights were available, the table only has 4 values, judged suitable to capture the variability and, more importantly, the expected values.

Table 3.6. SLR scenarios for Vermeer and Rahmstorf (2009)

Year	**Low (ft)**	**High (ft)**
2030	0.5	0.9
2050	1.0	1.7

3.6.3. Determine the damage- probability exceedance relation for present and climate change scenarios.

This is done by assigning a bridge damage cost to each elevation in Tables 3.7, a-e. Elevations were rounded off the foot and the corresponding cost found. It is assumed operation and maintenance costs with the exception of flood repair and replacement is not a function of elevation and thus is not included in the analysis. The cost results are in these tables.

Table 3.7 Data and Costs for SLR Scenarios (A, present; B, 2033 Low; C, 2033 High; D, 2063 Low; E, 2063 High)

Return period	Probability of exceedance	Surge Height (ft)	MHHW (ft NAVD)	Wave height (ft)	2013 SLR	Elevation for Surge and Wave (ft NAVD)	Damage Estimate ($1000)		A
5	0.2	3.32	4.65	2	0	9.97	0.00		
20	0.05	4.58	4.65	2	0	11.23	0.00		
100	0.01	6.50	4.65	2	0	13.15	2.00		
200	0.005	7.54	4.65	2	0	14.19	4.00		
				Water elevations (ft NAVD)					
Return period	**Probability of exceedance**	**Surge Height (ft)**	**MHHW (ft NAVD)**	**Wave height (ft)**	**2033 Low SLR Scenario (ft)**	**Elevation for Surge and Wave (ft NAVD)**	**Damage Estimate ($1000)**		**B**
5	0.2	3.32	4.65	2	0.5	10.47	0.00		
20	0.05	4.58	4.65	2	0.5	11.73	0.00		
100	0.01	6.50	4.65	2	0.5	13.65	4.00		
200	0.005	7.54	4.65	2	0.5	14.69	6.00		
Return period	**Probability of exceedance**	**Surge Height (ft)**	**MHHW (ft NAVD)**	**Wave height (ft)**	**2033 High SLR Scenario (ft)**	**Elevation for Surge and Wave (ft NAVD)**	**Damage Estimate ($1000)**		**C**
5	0.2	3.32	4.65	2	0.8	10.77	0.00		
20	0.05	4.58	4.65	2	0.8	12.03	0.00		
100	0.01	6.50	4.65	2	0.8	13.95	4.00		
200	0.005	7.54	4.65	2	0.8	14.99	6.00		
Return period	**Probability of exceedance**	**Surge Height (ft)**	**MHHW (ft NAVD)**	**Wave height (ft)**	**Low SLR 2063**	**Elevation for Surge and Wave (ft NAVD)**	**Damage Estimate ($1000)**		**D**
5	0.2	3.32	4.65	2	1.1	11.07	0.00		
20	0.05	4.58	4.65	2	1.1	12.33	0.00		
100	0.01	6.50	4.65	2	1.1	14.25	4.00		
200	0.005	7.54	4.65	2	1.1	15.29	6.00		
Return period	**Probability of exceedance**	**Surge Height (ft)**	**MHHW (ft NAVD)**	**Wave height (ft)**	**High SLR 2063**	**Elevation for Surge and Wave (ft NAVD)**	**Damage Estimate ($1000)**		**E**
5	0.2	3.32	4.65	2	2.5	12.47	0.00		
20	0.05	4.58	4.65	2	2.5	13.73	4.00		
100	0.01	6.50	4.65	2	2.5	15.65	9.00		
200	0.005	7.54	4.65	2	2.5	16.69	12.00		

3.7 Determine the expected value of the damage for the present and each year in the scenarios.

This is the area under the curve resulting from Tables 3.7 a-e if the costs were plotted versus the exceedances. The area can be estimated by many methods. Here, the area was estimated by multiplying the average damage between two exceedance values by difference of the exceedance values. This was done for the years of 2010, 2033, and 2063. The value for each year was then discounted by 3.5 percent.

3.8 Determine the total expected discounted value of the damages for each scenario over all years.

The results from Section 3.7 can be plotted as expected value discounted damages for the planning years versus year. The area under this curve is the total expected discounted value of the damages for each scenario over all years. This results in the total expected discounted value of the damages of the low SLR scenario over the period 2013 to 2063 to be $2.15 M. The high scenario cost is $3.5 M. This is considerably less costly than replacing the Falmouth portion of the bridge for $15 M as described in Section 3.1 to avoid all the damages sometime over the next 50 years. In this case delaying the construction of a higher bridge also is wise because after several decades, there will be clearer indication of the amount of SLR expected. Of course, a similar analysis needs to be undertaken at the end of the bridge's lifetime for this example of 50 years.

Task 4. Step-by-step method for MDOT to collect hydrologic and coastal data and model cost/risk tradeoffs for transportation project needs along the Maine coast.

Sections 2 and 3 have presented the methodology and results for assessing bridge replacement and design for two sites in coastal ME with different tidal regimes. The assessment approach relies upon risk and scenario-based design, which can replace or augment the more traditional event based approach currently used by ME DOT. The use of scenarios is necessary because of future uncertain sea level rise rates. Similar to the event based approach, the methodology described here accounts for the possibilities of damaging floods due to both storm surges and river floods. In our two case studies, we did not carry out scour analysis but the method can be extended to include this. Here we summarize the methodology.

Step 1. Develop flood elevation-frequency curve at site

a. Analysis of Tide gage data

This is necessary to determine the frequency of present and future storm surges. Historical data tide can be obtained for the station nearest to the site from the NOAA Tides and Currents website: http://tidesandcurrents.noaa.gov/station_retrieve.shtml?type=Tide+Data. Water level anomalies, representing storm surge, can be computed by removing tidal influence and historical sea level rise following Kirshen et al. (2008). Frequency analysis is performed on the water level anomalies which are assumed to represent storm surge events. If the nearest gage does not represent the conditions at the site, short-term site measurements can be taken and used to correct observations at the nearest tide gage site at was done for the analysis of the Little Machias bridge (Section 2).

b. Analysis of Streamflow data

If stream discharge may be an influence on the bridge replacement or design, it is necessary to determine the flood frequency at the bridge site. If there are insufficient streamflow data for the site for a flood frequency analysis, another method can be used such as a published regression equation (as was done for the Little Machias bridge analysis) or a regression between annual maximum discharges at a nearby site (as was done for the Martin's Point bridge analysis).

c. Joint Probability Assessment

While we found in our two case studies that joint probability analysis turned out not to be necessary because the surge elevations always dominated the flood elevations independent of the river discharge, we recommend that this be evaluated at each site. This can be done by first establishing the independence of surge and floods by comparing how often river floods coincided with storm surges. If they are not coincident many times, then joint probability of flooding and the resulting elevations can be estimated using the methods in the case studies. If they are coincident a significant number of times, then joint probability can be determined using the methods that incorporate conditional probabilities.

An intermediate step is estimating the flood elevations so they can be compared to the surge elevations. This can be done by using a river model, such as HEC-RAS model calibrated to the selected site. HEC-RAS, a one dimensional river model developed by the Army Corps of Engineers is available at http://www.hec.usace.army.mil/software/hec-ras/. For our analysis of existing conditions, we used the 100-year peak discharge and for future conditions, we increased this discharge by 10%. In each of our two cases, we specified the boundary condition at the downstream end of the model. For instance, in the absence of storm surge or flooding in the Little Machias River, the downstream boundary condition was set at normal depth, using the downstream channel slope. Under storm surge and downstream flooding conditions, the downstream boundary condition was set to equal the storm surge height plus MHHW. In the case of the Martin's Point bridge, which spans a tidal estuary, mean higher high water (MHHW) was used as the downstream boundary condition to establish river flooding elevations in order to be consistent with the storm surge analysis (which also assumed MHHW conditions). Bridge geometry can be provided by the MaineDOT. Channel geometry can be determined from bathymetric maps, elevation contours, and aerial photographs manipulated by GIS.

Step 2. Develop Climate Change Scenarios

a. Sea Level Rise (SLR)

Because of the uncertainty in future climate change, scenarios of plausible ranges of flood elevations over the design life of the bridges must be developed. Estimates of global sea level rise (SLR) are periodically assessed by state, national, and international organizations as well as published by scientists in peer-reviewed disciplinary journals. To these global scenario ranges must be added local and regional changes available from the same reputable sources.

b. Streamflow Changes

Estimating changes in local river flood conditions can vary from estimating a reasonable range of changes based on similar studies (as was done on the two case studies here) or by carrying out hydrology/hydraulic modeling studies using watershed and river models that can incorporate projected future precipitation and temperature scenarios.

c. Flood Elevation Frequencies under Climate Change.

If flood elevations at the site are found to be independent of the river discharge and only dependent upon the surge elevation as was found in the two case studies, then flood elevation frequencies under climate change can be found by adding the scenario SLR and any local or regional changes to the desired tidal conditions. If river and surge flood conditions are statistically related, then conditional joint probability analysis must be carried out under climate change conditions to develop flood elevation frequencies for various climate change scenarios.

Step 3. Cost Analysis

a. Develop Depth – Damage Relationship.

This relates the depth of flooding and associated wave heights to bridge and adjacent roadway damage. Information on this can be obtained from the bridge designer. The range of conditions should cover the range of the flood elevations estimated in Step 2.

b. **Determine Expected Value Damages under Climate Change Scenarios**

This is done in two steps. The first step is for each time period of each climate change scenario summing the area under the curve that relates flood damage cost to its frequency. This results in the expected value of the damages at each time period for each scenario. The second step is summing the area under the curve for each climate change scenario that relates the expected value of flood damage cost to its year of occurrence. This results in the expected value of damage cost over the planning time period for each climate change scenario. To these costs can be added operation and maintenance costs and all costs can be discounted.

c. Determine Benefits of Adaptation

While not done in the two case studies, an additional step could be to estimate the benefits of adapting the bridge to climate change. Benefits for each time period of each climate change scenario can be estimated as the damages that would not occur if an adaptation action was undertaken.

d. Consider Environmental and Social Costs and Benefits

Also while not done in the two case studies, an additional step could be to estimate the costs and benefits of adapting the bridge to climate change using environmental and social metrics such as wetlands protected and improvement in emergency response times.

e. Decision

Using the methods outlined in a-d above, the results of not undertaking and undertaking a variety of adaptation options can be compared for each climate change scenario. Although the selection of the appropriate course of action differs based on a number of factors such as site conditions, and policy and regulatory priorities, the course of action is the one that functions reasonably well under all the climate change scenarios. If no one adaptation action dominates under all the scenarios, then more actions need to analysed until a dominant adaptation action is found.

4. References

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Appendix 1 – Alternative Machias Bridge Replacement Designs (from ME DOT).

Alternative #1 (Replace existing bridge in kind with 4 - 6 'span by 6' rise butted precast concrete box culverts with flapper gates. Invert of the culverts at about elevation 1.0’+/- (Same as existing)

1.) Very minor/slight damage

Wave action up to edge of road (elevation 11'), maybe very a few inches of water on the shoulder of the road - (road passable at al times)

Damage: Minor lose of riprap in isolated locations, removal of light debris, patch pavement along isolated spots along the edge of shoulder

Traffic impacts: Two lanes open to traffic with shoulder closures

Total Repair Cost: Under $25,000 +/-

Causeway only costs: $25,000

Structure costs: $0

2.) Minor damage

Wave action with run-up just over the shoulder, maybe 6"of water over the road (elevation 12' to 13')

Damage: Minor lose of riprap along the entire slope, removal of debris, patch pavement along spots/strip along the edge of shoulder, possible minor damage to flapper gates

Traffic impacts: Possible short road closure at height of storm for a few hours. During repairs two lanes open to traffic with shoulder closures

Total Repair Cost: $25,000 -$50,000 +/-

Causeway only costs: $20,000 to $40,000

Structure costs: $5,000 to $10,000

3.) Medium/Moderate damage

Moderate wave action with run-up over the road, maybe 1 '+/-of water over the road (elevation 14’ to 15') +/-

Damage: Moderate lose of riprap along entire slope, lost of fill around the corners of the bridge and in isolated spots on the side slopes, removal of heavy debris, lose of shoulder in isolated locations

Traffic impacts: Road closure at the height of the storm for a 1/2 day to 2 days. During repairs one lane open to traffic with flaggers

Total Repair Cost: $50,000 to $200,000+/-

Causeway only costs: $40,000 to $180,000

Structure costs: $10,000 to $20,000

4.) Serious damage

Strong wave action with run-up over the road, several feet '+/-of water over the road (elevation 15' +/-)

Damage: Significant lose of riprap along entire slope, lost of fill around the corners of the bridge and along the entire length of the causeway on the side slopes, removal of heavy debris, lose of entire shoulder and parts of the travel lane, one or more flapper gates damaged or missing

Traffic impacts: Road closure at height of storm for a 1/2 day to 4 days. During repairs one lane is open to traffic with flaggers

Total Repair Cost: $100,000 -$400,000+/-

Causeway only costs: $70,000 to $350,000

Structure costs: $30,000 to $10,000

5.) Severe damage

Very strong wave action with run-up over the road, multiple feet of water over the road (elevation 15' +)

Damage: Culvert failure with significant lose of fill around the culvert. Significant damage to the causeway, lose of most of the riprap along entire slope

Traffic impacts: Road not passable for many days maybe even a week or longer to install a temporary detour.

Total Repair Cost: $500,000 to $1,500,000+/-

Causeway only costs: $200,000 to $500,000

Structure costs: $300,000 to $1,000,000

Alternative #2 (Replace existing bridge with a single span bridge with integral abutments supported on piles driven to bedrock

1.) Very minor/slight damage

Wave action up to edge of road (elevation 11'), maybe very a few inches of water on the shoulder of the road - (road passable at al times)

Damage: Minor lose of riprap in isolated locations, removal of light debris, patch pavement along isolated spots along the edge of shoulder

Traffic impacts: Two lanes open to traffic with shoulder closures

Total Repair Cost: Under $25,000 +/-

Causeway only costs: $25,000

Structure costs: $0

2) Minor damage

Wave action with run-up just over the shoulder, maybe 6" to 1' +/- of water over the road (elevation 12' to 13')

Damage: Minor lose of riprap along the entire slope, removal of debris, patch pavement along spots/strip along the edge of shoulder

Traffic impacts: Possible short road closure at height of storm for a few hours. During repairs two lanes open to traffic with shoulder closures

Total Repair Cost: $25,000 -$50,000 +/-

Causeway only costs: $25,000 to$50,000

Structure costs: $0

3.) Medium/Moderate damage

Moderate wave action with run-up over the road, maybe 1 '+/-of water over the road (elevation 14' to 15') +/-

Damage: Moderate lose of riprap along entire slope, lost of fill around the corners of the bridge and in isolated spots on the side slopes, removal of heavy debris, lose of shoulder in isolated locations, minor damage to bridge fascia due to debris impacts

Traffic impacts: Road closure at height of storm for a 1/2 day to 2 days. During repairs one lane open to traffic with flaggers

Total Repair Cost: $50,000 to $200,000+/-

Causeway only costs: $40,000 to $180,000

Structure costs: $10,000 TO $20,000

4.) Serious damage

Strong wave action with run-up over the road, several feet '+/-of water over the road (elevation 15' +/-)

Damage: Significant lose of riprap along entire slope, lost of fill around the corners of the bridge and along the entire length of the causeway on the side slopes, removal of significant heavy debris, lose of entire shoulder and parts of the travel lane, buoyancy forces my have lifted bridge up slightly,

Traffic impacts: Road closure at height of storm for 1/2 day to 5 days. During repairs one lane open to traffic with flaggers

Total Repair Cost: $100,000 to $400,000+/-

Causeway only costs: $70,000 to $350,000

Structure costs: $30,000 to $50,000

5.) Severe damage

Very strong wave action with run-up over the road, many feet '+/-of water over the road (elevation 15' +)

Damage: Bridge damage could range from minor to possible lose of the superstructure due to water forces. The abutments should remain intact with pilings exposed. Significant damage to the causeway, lose of most of the riprap along the entire slope, lose of entire shoulder and parts of the travel lane,

Traffic impacts: Road not passable for days maybe even weeks or longer to install a temporary detour.

Total Repair Cost: $500,000 to $3,000,000+/-

Causeway only costs: $200,000 to $750,000

Structure costs: $300,000 to $2,250,000