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Improving scour risk management for Iowa by utilizing improved flood frequency estimates and modified HYRISK

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Improving scour risk management for Iowa by utilizing improved flood frequency estimates and modified HYRISK

by

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ABSTRACT

Two approaches can be utilized for determination of the design flood discharge depending on the presence of the USGS gages in the streams that bridges are built on. For bridges on gaged streams, the annual peak discharge data can be used in order to estimate the design discharge. For ungaged sites however, such data is not available and the only possible method is the use of regional flood frequency models that are developed based on estimations of gaged data. The high prediction errors associated with such models along with the fact that climate change and urbanization can also undermine accuracy of the estimated discharges, motivated this research. It was assumed that the most recent available flood prediction model was used by Iowa DOT for determining design discharge at the time of bridge construction. In this regard, the estimated bridge design discharge was compared with the latest estimate of the same flood event. The results showed that as the basins get larger, the estimated discharges are more reliable. It was also concluded that bridges built before 1980s are more prone to experience an increase in their estimated discharges.

Floods and resulting scour are responsible for about half of bridge failures in the United States. Catastrophic consequences of bridge failures along with guidelines from the Federal Highway Administration (FHWA) motivated the development of scour assessment tools. HYRISK is one of the available tools for network-level scour analysis and is developed by the FHWA for prioritizing bridges based on their expected scour risk. This study proposed three major modifications to improve and customize HYRISK estimations for Iowa. Soil erodibility was incorporated into the HYRISK along with a modified failure cost calculation accounting for scour countermeasure installation cost rather than bridge reconstruction that was originally being considered. The modified HYRISK was used to estimate the annual cost of scour risk for Iowa DOT bridge network and also the damage to the affected bridges by the 2008 flood in Upper Mississippi River basin. The results were significantly different from original HYRISK estimations and were in line with the actual annual expenditure on scour maintenance program and also the reported damage from the 2008 flood.

CHAPTER 1. GENERAL INTRODUCTION

Scour at Bridges

Scour is the erosion of the soil material due to flowing water from around the bridge piers and abutments. If scour is not treated, it can cause serious threats to the bridge or even failure. Half of the bridge failures in U.S. are due to flooding and scour and considering the high consequences of bridge failures, addressing scour risk is one of the most critical tasks of state Departments of Transportation (DOT). Following the requirements of the Federal Highway Administration (FWHA), existing bridges as well as new bridges should be evaluated and designed for scour. In this regard, many tools were developed for assessment of individual structures as well as a network of bridges. HYRISK, developed by FHWA, is one the tools for network-level scour assessment. The results from HYRISK can be used by managers to prioritize bridges in their network for scour management.

One of the most critical steps toward scour assessment is determining the scour design discharge, which is normally a 100-year or 200-year flood for more important structures. For bridges located on gaged streams, the design discharge can be developed by using the historical annual peak discharges. However, for bridges on ungaged streams, the only way of determining design discharge is using the regional flood frequency models that are based on estimated discharges from the gages on other streams.

Previous studies showed that factors such as climate change, developed agriculture, urbanization, and changed land-use can undermine the accuracy of the estimated discharges. However, flood frequency models are the only available source for Iowa DOT regarding determination of the design discharge for bridges on ungaged streams. The results from an underestimated discharge can be devastating and therefore, a systematic approach needs to be used for identifying bridges that are more prone to experiencing a change in their estimated discharge.

Research Motivation

The state of Iowa has numerous small and large streams with a precipitation average of around 34 inches annually. Majority of the 3,325 state-owned bridges are located on waterways and the annual cost of scour management and maintenance is estimated to be around one million dollars.

Iowa DOT is currently monitoring its scour critical bridges by using a web-based program, called BridgeWatch. This online alert system gathers real-time data from USGS streamgages for current water surface level, from SNOTEL (Snow Telemetry) sensors for snow melting, and from NEXRAD (Next Generation Weather Radar) system for predicting precipitation. An alert would be sent to Iowa DOT officials and corresponding personnel whenever water surface at monitored bridges reaches its critical level or a significant discharge (at least 25-year flood) is anticipated. Use of BridgeWatch system enabled Iowa DOT to proactively monitor its scour critical bridges and concentrate the personnel and effort only on critical sites before occurrence of major flood events. Although BridgeWatch program helps Iowa DOT to be more proactive in response to floods, it cannot identify scour critical bridges. One of the goals of this study is to modify HYRISK program based on Iowa DOT experiences and policies regarding scour management in order to identify aspect of bridges that make them vulnerable to scour. The results will be helpful for Iowa DOT for prioritizing their investments more efficiently.

Another motivation of this study is investigating the accuracy and reliability of the design flood discharge used by Iowa DOT for hydraulic and scour design of the bridges. The biggest concern are bridges on ungaged streams where no available verification means are available and the only possible tool for determining design discharge is using available flood frequency models.

Thesis Organization

This thesis is divided into four chapters. Chapter 1 covered the introduction to the research and the research motivation. The remaining chapters are discussed below.

Chapter 2 corresponds to the assessment of the accuracy of scour design discharges for on-waterway bridges in Iowa. In this regard, four available regional flood frequency models were used for estimating the design discharge at the time of bridge construction. Finally, the bridges that are more prone to be under-designed were identified.

Chapter 3 is a research done on identification of the HYRISK limitations and modifying it to be more applicable in Iowa. Soil erodibility along with a new adjustment factor and a modified failure cost calculation method were incorporated into the HYRISK. Results from the modified HYRISK can improve Iowa DOT decision making regarding bridge scour management.

Lastly, Chapter 4 is a summary of the conclusions drawn from the two studies and also the impacts of this study on the overall asset management practices.

CHAPTER 2. ASSESSING ACCURACY OF THE REGIONAL FLOOD FREQUENCY MODELS ON IOWA BRIDGE SCOUR MANAGEMENT

A paper to be submitted to *The Journal of Infrastructure Systems*

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Abstract

One of the most critical steps in bridge scour assessment is the determination of the design flood discharge. For this purpose, two approaches can be utilized depending on the presence of the U.S. Geological Survey (USGS) gages in the streams that bridges are built on. For bridges on gaged streams, the annual peak discharge data can be used in order to estimate the design discharge. For ungaged sites however, such data is not available and the only possible method is the use of regional flood frequency models that are developed based on estimations of gaged data. The high prediction errors associated with such models along with the fact that climate change and urbanization can also undermine accuracy of the estimated discharges, motivated this research. It was assumed that the most recent available flood prediction model was used by Iowa DOT for determining design discharge at the time of bridge construction. In this regard, the estimated bridge design discharge was compared with the latest estimate of the same flood event. The results showed that as the basins get larger, the estimated discharges are more reliable. Especially, estimated discharges for bridges with drainage areas smaller than 30 mi^2 were found to be less accurate. It was also concluded that bridges built before 1980s are more prone to experience an increase in their estimated discharges. Therefore, the consequences of an underestimated design discharge can be critical for bridges with mentioned aspects.

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Introduction

U.S. Geological Survey (USGS) have been installing gages on streams all over the US in the past decades in order to collect water elevation and discharge data. By using the collected annual peak discharges, magnitudes of floods with various return periods can be predicted which are unique for each site. In order to assess the flood frequencies for ungaged sites, a regional flood frequency model can be developed based on discharges of the gaged sites. Therefore, without using any historical discharge data, the regional flood frequency models can be used for predicting flood discharges for both gaged and ungaged stream sites in the region.

Flood frequency models are being updated every 10 to 15 years in order to utilize recently collected data as well as using newer and more accurate statistical models. Therefore, each model might have its own set up and might require different parameters for estimating the flood discharges. Generally, earlier models are only based on drainage area of the basins and as the knowledge about floods becomes more mature and more data is available, new parameters such as slope and rainfall intensity have become essential parts of the models. For Iowa, six different frequency models were developed in the past decades. However, only four of them provided equations for estimating the 100-year flood discharge which is used widely for bridge design. These four models were published in years 1973, 1987, 2001, and 2013.

The latest Federal Highway Administration (FHWA) guidelines (*1*) recommend the use of flood discharge with a maximum return period of 200 years for bridge scour design and 500 years for scour check. Also use of magnitudes of other flood events might be required for other hydraulic design purposes. Main sources for Iowa DOT regarding the determination of flood discharges are the regional flood frequency models. Therefore, it was assumed that the most recent available model was used at the time of bridge design. It is possible that different models lead to different estimates of the same event for a specific site. This study aimed to identify groups of bridges that experienced higher changes of design discharge by comparing estimations of the past models for Iowa.

Literature Review

One of the main assumptions toward modeling streamflow at gaged sites is that floods are stationary and will not change in the future. The most used approaches for checking the validity of this assumption are trend analysis, where gradual changes of discharges are being identified, or identifying change-points where sudden changes in mean or variance are observed. Numerous researchers investigated the trends of annual peak discharges and conflicting results are reported in the literature. Some studies did not detect any significant trend in flood series, however, change-points reflecting abrupt changes in peak discharges due to change of land use, agricultural development, and urbanization were identified (*2*–*5*). Novotny et al. (*6*) studied flow records from 36 streamgages in Minnesota and showed increased frequency rather than intensity of the flood events due to climate change. Other studies also investigated effect of the climate change on floods and identified increase in both frequency and intensity of the floods (*7*, *8*). Therefore, it is possible that the estimated flood discharge for one specific site may change by passing time due to either climate change or urbanization.

Regional flood frequency models are based on estimated stream peak flows for gaged sites. As it was mentioned, streamflow at gaged sites is possible to change over time due to various factors. Rather than estimation error associated with predictions of regional flood frequency analysis, Leclerc and Ouarda (*9*) concluded that if nonstationarity exists among data, ignoring it would result in significant under- or overestimation of flood quantiles. One of the 19 FHWA pilot projects for climate change vulnerability assessment was done in Iowa (*10*). Two basins of Cedar River and South Skunk River with 50 years of historical data were studied. The results however, failed to reject the stationarity assumption of the data. Other basins yet need to be investigated and the stationary assumption needs to be assessed.

The following sections describe the regional flood frequency models that were used in this study.

1973 model

The oldest model that was used is developed by Lara based on 1972 water data from 136 streamgages having drainage area greater than 2 mi^2 (*11*). For addressing the difference between various locations of streamgages, two hydraulic regions were developed (Figure 2.1) and single-variable and two-variable regression equations were developed for each.

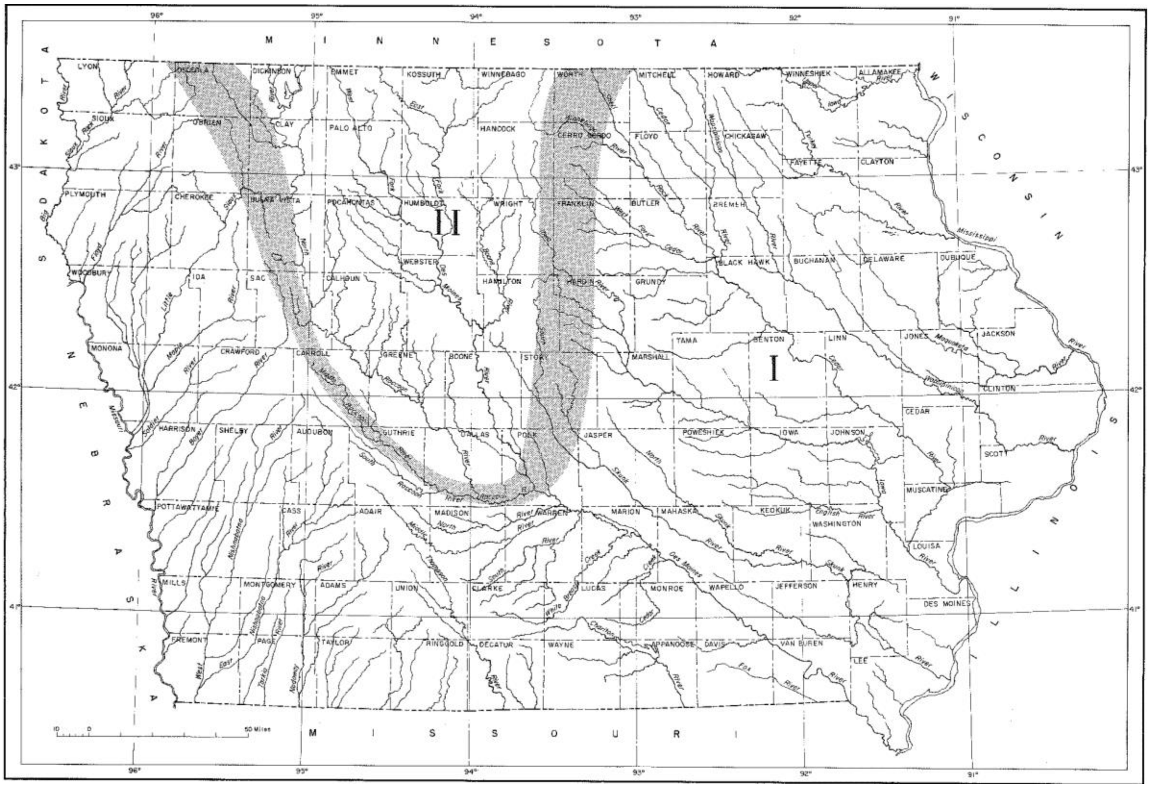

Figure 2.1 Hydraulic regions of the 1973 regional flood frequency model for Iowa (source: Iowa Natural resource Council Bulletin 11)

The developed regression equations can predict six Annual Exceedance-Probability Discharges (AEPD) and the largest discharge that can be estimated is 100-year flood with the standard error of 26 to 44 percent. Table 2.1 shows the coefficients for predicting 100-year flood in Equation 1 where Q_{100} is the discharge of 100-year flood, A is drainage area, S is the main channel slope, and C, X, and Y are model coefficients.

$$Q_{100} = C(A)^X(S)^Y \quad (1)$$

Table 2.1 Coefficients for the 1973 model

	Regression Type	C	X	Y	Std. error (%)
Region 1	One-variable	1,800	0.421	0	46
	Two-variable	571	0.524	0.305	44
Region 2	One-variable	212	0.642	0	34
	Two-variable	-	-	-	-

1987 model

This model was developed by using 1984 water data from 263 streamgages (*12*). For this model, five hydraulic regions were defined and for each region, a one-variable regression equation was developed for estimating discharges as large as Q_{100}. The average standard error for this model ranges from 24 to 41 percent. Figure 2.2 shows the locations of streamgages as well as the hydraulic regions of this model.

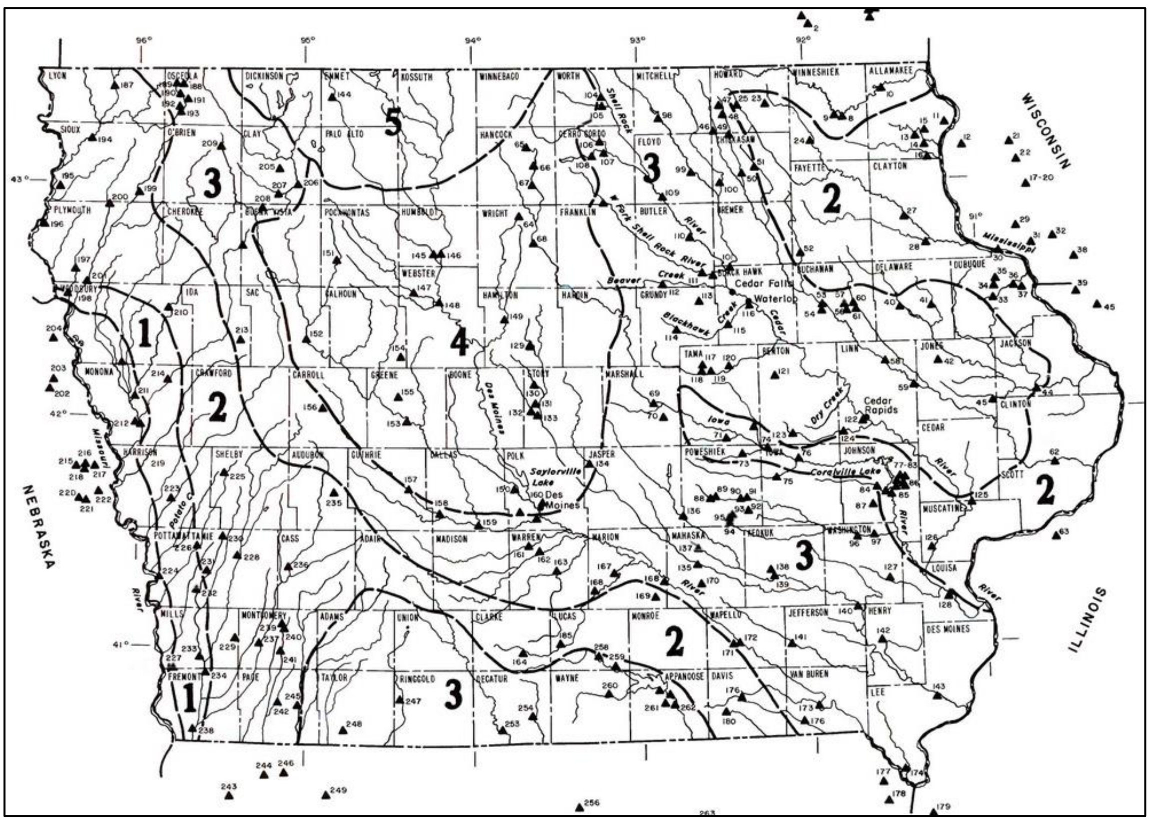

Figure 2.2 Hydraulic regions of the 1987 regional flood frequency model for Iowa (source: USGS WRI Report 87-4132)

For each region, the developed regression equation should be used for calculating 100-year flood discharge. The equations are only based on drainage area on the basins and Equation 2 describes the general form of the model.

$$Q_{100} = C(A)^B \tag{2}$$

Where A is drainage area of the basin, Q_{100} is 100-year flood discharge, and C and B are model coefficients. Table 2.2 shows the coefficients for five hydraulic regions for estimating 100-year floods.

Table 2.2 Coefficients for the 1987 model

Region	Number of gages	C	B	Std. error (%)
Region 1	19	1,880	0.60	24
Region 2	81	1,230	0.53	36
Region 3	119	851	0.53	41
Region 4	24	227	0.65	30
Region 5	8	50	0.80	26

2001 model

The third model was developed by Eash et al. (*13*) using 1997 water year data of 291 streamgages. There were three hydraulic regions defined for this model and for each region, single and multi-variable regression equations were developed for estimating discharges of flood events as large as 500-year floods. Also, the average standard error for equations ranges from 30.8 to 42.7 percent. Figure 2.3 shows the three regions of this model.

Availability of required data and newer tools, helped the development of three-variable regression equations for this model. Equation 3 is the general form of the regression equations used for 2001 model.

$$Q_{100} = C(DA)^X(MCS)^Y(DML+1)^Z \tag{3}$$

Where DA is drainage area, MCS is main channel slope, DML is ratio of the basin area within Des Moines Lobe, and C, X, Y, and Z are model coefficients that are shown in Table 2.3.

As expected, multi-variable equations have less estimation error compared to single-variable ones. Also, Table 2.3 shows that estimation accuracy of equations is higher than their prediction. Which means for the sites that have one or more required parameters not falling into the estimation range, the error associated with calculated discharge is larger.

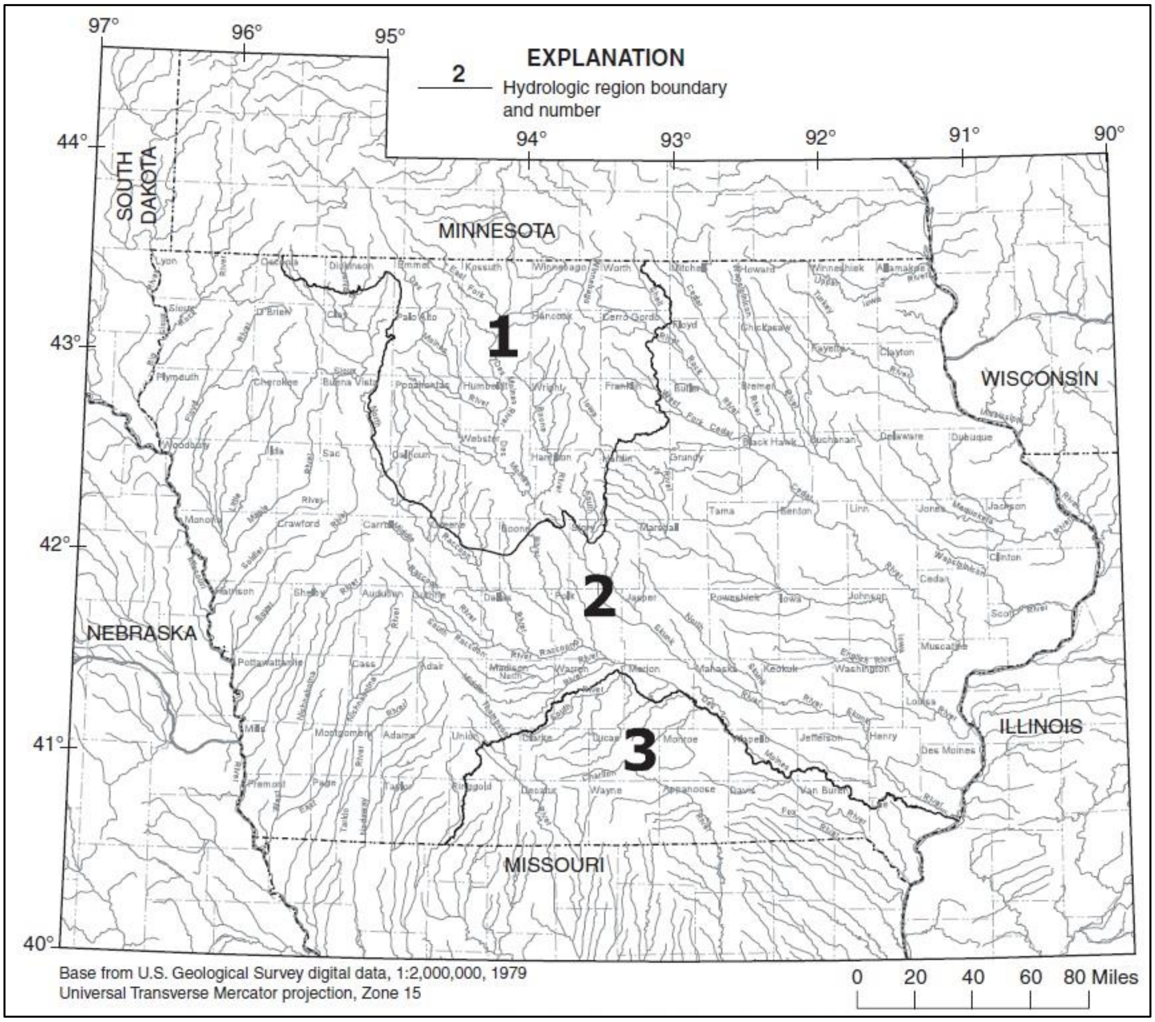

Figure 2.3 Hydraulic regions of the 2001 regional flood frequency model for Iowa (source: USGS WRI Report 00-4233)

Table 2.3 Coefficients for the 2001 model

	Regression Type	C	X	Y	Z	SEE (percent)	SEP (percent)
Region 1	One-variable	141	0.669	-	-	33.1	40.5
Region 2	One-variable	1,800	0.415	-	-	26.8	35.6
	Three-variable	531	0.542	0.313	-0.549	22.6	32.9
Region 3	One-variable	3,300	0.357	-	-	24.3	35
	Two-variable	158	0.652	0.809	-	18.6	31.6

2013 model

The latest model is developed in 2013 by using 2010 version of water data from 518 streamgages (*14*). For each site, peak discharges were estimated by using Pearson Type III distribution and the results were used to develop regional regression equations for estimating flood discharges with the return periods of 2,5,10, 25, 50, 100, 200, and 500 years. Total of six flood regions were defined for this model while only three of them are in Iowa (Figure 2.4) and the rest are completely outside of Iowa. The average standard error of predicting Q_{100} varies from 22.3 to 38.0 percent.

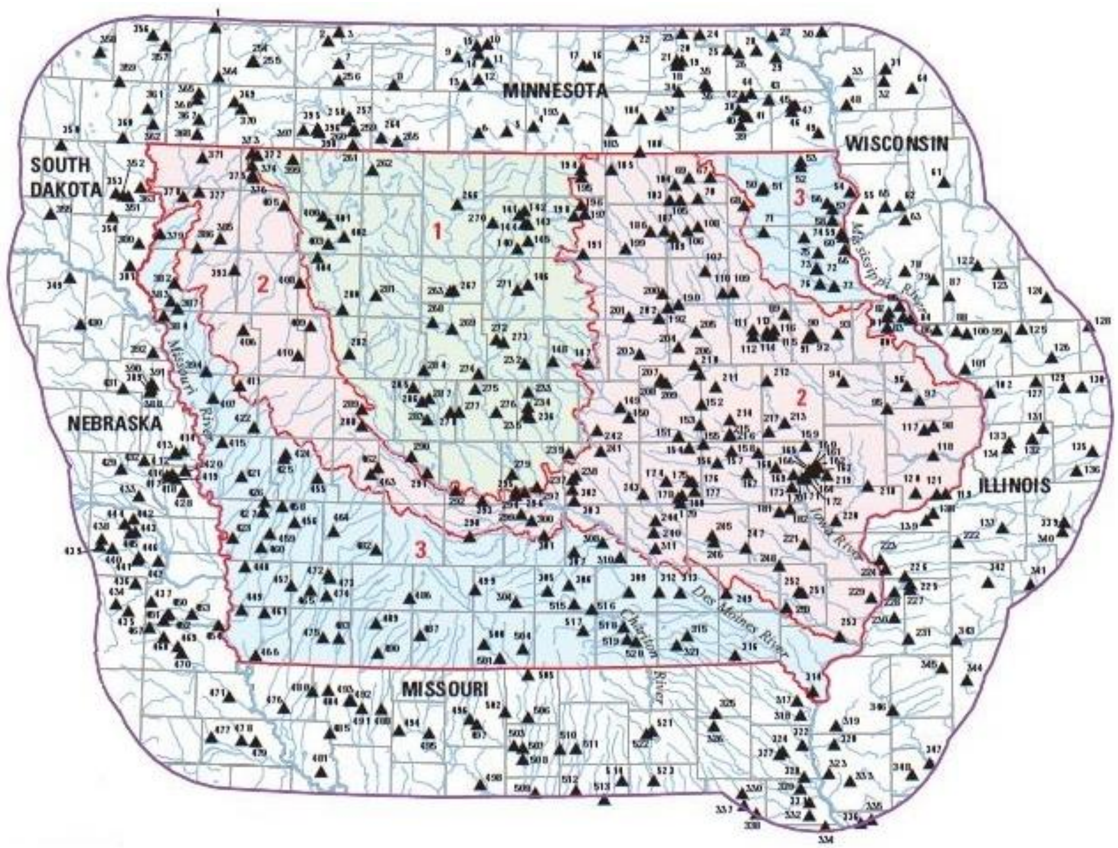

Figure 2.4 Hydraulic regions of the 2013 regional flood frequency model for Iowa (source: USGS SI Report 2013-5086)

Due to high number of available basin parameters, regression equations have different setups for each region. Table 2.4 gives a summary of equations and associated errors with them. Where *SEP* is average standard error of prediction, *SEM* is average standard error of model, *DA* is drainage area in mi^2, *CCM* is constant of channel maintenance in mi^2/mi, *DESMOIN* is percent area underlain by Des Moines Lobe, *I24H10Y* is the maximum 24-hour precipitation that occurs on average once in

10 years in inches, *BSHAPE* is a dimensionless basin shape factor for area, and *KSATSSUR* is the saturated hydraulic conductivity of the soil in micrometers per second.

Table 2.4 Coefficients for the 2013 model (source: USGS SI Report 2013-5086)

	Regression Type	**Regression Equation for Q_{100}**	**SEP (%)**	**SEM (%)**
Region 1	One-variable	$DA^{0.524}10^{2.67}$	-	-
	Three-variable	$DA^{0.566}10^{(0.917+0.567\times I24H10Y-0.742\times CCM^{0.55})}$	38.0	34.7
Region 2	One-variable	$DA^{0.453}10^{3.18}$	-	-
	Three-variable	$10^{(11.1-7.92\times DA^{-0.031}-0.002\times DESMOIN-0.025\times BSHAPE)}$	22.3	20.3
Region 3	One-variable	$DA^{0.455}10^{3.25}$	-	-
	Three-variable	$10^{(6.41-3.06\times DA^{-0.097}-0.009\times KSATSSUR-0.035\times BSHAPE)}$	29.1	27.2

Methodology

In order to assess the accuracy of the estimated flood discharges, this study compares the estimates of the 100-year flood discharges at the time of the bridge construction with the latest available estimates. For determining the original discharge, built years of the bridges were compared with publication years of the model reports and it was assumed that the latest model was used for computing the discharge at the time of construction. Iowa DOT has recently (since 2016) started using the 2013 flood prediction model for designing bridges, therefore, the results from this model were considered as a reference point to be compared with previous predictions. Also, since the oldest flood prediction model for Iowa was developed in 1973, for being conservative, all the bridges built before 1973 were evaluated based on that model.

Obtaining data from StreamStats

StreamStats is one of the most comprehensive available tools for calculating the basin characteristics (*15*). StreamStats is an online application available through USGS website and provides a wide range of tools and resources for hydrological purposes using Geographical Information System (GIS). This tool has the capability of both computing the basin characteristics and estimating the streamflow statistics for any user-specified point along streams. For this goal, StreamStats uses the equations and procedures obtained from Scientific Investigations Report 2013–5086 (*14*).

StreamStats uses a grid representing streams and drainage system of Iowa. For requesting basin characteristics for any site, the site location should be snapped to the points of that database. If the point is not exactly on the defined streams, no results will be calculated for that point. Therefore, before submitting a request, all bridges should be manually checked to verify that they are exactly located on defined streams and if needed, the location should be modified. It also should be checked that the adjusted location of the bridge is located on the main stream, not the smaller side streams that merge into main and larger ones. This issue is more pronounced in bridges located on larger streams since bridges are represented as points in NBI and with larger streams, the point can be located not exactly on the river centerline, but near it where there is the possibility of smaller streams merging into the bigger one.

In general, the defined streams are quite accurate and most of the bridges are within 75 feet of them. Figure 2.5 shows the grid of streams and also one bridge that its location needs to be adjusted.

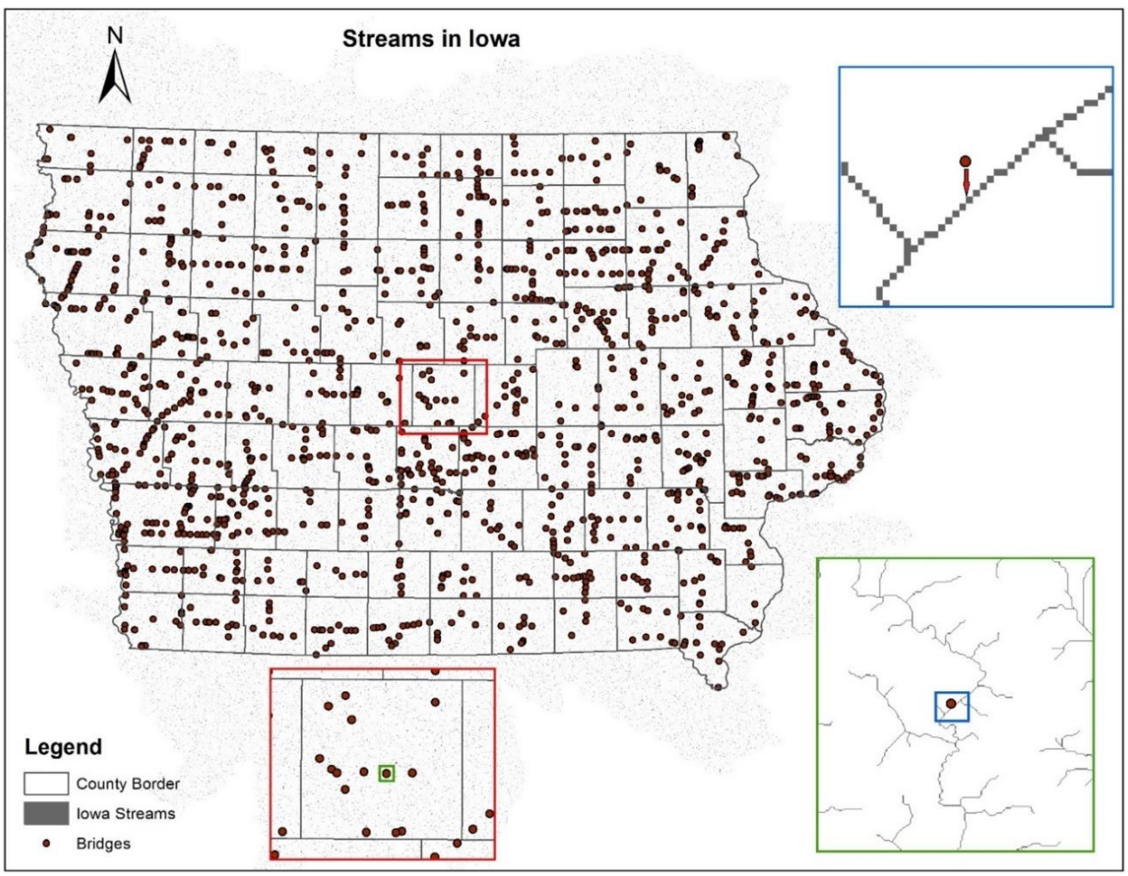

Figure 2.5 Study bridges and StreamStats database of Iowa streams

Due to the high number of on-waterway bridges (total of 1889 bridges), StreamStats Batch Processing Tool was used where a batch of maximum 200 points, in this case bridge locations, are being uploaded. The tool automatically delineates the basin, calculates requested basin characteristics and, if requested, estimates the discharges of various flood events. Therefore, considering the number of request points, total of ten requests were submitted to StreamStats Batch Processing Tool and then the results were combined (Figure 2.6). The requested parameters were drainage area, rainfall intensity, slope, basin shape factor, saturated hydraulic conductivity, constant of channel maintenance, and area underlain by Des Moines Lobe landscape. Also, the estimated discharges for floods with return periods of 2, 5, 10, 25, 50, 100, 200, and 500 years were requested.

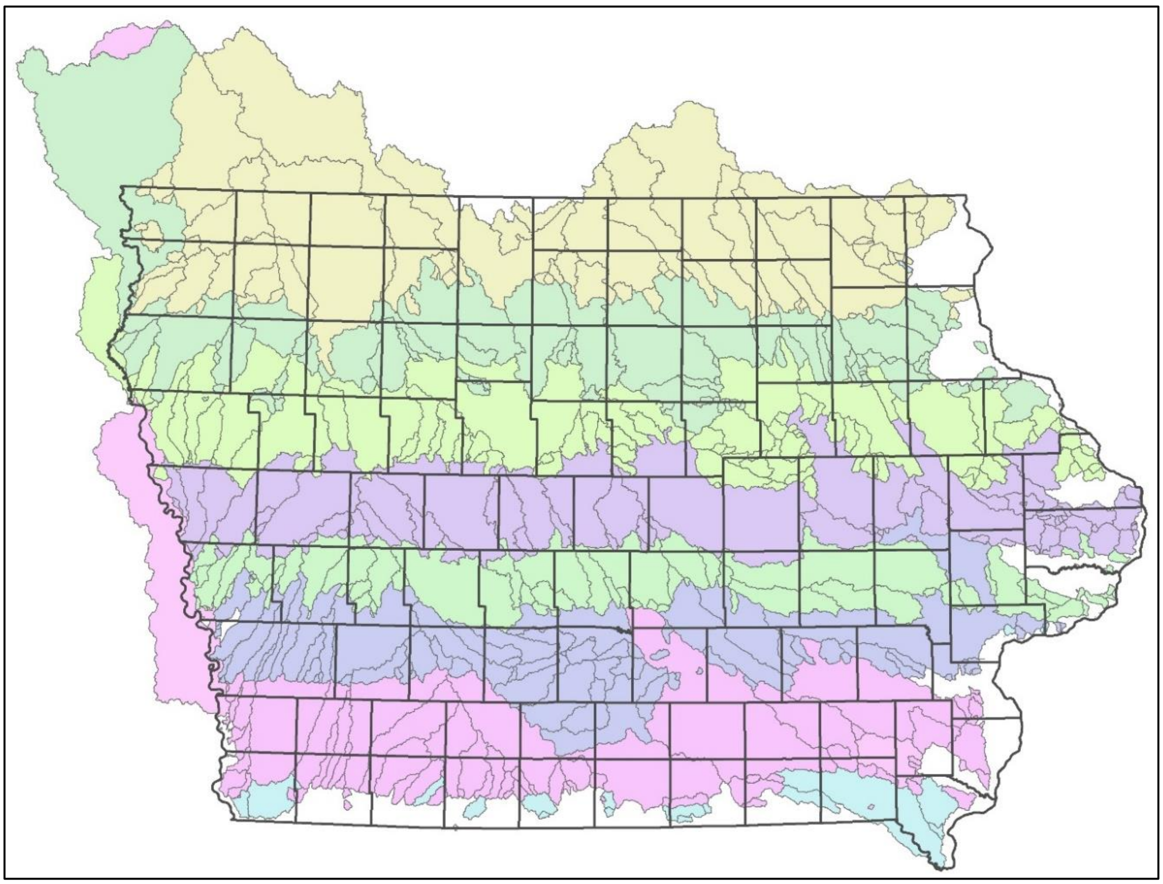

Figure 2.6 Bridge basins delineated by StreamStats

Figure 2.6 shows the location and area of the bridge basins. Each basin, has the requested parameters calculated for it and the different colors are representing different request lists submitted to the Batch Processing Tool. A total of 1828 out of 1889 bridges were successfully analyzed.

Determining difference of the estimated discharges from previous models

The acquired data from StreamStats were used for predicting design discharge for already built bridges (Figure 2.7). By assuming that basins have remained unchanged during the past decades, four different estimates of a 100-year flood were calculated and depending on the construction year of the bridge, the most recent one was compared with the 2013 estimate.

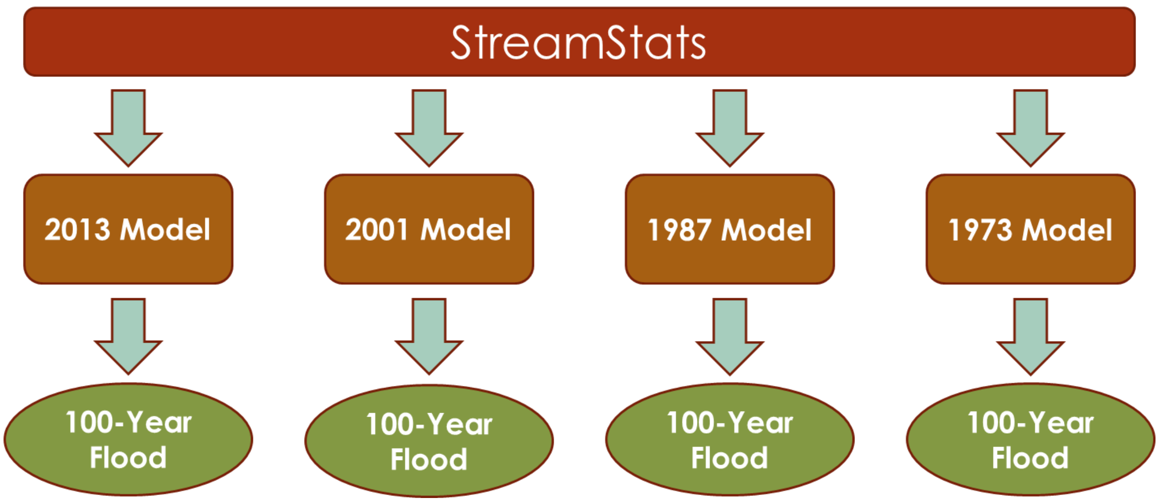

Figure 2.7 Procedure of estimating flood discharges from the available flood frequency models in Iowa

It should be noted that larger basins may fall into several regions and in that case, area-weighted discharge should be calculated. As shown in the following equation, the area-weighted discharge is the sum of estimated discharges for each hydraulic region multiplied by its percentage area (P_i).

$$Weighted\ Q_{100} = \sum_i Q_{100_i} \times P_i \quad (4)$$

The procedure for the application of flood prediction models is as follows:

1. Determine the boundaries of the basin
2. Determine the required parameters (such as drainage area and slope).
3. Obtain the current 100-year flood discharge from StreamStats.
4. Determine percentage of the drainage area falling into each hydraulic region.

5. Compute 100-year flood discharge by using appropriate equations for each hydraulic region.
6. Calculate the area-weighted discharge by using Equation 4.
7. Determine the change in design discharge by comparing the result from step 5 with the current one obtained from StreamStats.

There are several cases that can impact the procedure of calculating the difference in design flood discharge. The cases are explained in the next four sections.

Border bridges

Some bridges are located on Iowa borders such as Mississippi River on the border of Iowa, Illinois, and Wisconsin, and Missouri River for Nebraska and South Dakota. The basins of border bridges fall into neighboring states and therefore, the delineation process done by StreamStats would not be as accurate. Also, the flood estimation models are developed specifically for Iowa and are not applicable for other states.

A total of 26 bridges were identified as border bridges by using NBI Item 98 (Border Bridges) and they were excluded from the assessment. Table 2.5 shows number of bridges on the Iowa borders.

Table 2.5 Bridges on the border of Iowa and other neighboring states

	Neighboring State				
	Illinois	**Missouri**	**Nebraska**	**South Dakota**	**Wisconsin**
Number of Bridges	7	4	10	4	1

Bridges that already have scour countermeasures

The first FHWA guidance for bridge scour evaluation was published at 1988 and many states started evaluating their bridge network vulnerability against scour and develop a plan of action for the critical ones. Iowa DOT evaluated its bridges in early 2000 and since then scour countermeasures have been implemented. Therefore, it can be assumed that for designing scour countermeasures, Iowa DOT utilized the most recent available flood prediction model that was published in 2001. Therefore, that model was used for all bridges that NBI item 113 is coded as 7 as well as the ones that were manually reviewed and the presence of scour countermeasure was verified for. The summary of bridges with scour countermeasures is shown in Table 2.6.

Table 2.6 Bridges with countermeasures

	Type of protection					
	NBI Item 113 coded as 7			All bridges		
	Pier protection	Abutment protection	Total	Pier protection	Abutment protection	Total
Number of bridges	92	112	119	320	592	626

Bridges on very small streams, overbanks, and lakes

StreamStats has a database of drainage system of Iowa and for requesting basin characteristics for any site, the site location should be snapped to the points of that database. Unfortunately, the database does not include very small streams, small lakes, and overbanks. Therefore, basin delineation and flood discharge estimation cannot be done for those locations and even if it was possible, the resulting basin would be very small (less than 0.02 mi^2) with high associated estimation error. Therefore, this group of bridges was not assessed and they were coded as "No Estimate".

Bridges with extreme basin characteristics

Each flood prediction model requires a set of basin parameters that should be determined for each site before applying. For bridges that their basin characteristics do not fall into the range of model input data, the prediction error is significantly higher. Therefore, flood discharges for those bridges should not be calculated and they were coded as "Not Applicable". Only 2001 model reported its input range and therefore assessing the applicability of previous models was not possible.

A total of 451 bridges that were built or reconstructed after 2001 and therefore, 2001 model was used for their discharge assessment. After evaluating the bridges, it was found that the model is not applicable for 18 of them and they were coded as "Not Applicable".

Results and Discussion

Data Summary

StreamStats Batch Processing Tool was used for determining bridges' basin data. Depending on location of the bridge and the stream it overpasses, drainage area and other parameters can change significantly. Table 2.7shows the descriptive statistics of the parameters obtained from StreamStats.

Table 2.7 Summary of the basin characteristics

Parameter	Average	Max	Min	Std. deviation
Slope (ft/mi)	15.92	314.51	0	20.25
Drainage Area (mi²)	524.5	15189.4	0	1852.0
Maximum 24-Hour Precipitation that Occurs on Average Once in 10 Years (inch)	4.29	5.00	0	0.72
Constant of Channel Maintenance (mi²/mi)	1.34	179.21	0	7.46
Area Underlain by Des Moines Lobe Landscape (percent)	21.08	100	0	38.34
Basin Shape Factor for Area (dimensionless)	3.75	18.45	0	2.54
Saturated Hydraulic Conductivity (micrometer/sec)	10.0	247.3	0	7.5

For some parameters such as slope or drainage area, the minimum value of zero is unrealistic. However, since several hydraulic regions are developed for every flood frequency model and every parameter is not required for all of them, StreamStats might only calculate the required parameters and leave others as zero. For example, 2013 flood prediction model does not require Saturated Hydraulic Conductivity for basins located in hydraulic region 1 and therefore, it is possible that for those basins, the estimated Saturated Hydraulic Conductivity be equal to zero.

In general, basins of bridges in Iowa can be significantly different in size and other characteristics. As a result, the standard deviations for some parameters are relatively large compared to the average values. Therefore, the associated error is less when all the bridges in the network are being assessed compared to bridge-level analysis or smaller portion of the network.

Assessing the change of the design discharges

In this study, the difference between the 100-year flood discharge at the time of bridge construction and the current prediction was calculated. For this goal, the procedure explained in the Methodology section was used. The fourth step requires the digitized borders of the hydraulic regions for each prediction model, however, the original shapefiles of the regions were not available. Therefore, they were reproduced by overlaying the available pictures from the reports in ArcMap software and also using the hydraulic unit boundaries available from Iowa DNR GIS Library (*16*). Reproduced borders were tried to be made as similar as possible to the original pictures and also not crossing the existing hydraulic boundaries.

The colored lines in Figure 2.8 through Figure 2.10 represent the reproduced boundaries for the three flood prediction models. It can be seen that in general, the new borders are matching with the original picture.

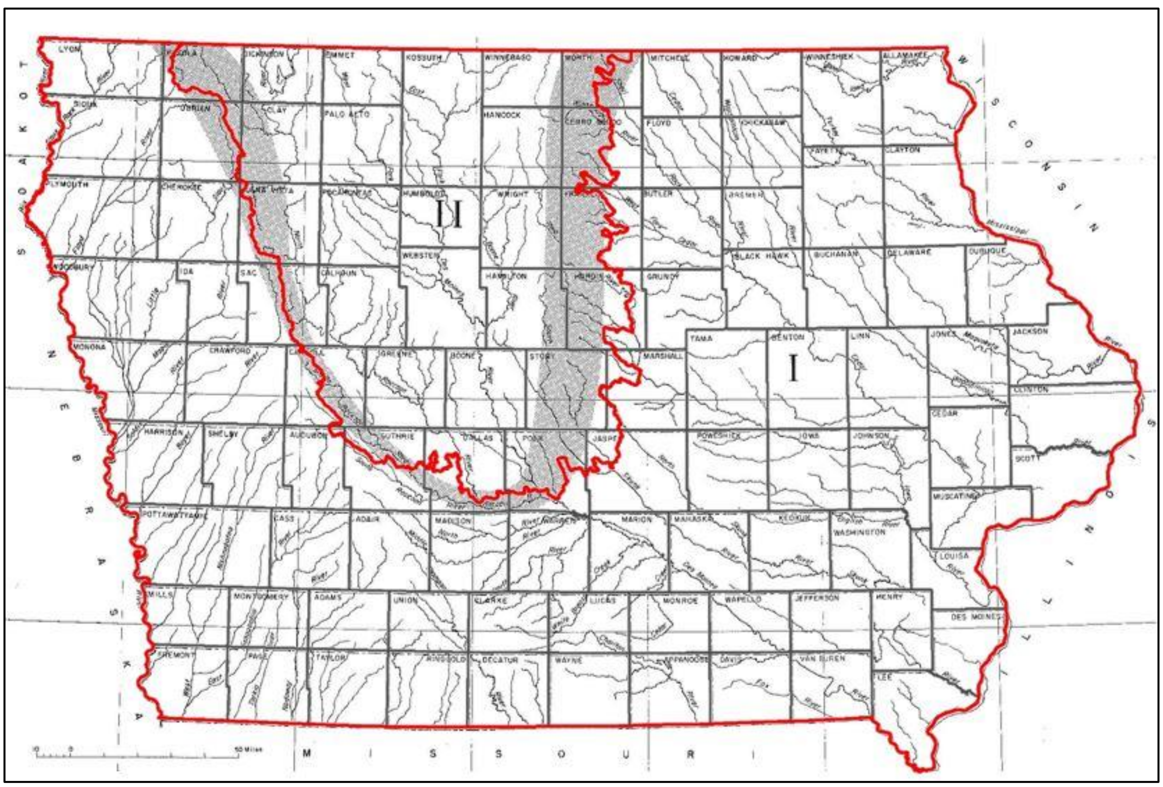

Figure 2.8 Digitized regions for the 1973 model

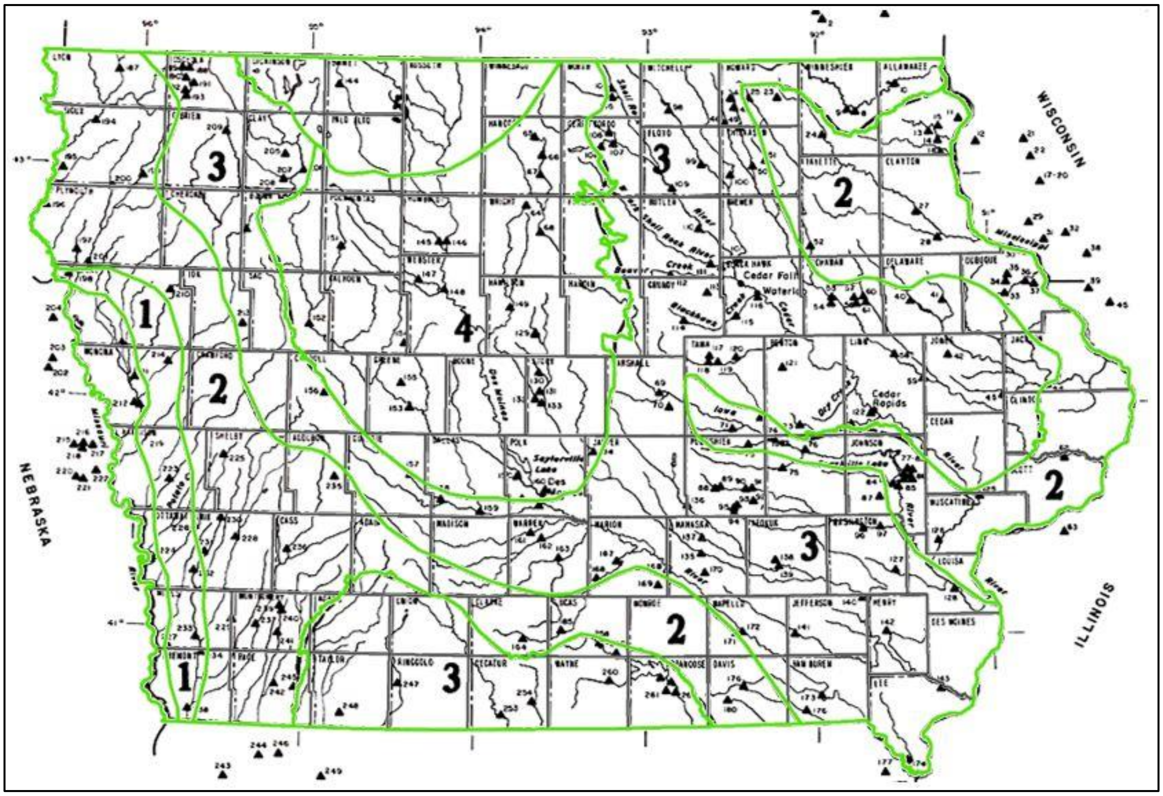

Figure 2.9 Digitized regions for the 1987 model

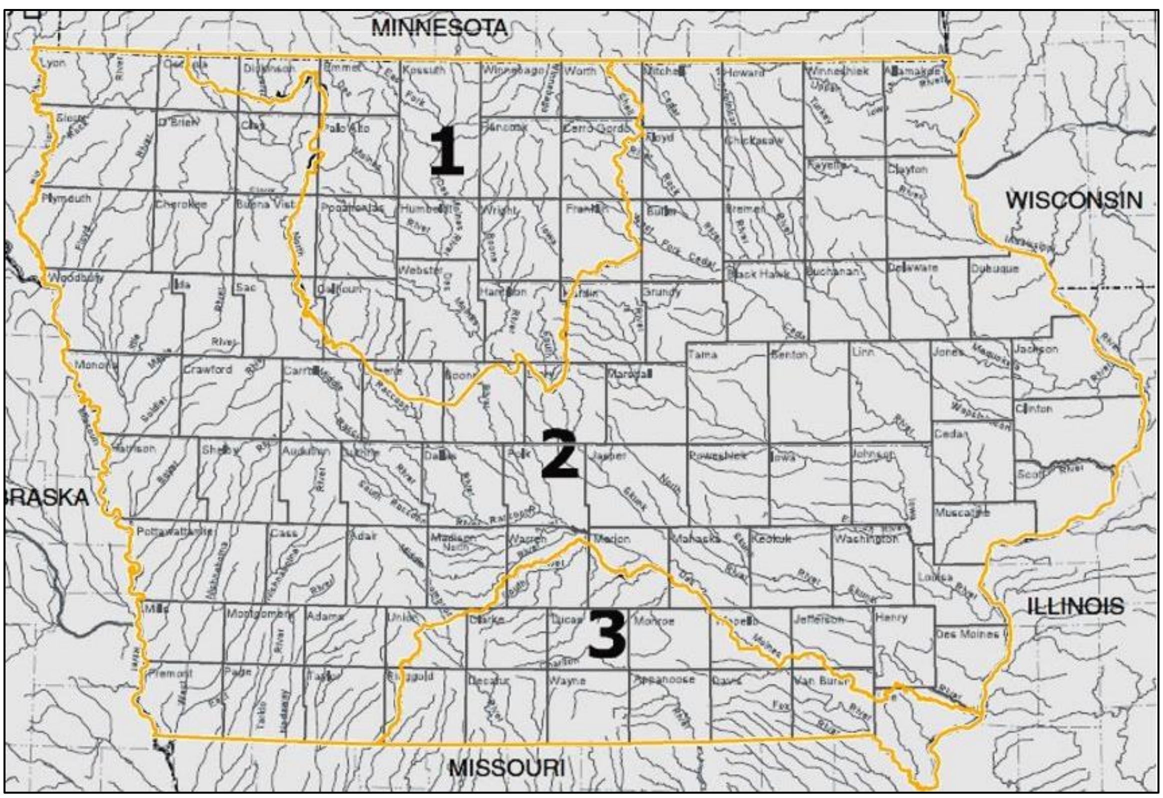

Figure 2.10 Digitized regions for the 2001 model

The ArcMap software was used for determining percentage of the bridge drainage area in each hydraulic region by utilizing the "Union" and summarizing the output data. For the portions that fell outside of the Iowa border, they were assumed to be part of the hydraulic region with the highest share.

Obtained data from StreamStats along with the share of basin in each hydraulic region were used as input for equations from three available flood frequency models for estimating Q_{100}. Bridge built years were compared with publication years of the flood prediction models and the most recent estimate at the time of bridge construction was compared with current one (2013 version).

For better explanation of the procedure, calculation of the Q_{100} for a bridge in Des Moines County of Iowa is shown. The bridge was built in 1970 and therefore, the 1973 model was used for estimating the design flood at the time of construction. Figure 2.11 shows the boundary limits of the basin as well as the hydraulic regions for the 1973 flood prediction model. The drainage area of the basin is 4341.87 mi^2 and basin slope is 1.86 ft/mi. By using ArcMap, it was found that 27.1 percent of the basin falls into region 2 and the rest 72.9 percent is covered by region 1. Therefore, by using equations from Table 2.1 the 100-year flood discharge for each region and the area-weighted discharge would be:

$$Q_{100} \text{ in Region 1 } = 571\,(4341.87)^{0.524}(1.86)^{0.305} = 55{,}587.9\ cfs$$

$$Q_{100} \text{ in Region 2 } = 212\,(4341.87)^{0.642} = 45{,}890.7\ cfs$$

$$Weighted\ Q_{100} = 0.729\,(55{,}587.9) + 0.271\,(45{,}890.7) = 52{,}960.0\ cfs$$

The 100-year flood discharge at the bridge construction time is estimated to be 52,960 ft^3/s and also StreamStats estimation for the current discharge is 82,500 ft^3/s. Therefore, it can be seen that there is 55.8 percent increase in the design flood discharge compared to the discharge at the time of bridge construction.

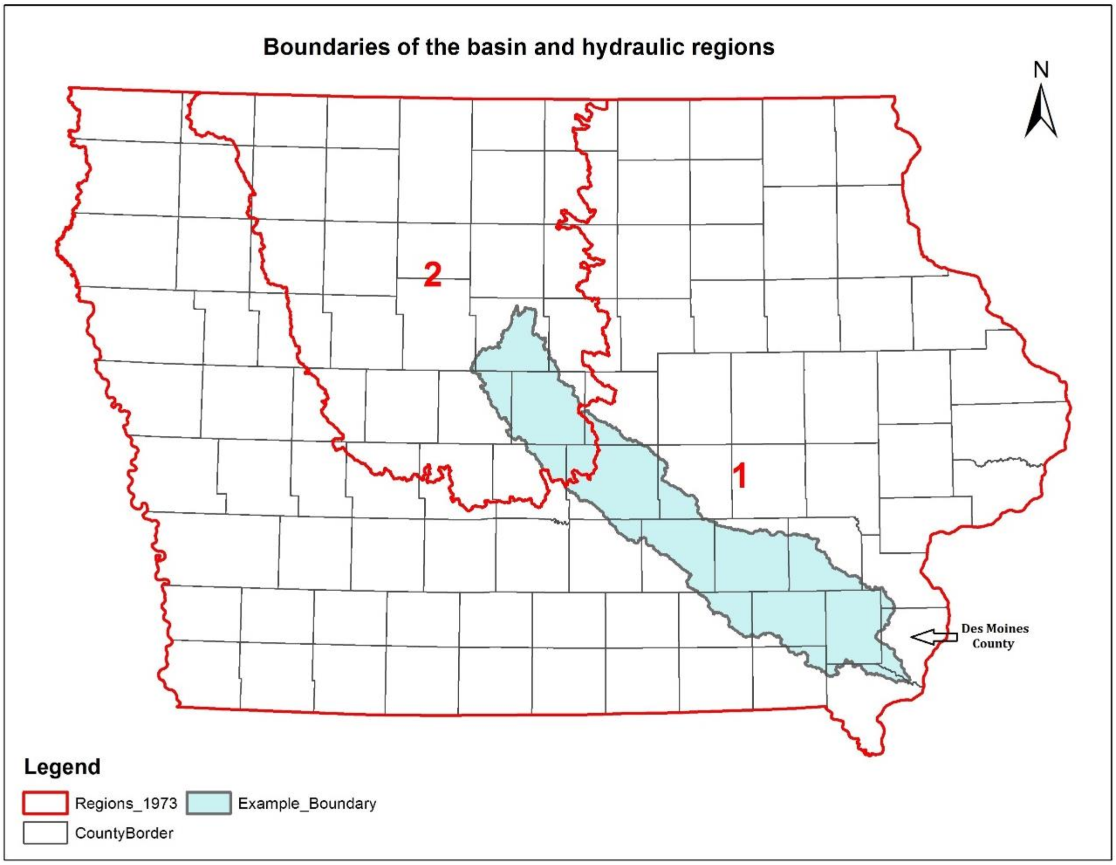

Figure 2.11 Limits of hydraulic regions for 1973 model and the example basin

Similarly, the percentage change of the design discharge was calculated for other bridges. It was found that the results were very sensitive to the size of drainage area especially for the basins smaller than 2 mi^2 and therefore, they were excluded from the assessment. This is in line with Iowa DOT recommendations toward calculating design discharge where sites with drainage areas smaller than 2 mi^2 should be assessed with another approach. The results are shown in Table 2.8.

Table 2.8 Average change of design discharge by prediction model version

Prediction model year	Average change in 100-year flood discharge	Count of bridges
1973	119.7%	899
1987	28.3%	107
2001	11.1%	552
Total	70.7%	1558

As expected, it can be seen that the older discharge predictions experienced more increase, and the results are showing less variability when newer equations were used. As it was mentioned earlier, the change in the flood discharges can be due to three factors: first, the error associated with discharge estimation that can be as high as 40 percent in some cases; second, the effects of the climate change and increased rainfall frequency and intensity; third, urbanization and human activities such as artificial drainage and changed land-use due to agriculture. However, the contribution of each factor to the discharge increase is not clear and more effort is needed in the future to capture that.

It should be noted that regardless of the models' accuracy and other factors that are responsible for the increase in estimated discharge, the estimates are the main sources of Iowa DOT for design purposes at ungaged sites. For bridges on gaged streams, the historical peak discharges can be analyzed for estimating the design flood discharge. However, such data is not available for bridges on ungaged sites and using results of the flood frequency models is the only available solution even if the results are over- or underestimated.

For assessing the sensitivity of the results to the size of drainage area, bridges were categorized based on the drainage area quartiles and for each category, changes of the design discharges were evaluated. Table 2.9 shows the statistics summary of the discharges and as it can be seen, the standard deviations of the estimated discharges are decreasing as the drainage areas become larger. Also, the difference between the first and third quartiles was found to be smaller for larger basins. Therefore, there is less variation between previous models' results for bridges with larger basins.

Table 2.9 Summary statistics for change of design discharge by drainage area quartiles

Change in 100-year flood summary statistics	Drainage area (mi^2)			
	< 12.7	12.1 – 36.4	36.7 – 158.4	> 158.4
Standard deviation	487.9%	169.3%	99.7%	54.6%
Mean	136.7%	83.6%	51.1%	28.2%
First quartile	-8.7%	-1.9%	1.5%	2.7%
Median	-1.8%	11.4%	17.8%	19.3%
Third quartile	82.1%	55.2%	48.3%	39.3%
Count of bridges	389	390	390	389

Figure 2.12 shows the box plots of percentage change in the 100-yaer flood discharges for the four categories of drainage areas that were mentioned before. It can be noticed that compared to other groups, the estimated changes in the first group are more dispersed and more outliers were identified. The results also showed that range of the results is smaller for larger basins and the estimated changes seem to be more reliable.

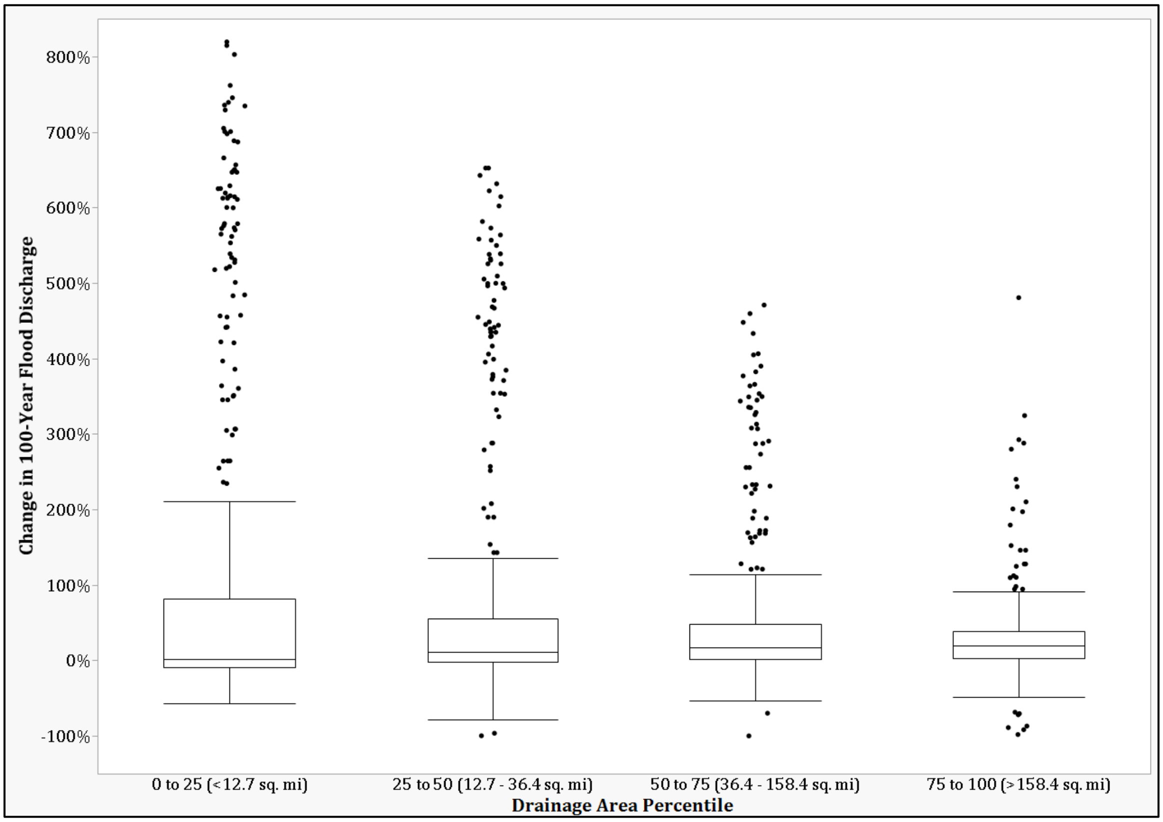

Figure 2.12 Box plots of the changes in discharges by drainage area percentiles

In order to further investigate the estimated changes in the first group, the average change of discharges for the basins smaller than 10 mi^2 by the prediction model are assessed (Table 2.10). It can be seen that the changes are more pronounced in the bridges that were built before 1987 and the design discharges on average have increased by about 136 percent for bridges with basins smaller than 10 mi^2. As a result, older bridges with smaller basins are subject to more change in the discharge and if available, other sources should be utilized for determining the design discharge.

It is very likely that scour was not considered for designing bridges that were built before 1980s. Therefore, bridges that were built after 1987 or a scour protection was installed on them were investigated and it was found that total of 77 of them experienced a change higher than 50 percent in their design discharge.

Table 2.10 Average change of design discharge for basing smaller than 10 mi2 by prediction model version

Prediction model year	Average change in 100-year flood discharge	Count of bridges
1973	195.8%	263
1987	61.1%	22
2001	3.4%	104
Total	136.7%	389

Conclusions

Determining flood discharge is one of the most critical steps toward bridge scour assessment. Scour is known to be the most-frequent cause of bridge failures in the U.S. and therefore, the accuracy of the estimated discharges is very critical for accurate scour risk assessments. Results of this study showed that bridges with small basins that were built before 1980s are more prone to the change in design discharge. The results are in line with Iowa DOT policies where design discharge for bridges with basins smaller than 2 mi^2 is recommended to be determined from Iowa Runoff Chart that was adapted from a study done by W.D. Potter (*17*).

Results from this study will help Iowa DOT to prioritize its bridges for scour management by identifying the group of bridges that are subject to an increase in the estimated design discharge. Among 77 bridges that were identified to have a high change in their design discharge, 67 bridges are not scour-critical, with NBI Item 113 coded as a 5 or 8. Therefore, if those structures were designed today, their design discharges would be higher and their scour assessments could be different. Ideally, their design discharges should be reevaluated by using the most current regional flood frequency analysis (SI Report 2013–5086) or if available, historical peak discharges at the nearest gages should be used.

This study can also be helpful for preliminary scour assessment of already existing bridges in order to reduce the cost and time of the evaluation. Iowa DOT has a three-level scour assessment where the first level, level A, is intended to differentiate between the bridges that are either safe, scour critical, or requiring further review. Level A Scour Assessment is a point-based approach that assigns points to the candidate bridges based on their different scour related characteristics such as foundation type and history of scour at the bridge. Based on the results from this study, size of the drainage area as well as being on an ungaged stream can be considered in Level A assessment where a higher point should be assigned to bridges that have smaller basins or the ones that are on ungaged streams. Although Iowa DOT will not use Level A scour assessment anymore, other smaller agencies can still benefit from the proposed modifications.

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CHAPTER 3. APPLICATION OF THE MODIFIED HYRISK METHODOLOGY IN BRIDGE MANAGEMENT IN IOWA

A paper to be submitted to the *Transportation Research Record*

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Abstract

Floods and resulting scour are responsible for about half of bridge failures in the United States. Catastrophic consequences of bridge failures along with guidelines from the Federal Highway Administration (FHWA) motivated the development of scour assessment tools. HYRISK is one of the available tools for network-level scour analysis and is developed by the FHWA for prioritizing bridges based on their expected scour risk. According to scour management history and experiences in Iowa, this study proposed three major modifications to improve and customize HYRISK estimations for Iowa. Soil erodibility was incorporated into the HYRISK along with a modified failure cost calculation accounting for scour countermeasure installation cost rather than bridge reconstruction that was originally being considered. The modified HYRISK was used to estimate the annual cost of scour risk for Iowa DOT bridge network and also the damage to the affected bridges by the 2008 flood in Upper Mississippi River basin. The results were significantly different from original HYRISK estimations and were in line with the actual annual expenditure on scour maintenance program and also the reported damage from the 2008 flood. Also in order to compare the results from the original and modified versions of HYRISK, a random sample of 30 bridges were selected and ranked based on the estimated risks by the two methodologies. The results showed significant changes in the rankings and it was also concluded that Iowa DOT would need to install six abutment protections and five pier protections in the next year.

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Introduction

Erosive action of flowing water can remove sediments around the bridge abutments and piers which leads to forming of a hole, called scour hole. If scour holes are not considered in the design of footings and piles of the bridge, in some cases they can undermine the footings and reduce the integrity of the bridge, and eventually cause structural failure. Based on a review of more than 500 bridge failures between 1989 and 2000, scour and flooding account for about 50 percent of all the failures in United States (*1*). Also, the effect of scour is more pronounced during flood events when speed and depth of the flowing water are maximum. The 1993 flood in Upper Mississippi basin caused 23 bridge failures and $15 million in damage. Also, total damage to Georgia Department of Transportation (GADOT) from storm Alberto in 1994 was estimated to be $130 million (*2*).

Until 1988, bridges were not necessarily designed to withstand scouring effect of floods. After failure of Schoharie Bridge in New York, Federal Highway Administration (FHWA) published the Technical Advisory (TA) T5140.20, establishing national scour evaluation program that provides guidelines and recommendations for assessment of the scour risk at bridges. In 1991, the Technical Advisory T5140.23 (*3*), "Evaluating Scour at Bridges", superseded the previous TA. For implementing the recommendations of T5140.23, in 1991 FHWA published the Hydraulic Engineering Circular No. 18 (HEC-18) containing the required models and equations for estimating scour depth that can be used for designing new bridges. HEC-18 also provides guidelines for assessing already existing bridges for scour vulnerability. FHWA published four editions of HEC-18 since 1991 with help of the advances made in estimating scour at bridges. Among the major changes of different versions of HEC-18, increased accuracy of the equations and more conservative design floods are the primary ones. As an example, in the earlier versions of HEC-18, depending on the size and importance of the bridge, a flood event as large as a 100-year flood was considered as design flood, however, in the fifth edition, the design flood can be as large as a 200-year flood.

The Moving Ahead for Progress in the 21st Century Act, MAP 21, mandated state DOTs to develop and utilize a risk-based decision making framework in their Transportation Asset Management Programs (TAMPs). Consequently, MAP-21 has been another motivation for developing new scour analysis tools to help DOTs and decision makers with better assessment of the scour risk for existing bridges.

Bridge failures have catastrophic consequences, therefore, identifying the bridges that are more vulnerable is crucial for transportation agencies. In general, scour vulnerability assessment for individual structures has higher accuracy and it is less costly compared to network-level assessment. Also, there are many project-level tools and methodologies available to help managers have a better understanding of the current condition of their bridges and make more informed decisions. Therefore, there is high need for an accurate comprehensive tool that can be applied to the network of the bridges without requiring expensive data collections.

Literature Review

HYRISK is one of the available tools for network-level scour analysis for prioritizing bridges based on their expected scour risk. Scour risk is a function of probability of scour failure and its associated cost (*4*). For estimating the Probability of Failure (POF), HYRISK uses 6 Items from the National Bridge Inventory (NBI):

- Item 26: Functional Classification of Inventory Route
- Item 43: Structure Type
- Item 60: Substructure Condition Rating
- Item 61: Channel and Channel Protection Condition Rating
- Item 71: Waterway Adequacy
- Item 113: Scour Critical Bridges

HYRISK methodology for scour risk assessment

In 1994, FHWA developed a methodology for estimating relative scour risk of bridges by using the NBI database (*5*). In 1999, FHWA used the methodology to develop HYRISK. HYRISK is intended to be used to prioritize bridges in a network based on scour risk. The results help decision makers to allocate the available budget in a more efficient way. HYRISK procedure for estimating the scour risk consists of two components: probability of occurrence and cost associated with the failure. The details of the two components are summarized in the following sections obtained from HYRISK software manual (*4*).

Probability of failure

The first step toward estimating the probability of failure is determining the overtopping frequency. Overtopping occurs when the stream opening at bridge location is full of water and water elevation reaches the bridge superstructure. The importance of overtopping is that resulted scour has a direct relationship with depth and speed of water and it is maximum when overtopping occurs.

The definitions of frequencies are shown in Table 3.1 and are obtained from the description of NBI Item 71. By definition, each frequency has a range of return period, however, HYRISK considers return periods of 100, 50, 5, and 2 years for Remote (R), Slight (S), Occasional (O), and Frequent (F) frequencies respectively.

Table 3.1 Overtopping frequency ranges

Overtopping frequency	Return period	Annual probability
N (None)	Never	Never
R (Remote)	> 100	0.01
S (Slight)	11 to 100	0.02
O (Occasional)	3 to 10	0.2
F (Frequent)	< 3	0.5

Overtopping frequency can be extracted from the NBI database by using NBI Item 71, Waterway Adequacy, and Item 26, Functional Classification. As shown in Table 3.2, the higher the functional classification of the road is, the less frequent the overtopping would be which means bridges in higher functional classes are generally larger and designed to accommodate more severe flood events compared to the ones in lower functional classes.

Table 3.2 Overtopping frequency by NBI Items 26 and 71

NBI Item 26 (functional classification)	NBI Item 71 (waterway adequacy)								
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**N**
1, 11 - Principals and interstates	O	O	O	O	S	S	S	R	0
12 - Freeways or expressways	F	O	O	O	S	S	S	R	0
2, 14 - Other principal arterials	F	O	O	O	S	S	S	R	0
6, 16 - Major arterials	F	O	O	O	S	S	S	R	0
7, 17 - Major collectors	F	O	O	O	S	S	S	R	0
8 - Minor collectors	F	F	O	O	O	S	S	R	0
9, 19 - Locals	F	F	O	O	O	S	S	R	0

Once the frequency of overtopping flood event is determined, it is possible to estimate its discharge and also other discharges associated with lower water levels. For this purpose, HYRISK utilizes regression equations for estimating flood discharges developed by the FHWA (*6*) that are applicable to any small rural basin in the United States.

HYRISK assumes that the cross section of streams is a triangle and therefore, the hydraulic radius of the stream would be the same as flow depth. Therefore, the following equation, which is based on Manning's equation, can be used for estimating water discharge when water surface elevation is lower than the stream full depth.

$$\frac{Q}{Q_f} = \left(\frac{D}{D_f}\right)^{1.66} \text{ or } \frac{D}{D_f} = \left(\frac{Q}{Q_f}\right)^{0.6} \tag{1}$$

Where, Q is flow discharge, D is the depth of the water, and f represents the condition where the stream is full of water.

Using Equation 1 and the overtopping frequency, the stream discharge when water level is lower than full waterway depth can be calculated. As an example, if overtopping discharge is assumed to be 5000 ft^3/sec, the discharge when stream is at half of its full depth would be:

$$\frac{D}{D_f} = \left(\frac{Q}{Q_f}\right)^{0.6} \rightarrow \frac{0.5D_f}{D_f} = \left(\frac{Q}{5000}\right)^{0.6} \rightarrow Q = 1{,}582\ \frac{ft^3}{sec}$$

Similarly, discharges for water level at 25 and 75 percent of the full depth is calculated. The next step is determining the associated annual probability of the resulted discharges by using flood estimation models. Once the annual probabilities are determined, the probability of water level being in different depth ratios can be calculated. For example, if it was assumed that a bridge has an overtopping frequency of 2 percent (Slight overtopping) and the annual probabilities of water level being at 25, 50, and 75 percent of the full depth are 78, 45, and 10 percent respectively, the probability of water level being in different depth ranges can be calculated as follows:

P(Overtopping) = 0.02

P(0.75 to 1.0) = P(0.75) - P(Overtopping) = 0.10 – 0.02 = 0.08 = 8%

P(0.50 to 0.75) = P(0.50) - P(0.75) = 0.45 – 0.10 = 0.35 = 35%

P(0.25 to 0.50) = P(0.25) - P(0.50) = 0.78 – 0.45 = 0.33 = 33%

P(0 to 0.25) = 1 - P(0.25) = 1 – 0.78 = 0.22 = 22%

Hence, the water level in that specific stream would be lower than $0.25D_f$ with 22 percent probability, between $0.25D_f$ and $0.5D_f$ with 33 percent probability, between $0.5D_f$ and $0.75D_f$ with 35 percent probability, between $0.75D_f$ and D_f with 8 percent probability, and higher than D_f with 2 percent probability.

Ideally, each bridge has its own flood discharges and unique depth distribution. However, for easier application, HYRISK considers the average of probabilities for bridges with the same overtopping frequency. Table 3.3 shows the depth distributions by overtopping frequency. As it can be seen, the more frequent the overtopping is, the higher the expected water level would be.

Table 3.3 Water depth distribution by overtopping frequency

Overtopping frequency	**Depth ratio**				
	0 – 0.25	**0.25 – 0.50**	**0.50 – 0.75**	**0.75 – 1.0**	**>1.0**
Remote	0.12	0.48	0.31	0.08	0.01
Slight	0.12	0.34	0.43	0.09	0.02
Occasional	0.07	0.13	0.25	0.35	0.20
Frequent	0.04	0.08	0.15	0.23	0.50

When the water depth distribution is determined, the final probability of failure of a bridge can be calculated by developing a subjective failure probability for each water depth category. HYRISK uses a developed set of scour failure probabilities based on water level ratio and bridge scour criticality (NBI Item 113) as shown in Table 3.4.

Table 3.4 Bridge scour failure distribution by water depth

NBI 113	Depth ratio				
	0 - 0.25	**0.25 - 0.5**	**0.5 - 0.75**	**0.75 - 1**	**> 1**
0	1	1	1	1	1
1	1	1	1	1	1
2	0.25	0.4	0.55	0.7	0.88
3	0.14	0.2	0.3	0.45	0.65
4	0.06	0.1	0.15	0.26	0.41
5	0.002	0.002	0.002	0.03	0.1
6	0.1	0.15	0.225	0.355	0.53
7	0.1	0.15	0.225	0.355	0.53
8	0.002	0.002	0.002	0.01	0.05
9	0.002	0.002	0.002	0.002	0.01
N	0.002	0.002	0.002	0.002	0.002

The last step is determining the final probability of failure (POF) of a bridge due to scour. POF is the product of failure probability for each depth category (Table 3.4) and the associated probability of water distribution (Table 3.3). Here is an example for calculating POF for a bridge with NBI Item 113 of 4 and Slight overtopping frequency.

$$POF = (0.06\times0.12) + (0.1\times0.34) + (0.15\times0.43) + (0.26\times0.09) + (0.41\times0.02) = 0.1373$$

Similarly, POF can be calculated for all ranges of NBI Item 113 and overtopping frequencies. The results are provided in Table 3.5.

Table 3.5 Probability of failure by overtopping frequency and NBI Item 113

NBI 113	Overtopping frequency			
	Remote	**Slight**	**Occasional**	**Frequent**
0	1	1	1	1
1	1	1	1	1
2	0.4573	0.4831	0.628	0.7255
3	0.2483	0.2673	0.3983	0.49510
4	0.1266	0.1373	0.2277	0.2977
5	0.00522	0.00648	0.0314	0.05744
6	0.18745	0.2023	0.313	0.3964
7	0.18745	0.2023	0.313	0.3964
8	0.00312	0.00368	0.0144	0.02784
9	0.00208	0.00216	0.0036	0.006
N	0.002	0.002	0.002	0.002

<u>Risk adjustment factors</u>

Based on the available information, for some bridges it might be reasonable to reduce the estimated POF. There are two risk adjustment factors, used in HYRISK, K_1 and K_2 and the product of them would be the final adjustment factor. K_1 is based on bridge type and structural continuity which is obtained from NBI Item 43.

Second risk adjustment factor (K_2) accounts for foundation design and type. K_2 should be developed separately for both piers and abutments, and the larger value should be used. The recommended values range from 0.2, for bridges built on rock, to 1.0 for unknown foundations. It should be noted that the required information for developing K_2 factor is not stored in NBI and, if available, other sources should be used.

<u>Scour risk cost</u>

The expected cost of scour for bridges, as represented in Equation 3, is the product of probability of failure (POF), adjustment factor (K), and failure cost.

$$Risk = POF \times K \times [Rebuild\ Cost + Running\ Cost +\ Time\ Cost] \qquad (2)$$

Where *Rebuild Cost* is the required money for reconstruction of the failed bridge, *Running Cost* and *Time Cost* are the costs associated with the vehicle operation and value of the time of the bridge users.

Risk cost estimated by HYRISK is in annual basis, however it should be noted that the numbers are not representing real money and they should only be used for comparing relative risks of bridges.

Soil erodibility

Different soils scour at different rates and scour holes form rapidly in loose soils while cohesive soils are more resistant. Therefore, given the same final scour depth for different types of bed materials, the time needed for reaching to that final depth is maximum for the soil with the highest shear resistance, which means more flood events are needed to occur for forming final scour depth. Therefore, in the scour analysis process, there should be a differentiation between bridges located on more resistant soil layers compared to the ones constructed on looser and more granular materials.

HYRISK does not consider the characteristics of the soil that a bridge is located on as a contributing factor. To address that, Georgia DOT with the cooperation of the Georgia Institute of Technology, extended the original HYRISK methodology by incorporating soil properties into it. Similar to previous studies (*7*) Bones et al. (*8*) used the collected data from 68 soil samples at bridge locations to develop five categories for soil erodibility ranging from "Very Erodible" to "Very Resistant". Also, a downward adjustment factor ranging from 0.2 to 1 was developed according to previously defined categories in order to modify the estimated POF by HYRISK.

Determining soil shear strength and erodibility for abutments and piers of all bridges in the network is extremely expensive and time consuming. Therefore, Bones et al. associated the erodibility categories with soil classifications based on Unified Soil Classification System (USCS). Soil classification is usually provided and in bridge documents and boring logs and is easily accessible. As a result, by using the soil classification at bridge locations, the estimated POF can be adjusted to be more realistic for bridges with more resistant soils.

Methodology

The main goal of this study is improving HYRISK scour risk predictions by addressing some shortcomings of this software and also applying required modifications based on Iowa DOT experiences and policies regarding bridge scour management. Figure 3.1 shows the general procedure of the original HYRISK for estimating scour risk as it was elaborated earlier. Green boxes represent the contributions of this study and the modifications that were applied into the original HYRISK, and the red box (user cost) is recommended to be completely excluded.

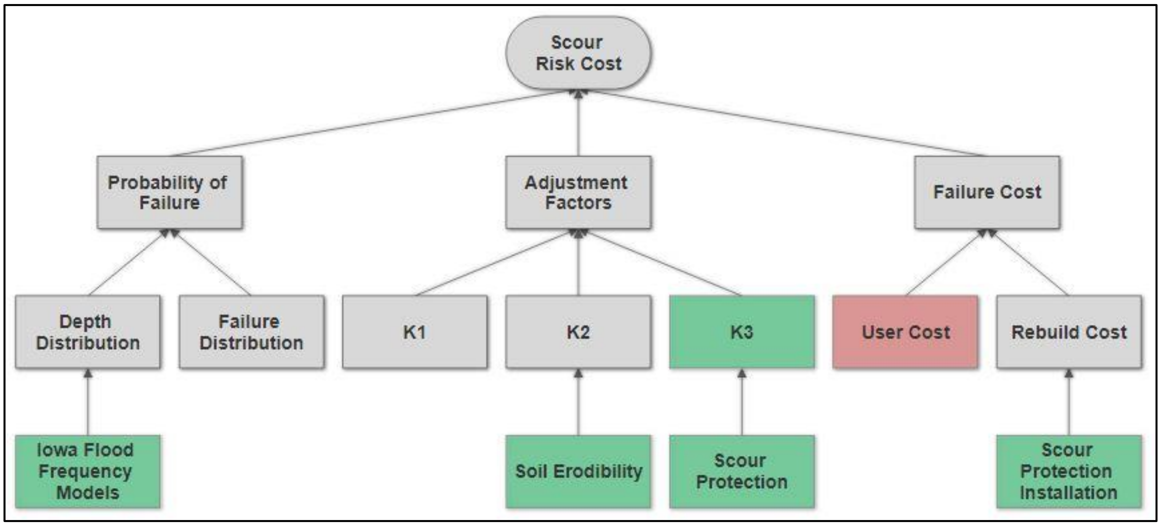

Figure 3.1 HYRISK procedure

The shortcomings of HYRISK are discussed in the following section and are followed by proposed modifications and data collection procedure.

HYRISK limitations

HYRISK software, in spite of being comprehensive, has its own limitations that limited its use by state agencies. Three limitations are addressed in this study and the rest are presented in Discussion section.

The first and most important limitation of HYRISK is overestimation of the bridge failures. In 2005, all 356,378 bridges in the US were analyzed by HYRISK and it was estimated to have 60,511 bridge failures each year, or in other words, approximately 1 out of every 6 bridges. However, based on an interview done with 25 states (*9*), the actual number of bridge failures due to scour is about 1 in 5,000 bridges. Therefore, the estimated POF is not realistic and should be calibrated or modified.

The second shortcoming is not incorporating the soil erodibility that bridges are built on. Shear strength of the soil plays an important role for assessment of the bridge scour vulnerability. The expected depth of the scour hole is less in soils with higher shear strength, such as clays, compared to weaker soils (*2*). However, HYRISK reduces the POF only for foundations that are built on rock and no reduction is considered for other soil types. Therefore, a risk adjustment factor based on erodibility of the soil would improve the predictions.

Failure cost overestimation was identified as the last limitation of HYRISK. Although half of the total bridge failures in US are due to floods and scour, the damage from scour does not necessarily cause bridge failure. Especially in Iowa, there were very limited number of state-owned bridge failures due to scour in the past recent years. Therefore, HYRISK overestimates the scour consequences and in this study, scour protection installation cost for piers and abutments was considered as scour damage outcome rather than bridge failure and reconstruction.

Proposed modifications for HYRISK

As it was mentioned earlier, in general, HYRISK overestimates both the probability of failure and failure consequences. Based on the identified limitations and available resources from Iowa DOT, some modifications were proposed in order to increase accuracy and applicability of the original HYRISK methodology.

<u>Modified estimation for failure cost</u>

Failure cost calculated by HYRISK significantly overestimates the actual cost since there were few state-owned bridge failures in the state of Iowa. When a bridge experiences scour, installing the suitable scour countermeasure typically is sufficient for reducing the vulnerability. Therefore, the cost of bridge reconstruction and its associated user cost would be much higher than installing a countermeasure.

For purpose of this study and by considering the history of scour related actions in Iowa, only the cost of implementing scour countermeasures was considered as scour damage consequences. The scour protection cost depends on bridge type and type of scour damage. Iowa DOT estimates the cost of installing pier protection to be $50,000 and abutment protection to be $70,000 and $150,000 for single-span and multiple-span bridges respectively.

Calibrating depth distribution using Iowa-specific flood prediction equations

The original depth distributions of HYRISK were developed using 1977 flood estimation equations that were applied to all bridges in the US. Custom equations, enabled by advanced technology and improved flood estimation tools, with enhanced accuracy can today be developed and used. Therefore, the depth distribution was calibrated by using the latest flood estimation equations from USGS SI Report 2013-5086 (*10*).

For flood estimation, an online tool named StreamStats (*11*) was used which is developed by U.S. Geological Survey (USGS). StreamStats calculates basin characteristics and flood discharges for any user-specified point along the streams. The equations used for flood estimation are obtained from Scientific Investigations Report 2013–5086 (*10*) which are also used by Iowa DOT for bridge design. Eight flood events with return period of 2, 5, 10, 25, 50, 100, 200, and 500 years were calculated by StreamStats for each bridge.

For calibrating the depth distribution, the overtopping frequency from Table 3.2 was used to estimate the return period of other flood events associated with lower water levels. Flood events available from StreamStats were used to estimate the probability of water depth being less than 25, 50, and 75 percent of the full depth of the streams by using Equation 1. For this goal, linear interpolation was done between two most relevant available flood discharges.

The depth distribution is unique for each bridge since bridges have different basin characteristics, and therefore, different estimated flood discharges. However, for summarizing the results, average of all depth distributions is provided in Table 3.6.

Table 3.6 Calibrated depth distribution by overtopping frequency

Overtopping frequency	**Average probability of depth**				
	0 - 0.25	**0.25 - 0.5**	**0.5 - 0.75**	**0.75 - 1**	**> 1**
Remote	0.35	0.39	0.21	0.05	0.01
Slight	0.34	0.37	0.21	0.06	0.02
Occasional	0.18	0.23	0.24	0.15	0.20
Frequent	0.09	0.15	0.14	0.12	0.50

By comparing Table 3.6 with Table 3.4 it can be noticed that the expected value of the original depth distributions is higher than the calibrated ones. Originally, categories of “0.25 - 0.5” and “0.5 – 0.75” had the highest likelihood. However, after the calibration the peak shifted to “0- 0.25” and “0.25 - 0.5” categories.

It can be seen from Table 3.4 that higher depths are associated with higher probabilities of failure. Therefore, the calibration would result in smaller estimated POF for bridges and the reduction is less pronounced for bridges with higher values of NBI Item 113. Table 3.7 shows the reduction in POF by NBI Item 113 and overtopping frequency after calibrating the depth distribution.

Table 3.7 Difference in the probability of failure after calibrating the depth distribution

NBI 113	Overtopping frequency			
	Remote	**Slight**	**Occasional**	**Frequent**
0	0	0	0	0
1	0	0	0	0
2	0.0434	0.0689	0.0728	0.0426
3	0.0219	0.0388	0.0534	0.0320
4	0.0132	0.0222	0.0342	0.0205
5	0.0003	0.0006	0.0051	0.0032
6	0.0176	0.0305	0.0438	0.0262
7	0.0176	0.0305	0.0438	0.0262
8	0.0001	0.0002	0.0015	0.0009
9	0	0	0	0
N	0	0	0	0

New risk adjustment factors

In HYRISK, two risk adjustment factors were originally considered for structural continuity and foundation type. A modified version of the foundation type adjustment factor was previously developed for Georgia DOT (*12*) in order to account for soil erodibility. In this study, the same adjustment factor was used by incorporating soil information. Bridge design documents from Iowa DOT databases were reviewed for collecting soil data as well as any other scour related information such as presence of scour protections and their type.

A third risk adjustment factor was also introduced in this study to account for presence of scour countermeasures. Since 1990s, many state DOTs began to evaluate and retrofit their bridges against scour based on HEC-18 procedures. Therefore, it was not necessary to consider presence of scour protections as a contributing factor for scour risk assessment when HYRISK was originally developed. However, based on the soil review done on State-owned bridges in Iowa, more than 600 bridges were found to have at least one kind of scour protection. Also, Iowa DOT experts stated that presence of scour protections at a bridge would significantly improve its stability. They recommended a 75 percent decrease in scour risk in the presence of scour countermeasures. Therefore, the new risk adjustment factor was considered to be 0.25 for bridges that have scour protections and this factor should be developed separately for piers and abutments. For application of the proposed risk adjustment factor, the following equation was developed.

$$Risk\ Cost = POF \times [(CMC_{Pier} \times P_{Pier} \times K_{Pier}) + (CMC_{Abut} \times P_{Abut} \times K_{Abut})]$$

Where CMC_{Pier} and CMC_{Abut} are costs of countermeasure installation for piers and abutments respectively, P_{Pier} and P_{Abut} are the probabilities of having pier or abutment scour damage respectively, K_{Pier} and K_{Abut} are the risk adjustment factors for presence of pier or abutment protection respectively. The likelihood of having specific type of scour damage is developed based on the previous protections that were installed by Iowa DOT.

Collecting soil properties and developing adjustment factors for Iowa

As it was explained earlier, type of the soil underneath the bridge has a significant effect on scour vulnerability of bridges. By assuming that soil in Iowa holds the same properties as Georgia (i.e. critical shear stress and median grain size), the same adjustment factors based on soil classification were used to reduce the risk. Unfortunately, neither NBI nor any other databases have soil characteristics available for every bridge. Therefore, manually reviewing the bridge documents was the only way of collecting soil data.

In general, DOTs maintain databases for storing the original design and as built documents which are the best available sources for collecting soil data. Databases from Iowa DOT that were used in this study are Structure Inventory and Inspection

Management System (SIIMS) and Electronic Record Management System (ERMS). SIIMS is used for storing and reviewing bridge information and contains details of the latest inspection, documents of the last reconstruction and original construction, or details of any major maintenance action done on bridges. ERMS is a place for migrating and keeping any documents for different projects of Iowa DOT including bridge and roadway maintenance and construction.

For filtering out the document of interest, various bridge identifiers can be used in SIIMS and ERMS. However, one of the easiest ways would be using the bridge FHWA Number in SIIMS database to find the desired design document.

In the last decades, the way that the design documents are arranged and the details they cover have changed and improved. The information about thickness of the different soil layers below the ground and types of the soil and its classification are mostly included in design documents and they are usually put in the "Situation Plan" or other sections related to geotechnical design. However, it might happen that the soil data is missing from the document gathered from SIIMS, and therefore, the project number of that design should be used for searching in ERMS.

While reviewing the documents, the reviewer should be familiar with different scour outcomes and the effect of soil types at different depths on scour. For example, abutments and piles are impacted at different depths based on bridge design. The following is a description of scour related issues for bridges and their contributing factors.

<u>Abutment related erosion</u>

Every bridge has two abutments that have downward slopes called "berms". One of the most common type of scour issues in Iowa is berm erosion where the slope gets washed away due to shear stress of the flow. As a result, the abutment piles and foundations will be undermined and if not treated, it can cause bridge instability and even failure. However, presence of long piles in abutment foundations will substantially increase their stability.

Another concern with bridge abutments is the erosion of approach materials rather than the soil around the abutment foundation. In this case, the water continues washing materials from beneath the approaches and make them vulnerable. The bridge itself might remain stable, however, the approaches will be at risk and if they fail, the bridge cannot be used anymore and should be closed to traffic.

Figure 3.2 is an example of abutment scour where approach materials are washed out and there is high risk of approach failure. However, unlike the example in the picture, the hole below the approaches might not be always visible which makes it more difficult for the inspectors to identify that.

Figure 3.2 Scour at bridge approach (Source: USGS)

Based on the experiences of Iowa DOT staff, most of the scour issues are at abutments or approaches while pier scour is less important because of Iowa DOT's practice of implementing long piles.

Pier related scour

In addition to abutments, multi-span bridges also have piers that are usually more exposed to flowing water. The resulting shear stress of water forms a hole around the pier and the higher the stress, the deeper and wider the hole would be. In order to evaluate the bridge against scour, the expected depth of the scour hole should be assessed and the structural stability should be assessed based on that. However, as a rule of thumb by Iowa DOT, when unbraced length of the pier is more than 20 feet or when exposed length of the pile is more than 50 percent of its total length, the bridge can be vulnerable and should be assessed in more details.

In general, pier scour is more pronounced when piles are short or the bridge does not have piles. Longer piles can withstand deeper scour holes and therefore, mostly there is no need for implementing countermeasures for reducing the scour risk at piers. Fortunately, Iowa DOT has been designing and implementing relatively long piles since early 1940s, and therefore, there are few bridges that do not have piles, except the ones that have shallow foundations located on near-ground bedrock. As a result, pier scour is not as critical as abutment scour for Iowa DOT bridges. However, Iowa DOT might install countermeasures at piers not because of the risk of scour, but due to risk of erosion of the exposed piles and foundations to the water or air.

Bridge overtopping

Water surface elevation depends on the intensity of the flood events. On-waterway bridges are usually high enough to accommodate 100-year or more severe floods. However, when a flood occurs that its corresponding water elevation reaches the bridge deck, the bridge would be overtopped and might be closed to traffic for several days. Therefore, the economic assessment of overtopping requires determining the flood intensity that causes bridge overtopping.

Overtopping frequency can be obtained from the NBI database. However, it was found that it is not accurate enough when it was compared with bridge design documents. Also, the most severe flood considered in HYRISK is 100-year flood.

Therefore, bridge overtopping was not assessed in this study due to lack of required data.

Bridge document review procedure

FHWA Number (NBI Item 8: structure number) was used to query the bridges in SIIMS database and for each bridge, the documents that were related to scour as well as original design and reconstruction plans were downloaded for review purpose. If the scour treatments were already installed at any specific bridge, a document explaining the type, design specifications, and date of implementation of the treatment was available in SIIMS. Also, if the bridge was identified as scour critical bridge, the developed Plan of Action (POA) was available and downloaded to be reviewed.

Documents of bridges that were built after 1990s, contain the estimated scour depth at piers and/or abutments. The estimated scour depth defines the depth that soil should be reviewed for pier related scour. Also for abutment related scour review, limits of the abutment footing or area around the berm should be evaluated in order to find the weaker or more critical soil layers. In the review process, presence of scour countermeasures, type of countermeasures and the weakest identified soil layer were collected.

As an example, Figure 3.3 illustrates a three-span bridge where the bottom of the scour hole is estimated to be at elevation of 977 feet. Therefore, the red areas covering the soil from top of the ground down to the estimated scour hole should be considered for pier scour. Also, for abutments, green areas should be considered since they are the approximate areas that if washed away, can cause serious threats to either the bridge itself or the approaches. It can also be noticed that the bridge does not have any scour countermeasures around its piers or abutments.

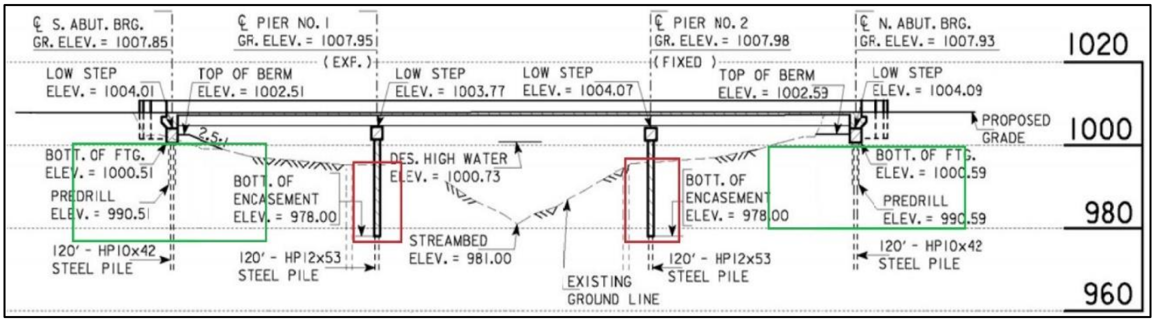

Figure 3.3 Longitudinal section of a bridge with known scour depth

Bridges that were built before 1990, do not have scour depth calculated for them. In those cases, the minimum required depth of the scour hole for bridges to be considered as scour critical was evaluated. The minimum required depth was estimated using the general rules that was explained in the pier related erosion section and it is the maximum depth of either 20 feet of unbraced column length or exposure of at least 50 percent of the piles.

Figure 3.4 is an example of a bridge that does not have estimated scour depth. For abutments, similar to previous example, the green areas near the abutment piers and under the approaches should be considered. Also for piers, the soil near the pile cap down to 50 percent of the piles should be assessed, which in this case would be around elevation of 1118 feet.

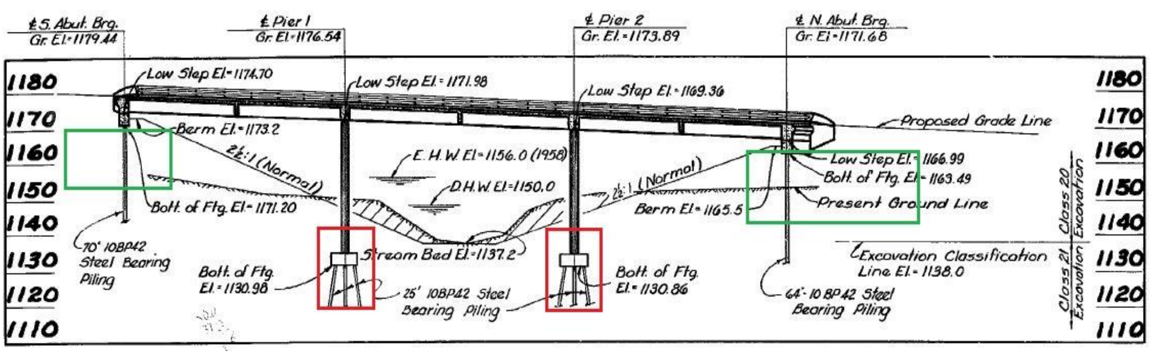

Figure 3.4 Longitudinal section of a bridge with unknown scour depth

Once the area of interest is determined, the soil layers were reviewed and in order to be conservative, the weakest layer was identified and assigned to the bridge. Table 3.8, adapted from GADOT Research Project 14-35 (*12*), was used as a guide for comparing erodibility of different soil types based on Unified Soil Classification System (USCS).

Table 3.8 Soil erodibility based on USCS (source: GADOT Research Project 14-35)

	Soil type	Adjustment factor	Erodibility
Coarse-grained soils (sand and gravel)	SW & SP	1	Very Erodible
	SM & SC	0.8	Erodible
	GW & GP	0.6	Moderately Resistant
	GM & GC	0.4	Resistant
	Rock	0.2	Very Resistant
Fine-grained soils (silt and clay)	CL	1	Erodible
	CL-ML	0.8	Erodible
	ML	0.6	Moderately Resistant
	MH	0.4	Resistant
	CH	0.4	Resistant

As a demonstration, the weakest soil layer corresponding to the piers of the bridge depicted in Figure 3.4 is identified (Figure 3.5). As it was indicated before, for piers, the area that should be evaluated is from the surface of the ground down to elevation of 1118 feet. As it can be seen below, pier 2 has sand and coarse sand in that vicinity which are the weakest soil types (Table 3.8), and the corresponding erodibility adjustment factor is 1.0 which means no reduction in the scour risk.

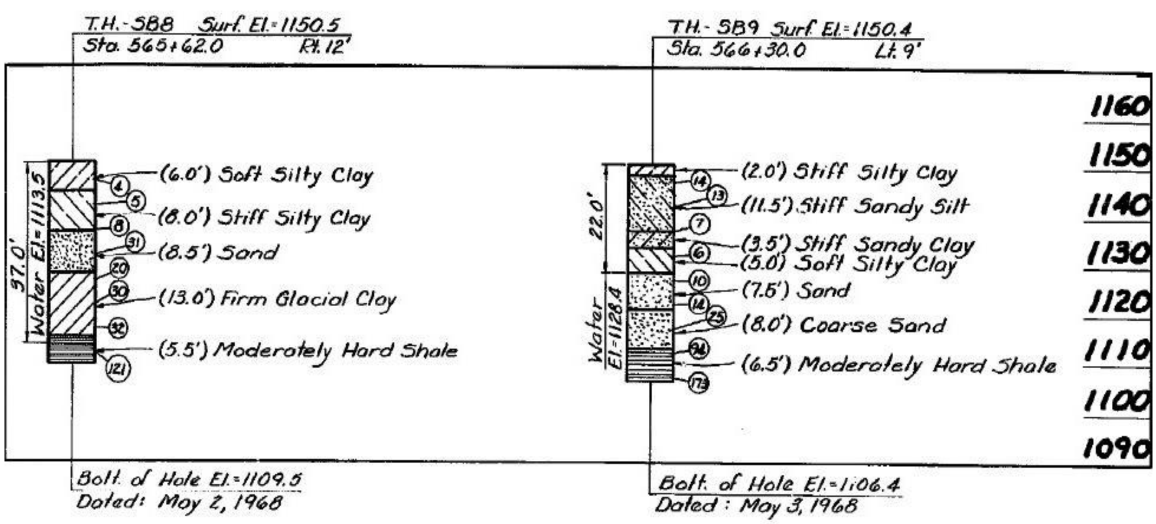

Figure 3.5 Example soil layers

It should be noted that not all the design documents available through SIIMS have soil data. In order to gather soil properties for that group of bridges, the Project

Number of that bridge first needed to be identified and then used for querying in ERMS. Project Numbers can usually be found in the design documents. However, there were bridges that soil data was not available either through SIIMS or ERMS, and the soil type for this group was coded as "Unknown" and an adjustment factor of 1.0 was assigned to them.

Analysis and Results

The on-waterway bridges under Iowa DOT maintenance responsibility were reviewed and based on the collected soil types, an adjustment factor was assigned to each bridge and the results are summarized in Table 3.9. Adjustment factors were based on weaker soil layers, and therefore, majority of bridges are coded to have sand or silty clay soil. On average, the adjustment factor is 0.84 which means the risk estimated by HYRISK is being reduced on average by 16 percent.

Table 3.9 Bridge soil types and adjustment factors

Soil type	Name	Erodibility	Adj. factor	Bridge count	Percent of bridges
SW & SP	Sand	Very erodible	1.0	983	52.0
U	Unknown	Very erodible	1.0	135	7.1
CL	Lean clay	Erodible	1.0	84	4.4
CL-ML	Silty clay	Erodible	0.8	322	17.0
SM & SC	Clayey/silty sand	Erodible	0.8	68	3.6
ML	Silt	Moderately resistant	0.6	24	1.3
GW & GP	Gravel	Moderately resistant	0.6	14	0.7
CH	Fat clay	Resistant	0.4	100	5.3
GM & GC	Clayey/silty gravel	Resistant	0.4	0	0
MH	Elastic silt	Resistant	0.4	0	0
R	Rock	Very Resistant	0.2	159	8.4

As a part of the document review, presence of scour protections installed by Iowa DOT was also collected. Table 3.10 shows the summary of collected data. It can be seen that abutment related scour damage is more frequent (about 95 percent of protected bridges experienced abutment damage). Pier damage, on the other hand, is less pronounced which is due to the fact that Iowa DOT has been implementing long piles for several decades.

Table 3.10 Summary of scour countermeasures installed by Iowa DOT

	Type of protection					
	NBI Item 113 coded as 7			All bridges		
	Pier protection	Abutment protection	Total	Pier protection	Abutment protection	Total
Number of bridges	92	112	119	320	592	626
Percent of bridges	77.3%	94.1%	100%	51.1%	94.6%	100%

Application

Three proposed modifications were implemented in this study: modified cost calculation method; calibration of the flow depth distribution; and also making use of two new risk adjustment factors. The modified HYRISK can be used to estimate the expected cost of scour risk for a bridge network with two types of applications. First application is estimating the annual expected cost of scour risk under normal rainfall and stream discharges. The second application is estimating cost of scour risk for a group of bridges that are affected by a severe flood. Calculation procedures of the two applications are described in the next two sections.

Annual expected cost of scour risk

In order to calculate the annual expected scour risk in Iowa, the collected soil data as well as presence of scour protection were used and all the state-owned bridges were assessed by the modified HYRISK approach. For more elaboration on the calculation process, the expected cost of scour risk for a bridge with the following characteristics is calculated.

Scour Critical Bridges (NBI Item 113): 3 (unstable foundation)

Functional Classification (NBI Item 26): 1 (Interstate)

Waterway Adequacy (NBI Item 71): 8

Structure type (NBI Item 43): Multi-span, lengths <30m

Soil erodibility adjustment factor: 0.6

Scour protection: Only abutment protection

Bridge age (from NBI Item 27): 20 years

The first step is determining the overtopping frequency by using Table 3.2. The bridge has NBI Item 71 (Waterway Adequacy) of 8 and Item 26 (Functional Classification) of 1 and therefore, the overtopping frequency is "Slight". Based on depth distribution from Table 3.6 for overtopping frequency of "Slight", and also corresponding failure distribution from Table 3.4 for NBI Item 113 of 3, the POF is calculated as follows:

$$POF = [Failure\ Distribution] \times [Depth\ Distribution]$$

$$POF = \begin{bmatrix} 0.14 & 0.20 & 0.30 & 0.45 & 0.65 \end{bmatrix} \times \begin{bmatrix} 0.34 \\ 0.37 \\ 0.21 \\ 0.06 \\ 0.02 \end{bmatrix} = 0.2246$$

The next step is applying risk adjustment factors associated with the structural continuity (K_1) and the soil erodibility (K_2). Based on recommended values by HYRISK, K_1 factor for this bridge is 0.8 and also K_2 factor is given as 0.6. Therefore, the adjusted POF would be:

$$POF_{Adj} = POF \times K_1 \times K_2 = 0.2246 \times 0.8 \times 0.6 = 0.1078$$

Finally, expected cost of scour risk is calculated by using Equation 2:

$$Risk\ Cost = 0.1078 \times [(50{,}000 \times 0.511 \times 1) + (150{,}000 \times 0.946 \times 0.25)] = \$6{,}579$$

The first term in the parenthesis is representing the situation that pier protection is needed where cost of protection is $50,000, and there is no reduction in risk since the bridge does not have pier protection, and the probability of having pier damage based on Table 3.10 is 51.1 percent. Similarly, cost of abutment damage is presented in second term and since the bridge has abutment protection, the risk is reduced by 75 percent and a risk adjustment factor of 0.25 is used.

Cost of scour risk was calculated for all bridges by using modified HYRISK and the total annual expected cost was estimated to be $1,091,524 as shown in Table 3.11. Also for comparison, the expected risk cost from the original HYRISK was also calculated for Iowa DOT network and presented in Table 3.11.

It can be noticed that because of the applied modifications, the magnitude of the estimated risks are significantly different. The main reason for the difference is different failure cost calculation methods and exclusion of the user cost in the modified HYRISK. In order to assess the effects of other changes, the user cost component was excluded from the original HYRISK and total annual expected cost of scour risk was calculated. It was found that the majority of the difference in total expected risk costs by the two methodologies was due to the cost associated with users in the original HYRISK. However, there is still a significant gap between the two estimates which is a result of using the new adjustment factors as well as considering countermeasure installation rather than bridge reconstruction.

It should be noted that the risk costs provided in Table 3.11 are not representing real money and they should only be used for comparison or identifying groups of at risk bridges. However, the results from modified HYRISK are closer to Iowa DOT expenditure on scour maintenance which is around one million dollars. Also as Table 3.11 shows, by increasing NBI Item 113, the average expected scour risk cost estimated by the modified HYRISK is decreasing with the exception of Item 113 of 5 which can be due to the fact that the values for failure distribution from Table 3.4 for NBI Item 133 of 5 are lower. However, this pattern cannot be seen for the original HYRISK results since they depend on the detour length for estimating user cost and size of the bridges for reconstruction cost.

For better comparison of the impact of both methodologies in network-level prioritization of scour management, a random sample of 30 bridges was selected and the expected scour risk was calculated for them by using both methods. Due to the significant difference in the magnitude of estimated risks, the bridges were ranked based on their estimated risks and the rankings were compared. As shown in Table 3.12, the bridges that already have scour protections or are built on stronger soils, are generally ranked lower by the modified HYRISK and are located at the bottom of the list. Also by comparing the two rankings, significant changes in rank can be noticed that are a result of considering user cost as a component of failure cost in the original HYRISK as well as not considering the soil erodibility and presence of scour protections. The reason that user cost component was excluded from the total failure cost was that the original methodology estimates the cost of bridge reconstruction which is associated with a significant cost of the bridge users. The modified version however, estimates the countermeasure installation cost and since bridge closure is not necessarily required for it, the user cost was proposed to be removed.

Table 3.11 Average expected scour risk cost by different versions of HYRISK by NBI Item 113

NBI Item 113	Bridge count	Average of expected risk cost	Total expected risk cost
Modified HYRISK			
3 (Scour critical bridge)	13	$5,251	$68,268
5 (Scour within limits of foundation)	959	$540	$518,156
6 (Unassessed bridge)	3	$3,107	$9,320
7 (Scour countermeasure installed)	123	$1,566	$192,581
8 (Stable bridge foundation)	771	$328	$252,907
9 (Foundations on dry land)	20	$98	$1,963
Total	1889	$552	**$1,043,195**
Original HYRISK			
3 (Scour critical bridge)	13	$3,262,011	$42,406,139
5 (Scour within limits of foundation)	959	$902,239	$865,247,006
6 (Unassessed bridge)	3	$4,905,933	$14,717,799
7 (Scour countermeasure installed)	123	$4,742,199	$583,290,509
8 (Stable bridge foundation)	771	$469,254	$361,795,095
9 (Foundations on dry land)	20	$322,855	$6,457,098
Total	1889	$992,014	**$1,873,913,645**
Original HYRISK (user cost excluded)			
3 (Scour critical bridge)	13	$17,553	$228,183
5 (Scour within limits of foundation)	959	$6,784	$6,505,763
6 (Unassessed bridge)	3	$33,112	$99,335
7 (Scour countermeasure installed)	123	$15,131	$1,861,167
8 (Stable bridge foundation)	771	$4,768	$3,675,805
9 (Foundations on dry land)	20	$6,677	$133,547
Total	1889	$6,619	**$12,503,799**

Table 3.12 Comparison of the rankings from the original and modified versions of HYRISK

FHWA#	Erodibility factor	Abutment protection	Pier protection	Modified HYRISK		Original HYRISK	
				Cost	Rank	Cost	Rank
025390	1	0	1	$3,565	1	$2,480,043	3
014480	1	1	0	$1,402	2	$4,823,339	1
604630	1	0	0	$1,096	3	$6,430	28
043840	1	0	0	$787	4	$767,382	8
034791	0.8	0	0	$746	5	$2,040,080	4
025011	1	0	0	$681	6	$546,984	9
039791	1	0	0	$598	7	$262,378	17
606500	1	0	0	$595	8	$2,280	29
018271	1	0	0	$587	9	$2,881,042	2
029101	1	0	0	$576	10	$418,964	12
031240	1	0	0	$569	11	$270,984	16
602320	0.8	0	0	$456	12	$149,213	21
032090	1	0	0	$424	13	$8,823	27
699240	0.4	0	0	$418	14	$2,100	30
031270	1	0	0	$376	15	$180,185	20
604020	0.8	0	0	$257	16	$79,569	26
607795	0.4	0	0	$232	17	$125,066	24
014841	0.8	1	0	$172	18	$453,569	11
609175	0.2	0	1	$153	19	$213,187	18
043231	1	1	1	$124	20	$277,631	15
051141	0.2	0	0	$122	21	$943,327	7
019290	0.2	0	0	$120	22	$190,720	19
052630	0.2	0	0	$119	23	$1,555,049	5
021071	0.2	0	0	$112	24	$1,077,155	6
028070	1	1	1	$108	25	$129,547	23
021310	1	1	1	$99	26	$520,916	10
027081	0.4	1	1	$96	27	$148,518	22
017951	0.8	1	1	$94	28	$352,451	14
019741	1	1	1	$94	29	$359,078	13
050781	0.2	0	0	$74	30	$100,920	25

Expected cost of scour risk due to a flood event

The second application of the modified HYRIK is estimation of the damage or cost from a single flood event of interest. Knowing vulnerability of the bridges against different flood events can help decision makers to have a better understanding of the resiliency of the bridge network. In this application, there are no restrictions on the intensity of the flood event and the expected damage from floods with any return period can be assessed. However, as previously mentioned, floods events larger than a 100-year flood would have the same estimated risk cost since HYRISK assumes that a 100-year flood would overtop all bridges.

The process of risk cost calculation due to a flood event is very similar to the annual risk cost. The only difference is in the values of depth distribution. Depth distributions were previously based on the probability of water level being in different depths under normal conditions. However, when a specific flood event is being considered, the associated water level is known and depth distribution should be adjusted accordingly. Therefore, all the values in Table 3.6 for each overtopping frequency would be zero expect for the depth category that flood water elevation falls into.

For determining the flood water elevation depth category, the Annual Exceedance Probability Discharges (AEPD) of the desired flood should be compared with the values of Table 3.6. AEPD is the probability of occurrence of a flood in each year and it is inverse of the return period.

The updated versions of depth distributions for a 100-year and a 10-year flood, with AEPDs of 1 and 10 percent, are shown in Table 3.13 and Table 3.14 respectively.

Table 3.13 Calibrated depth distribution for a 100-year flood by overtopping frequency

Overtopping frequency	**Probability of depth**				
	0 - 0.25	**0.25 - 0.5**	**0.5 - 0.75**	**0.75 - 1**	**> 1**
Remote	0	0	0	0	1
Slight	0	0	0	0	1
Occasional	0	0	0	0	1
Frequent	0	0	0	0	1

Table 3.14 Calibrated depth distribution for a 10-year flood by overtopping frequency

Overtopping frequency	Probability of depth				
	0 - 0.25	0.25 - 0.5	0.5 - 0.75	0.75 - 1	> 1
Remote	0	0	1	0	0
Slight	0	0	1	0	0
Occasional	0	0	0	0	1
Frequent	0	0	0	0	1

By comparing Table 3.13 with Table 3.14 it can be noticed that both flood events have the same impact on bridges with overtopping frequency of Occasional and Frequent since they would be overtopped with both floods. However, for the bridges with Slight or Remote overtopping frequency, category of water surface level of 10-year flood is lower than 100-year flood and consequently, its expected damage would be less.

The updated depth distributions, similar to Table 3.13 and Table 3.14, should be used for each flood event of interest. Other steps of estimating cost of scour risk are unchanged and are the same as calculating expected annual scour cost. However, it should be noted that not necessarily all the bridges in the network should be assessed and only bridges affected by the specified flood should be considered.

A major flood occurred in Upper Mississippi River basin in 2008 that affected eastern parts of Iowa as well as neighboring states. Iowa DOT estimated the damage to the highway network including roadways, culverts, and bridges to be around $15M. As a case study, the modified HYRISK was used to estimate the expected damage from that flood. For that goal, the flooded area was determined and exported to ArcMap. The total of 1,261 bridges were identified to be flooded. Also, since the flood was severe, the values from Table 3.13 that are associated with a 100-year flood were used as depth distribution. Finally, the total bridge scour risk cost of $10,623,201 was estimated which is in line with the actual reported damage.

Conclusions

MAP-21 and FHWA bridge design requirements have been the motivation for state agencies to address different risk items and developing risk-based asset management plans. In bridge management, scour and flooding are among the most important risk items due to the high cost associated with them. These risks are especially more critical in areas with higher precipitation. Several state DOTs (i.e. California, Texas, and Pennsylvania) were successful in developing frameworks for assessing their bridge networks against scour. However, there is still a need for scour management frameworks. The implementation of the procedure developed in this study requires minimal effort and resources and can be helpful for such agencies to have a starting point for scour risk assessment. Use of the modified HYRISK is preferred over the original one and it is recommended that agencies customize the costs and other default values based on their own policies and experiences.

Use of tools such as BridgeWatch enables state DOTs (such as Iowa DOT) to be proactive in monitoring and inspecting their scour critical bridges and focus their resources more efficiently when it is needed. In general, every agency can also benefit from being proactive in scour management in order to identify at-risk bridges before they become vulnerable. As a result, bridges that are more prone to experiencing a scour damage can be identified through both the normal bridge inspection process that every agency does, and also using the proposed methodology in this study. Specifically, the need for such tools is essential for Iowa, where 85 percent of the state-owned on-waterway bridges are built on “Very Erodible” and “Erodible” soils, and majority of them are unprotected. Therefore, the modified HYRISK methodology proposed in this study can be beneficial to agencies to be even more proactive in scour management.

Lastly, the low cost and ease of application of the modified HYRISK model make it usable by any agency. Application of the modified HYRISK requires only six items from NBI, the soil properties, and verification of presence of the scour protections at bridge locations. NBI data is already being maintained by all state DOTs and other agencies for structures that are longer than 20 feet. Therefore, since many agencies keep soil boring data and history of scour protection installation, the model can be implemented with a short data integration process, and without any significant costs. Also, using state-specific flood frequency models, if available, would add to the accuracy of the model results.

Recommendations for Future Work

Determining the overtopping frequency is one of the most important steps toward scour risk assessment. HYRISK uses NBI data to extract the overtopping frequency that can have four possible values of return period ranging from 2 years to 100 years. However, in the process of soil data collection, the overtopping frequencies of several bridges were assessed and it was found that they are not accurate enough. Unfortunately, there was no other available source and the data obtained from NBI was used.

Another issue with HYRISK definition of overtopping frequency is that it does not account for floods with return periods larger than 100 years. Any flood event more severe than a 100-year flood would have the same expected impact on the bridge network which is not true. Therefore, by obtaining and using the actual overtopping frequencies, the resulted risk estimates would be more realistic and more accurate.

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CHAPTER 4. GENERAL CONCLUSIONS

Results from the two studies are beneficial for both network-level and project-level scour assessment. Specifically, Chapter 2 contributes to the reliability of project-level scour assessment and it was shown that estimated flood discharges from different regional flood frequency models can be significantly different. Use of flood frequency models is unavoidable for ungaged sites where no historical peak discharge is available. A group of 67 bridges that experienced the highest changes in their design discharges were identified and it was recommended that Iowa DOT needs to reassess those bridges and perform required actions if needed. Other states also can utilize the methodology developed in this study for evaluating the accuracy of the bridges' design discharges.

The study in Chapter 3 was focused on modifying HYRISK methodology in order to get more accurate and realistic results. The soil erodibility was incorporated into the original procedure along with modifications for adjustment factors and calculation of scour failure cost. The application of the modified HYRISK can be one step toward risk-based asset management and meeting the MAP-21 requirements. The implementation of the modified HYRISK requires minimal resources and personnel and therefore, most of the state DOTs as well as other smaller agencies can benefit from it.

Lastly, the combined results from both studies can be more beneficial toward bridge scour evaluation. Bridge network's resiliency can be assessed for different flood events that can be beneficial for budget allocation. Also, aspects of bridges that are contributing to a higher expected scour risk costs were identified which are valuable for long term planning. The results are most useful for agencies that have not evaluated their bridge network against scour while other agencies can benefit from it as well.