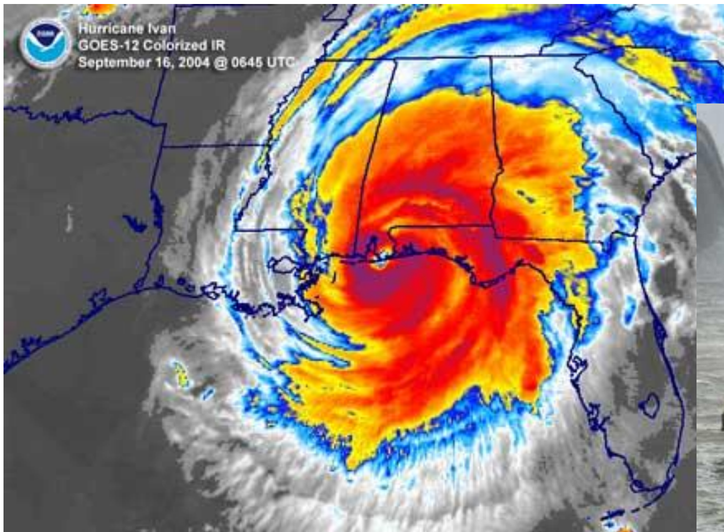
FINAL REPORT

Development of Risk Models for Florida's Bridge Management System



(Reuters)

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Disclaimer

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	Inches	25.4	millimeters	mm
ft	Feet	0.305	meters	m
yd	Yards	0.914	meters	m
mi	Miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	Square inches	645.2	square millimeters	mm^2
ft^2	Square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003).

APPROXIMATE CONVERSIONS FROM SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	Meters	3.28	feet	ft
m	Meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	Hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	Liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		MASS		
g	Grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric	1.103	short tons (2000	T

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		ILLUMINATION		
lx	Lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
N	Newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003).

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16. Abstract

Florida Department of Transportation (FDOT) has been actively implementing the American Association of State Highway Transportation Officials (AASHTO) Pontis Bridge Management System (BMS), recently renamed AASHTOWare Bridge Management (BrM), to support network-level and project-level decision making in the headquarters and district offices. This system is an integral part of a Department-wide effort to improve the quality of asset management information provided to decision makers. With the success of FDOT's previous research efforts, it was necessary to extend bridge management tools and processes to an area that is receiving increasing attention nationally: risk management. The state of Florida is exposed to risk on its bridges from many natural and man-made hazards, including hurricanes, tornadoes, flooding and scour, and wildfires, as well as advanced deterioration, fatigue, collisions, and overloads.

This study developed a comprehensive framework and components of a risk model for these listed hazards. For each hazard, historical data were utilized to develop risk assessment models which predicted the likelihood of such events and also quantified the consequences of the hazard event. Sources of data with several years of recorded events included the following: the Department's databases on bridge inventory and inspection; District's records of damage after hazards; NOAA's climatic data; FEMA; and the Florida Department of Forestry.

The research identified the types of bridges (design type and material type) and specific bridge elements that are most vulnerable to damage under the hazard events. The overall risk model was used to identify the top 20 bridges that are most vulnerable under each of the hazard types. Finally, recommendations are presented as well as modifications to the Project Level Analysis Tool (PLAT).

17. Key Words	18. Distribution Statement
Bridges, risk, hurricanes, tornadoes, wildfires, scour, deterioration, fatigue.	This document is available to the public through the National Technical Information Service, Springfield, Virginia, 22161.

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Unclassified	Unclassified	298	

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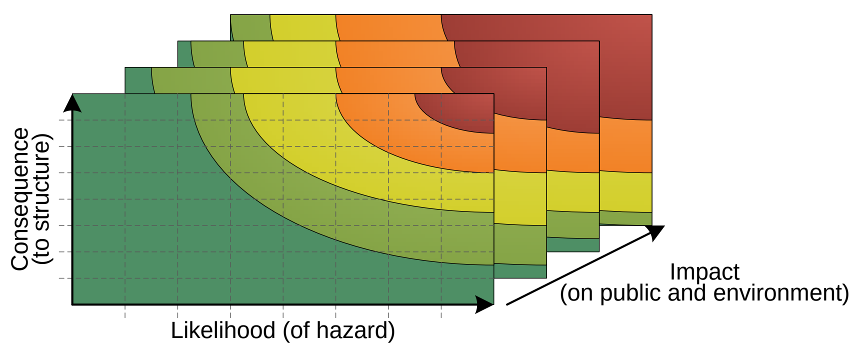
Executive summary

The FDOT has been actively implementing the American Association of State Highway Transportation Officials (AASHTO) Bridge Management Software (BrM) to support network-level and project-level decision making in the headquarters and district offices. The BrM**,** formerly known as Pontis, is an integral part of a FDOT-wide effort to improve the quality of asset management information provided to decision makers. The credibility and usefulness of this information is also essential for satisfaction of the requirements of the Government Accounting Standards Board Statement 34 (GASB 34) regarding the reporting of capital assets. Previous FDOT research has identified analytical needs for implementation of the economic models of the BrM, and has made significant progress in the development of these models. With the success of these research efforts, it was necessary to extend bridge management tools and processes to an area that is receiving increasing attention nationally: risk management. Incorporation of risk assessment and risk management is now being nationally recognized as an improvement to the Pontis.

The bridges in the state of Florida are exposed to risks from many natural and man-made hazards, including hurricanes, tornadoes, flooding and scour, and wildfires, as well as advanced deterioration, fatigue, collisions, and overloads. At the beginning of this study, a review of the current management tools within the FDOT showed that risk management is not being implemented, except for a study done on the application of risk analysis to the project delivery system. In this study, hazards were assessed in terms of their likelihoods, as well as the consequences to the structure and the impact on the public and environment.



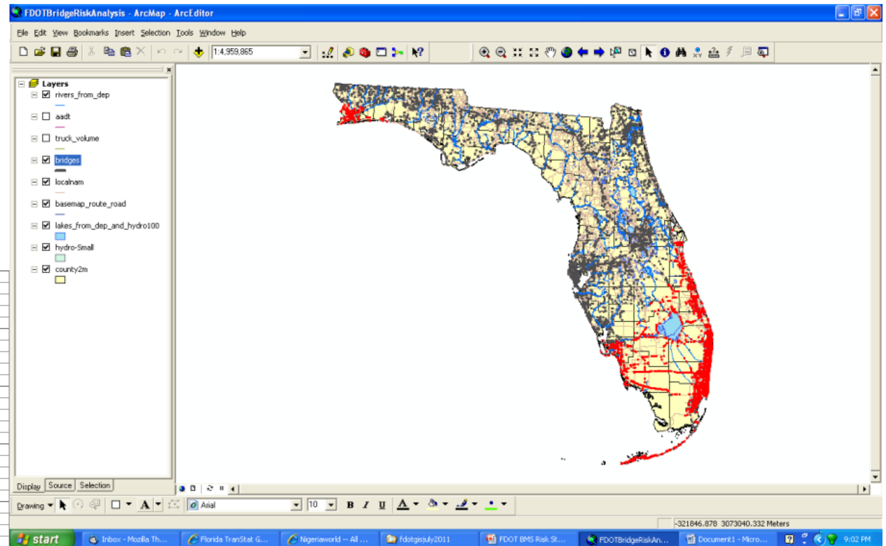
The major accomplishments of this study are summarized in the following paragraphs.

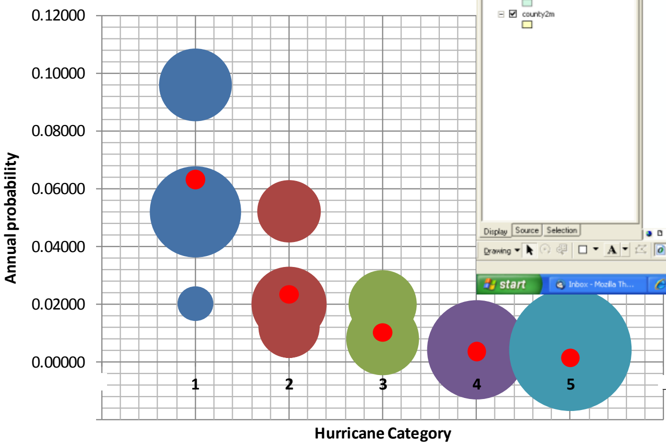
Hazards identification in Florida

Based on review of historical records of hazard events at both national and state levels, it was concluded the predominant natural hazard in Florida is the hurricane, followed by wildfires, tornadoes, flooding, and scour. Earthquake history in Florida was reviewed, and it was found that the hazard is not common in recent times. There are recorded earthquake events of significant intensity in the 1890s and 1990s, but many of them were doubted as being natural earthquakes. Given the high traffic of vehicles, especially those of trucks, on the roadways and vessels on the waterways in Florida, the risk due to collisions, which may also result in fire incidents, was found to also constitute a significant risk to Florida bridges. Lastly, the natural aging of bridges and the associated deterioration make significant the risk of fatigue and advanced deterioration.

Risk models for hurricanes

From published Hazards United States (HAZUS) hurricane maps from FEMA and data from other sources of previous research on hurricane winds, GIS and other analytical tools were used to establish the probability distributions of hurricane wind speeds at Florida bridge locations. By classifying these speeds into hurricane categories (according to the Saffir-Simpson scale) at each bridge location, the probability (likelihood) of having a designated hurricane category wind within a specified period of time was estimated assuming the exponential distribution of times between occurrence of events. It was observed that overall, the mean annual probabilities of hazard events decrease with increase in hurricane intensity (category). As expected, the coastal bridges on Florida's northwest panhandle and bridges south of the Tampa area have significant exposure to hurricane categories 1, 2 and 3, with the Florida Keys being the only area with significant chances of categories 4 and 5.

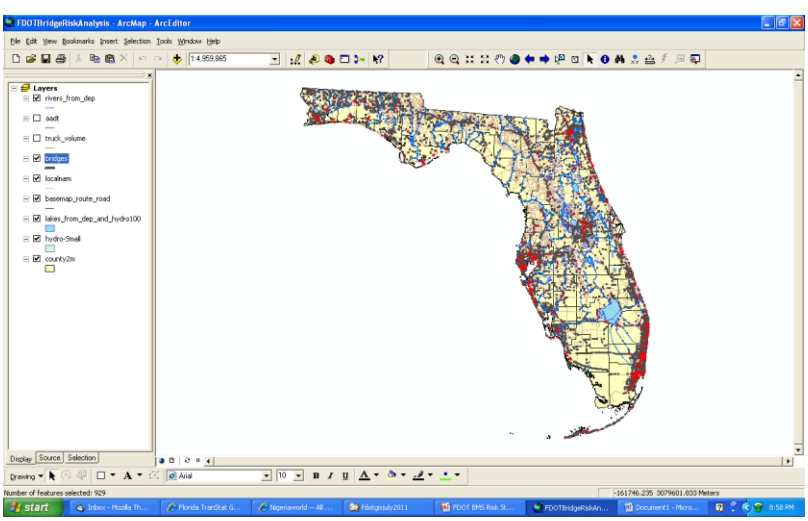




Consequences of hurricanes were reviewed based on the most recent experience of Florida, which was primarily from 2004 to 2006; other recent hurricanes have not really affected Florida in terms of damage to the bridges. The most significant the damage was to the I-10 Escambia Bay Bridges due to Hurricane Ivan in 2006. Other experiences from outside Florida, including those of Hurricanes Katrina and Ike, were also studied in detail to learn what consequences may be applicable to Florida bridges. Data from Florida's inspection after the hurricane events were analyzed to identify which bridge elements were most vulnerable to damage as well as the agency costs of repairs and roadway closure durations. Based on the methodology developed in this study, the top 20 bridges vulnerable to hurricanes were identified.

Risk models for tornadoes

Data were obtained from the National Weather Service GIS Data Portal for tornado events recorded as occurring in Florida from 1950 to 2010, categorized using the Fujita scale. Applying GIS and other analytical tools to these data, estimates were made of likelihood of occurrence of tornadoes at Florida bridge locations. The first approach was to identify how many tornado touch-downs occurred within a one-mile buffer of specific bridges, using this statistic to estimate annual occurrence of tornadoes in that vicinity. The other approach involved using the data recorded on occurrence of tornadoes in each county for the same 1950 to 2010 time period, i.e., establishing annual rates of tornadoes for each county.

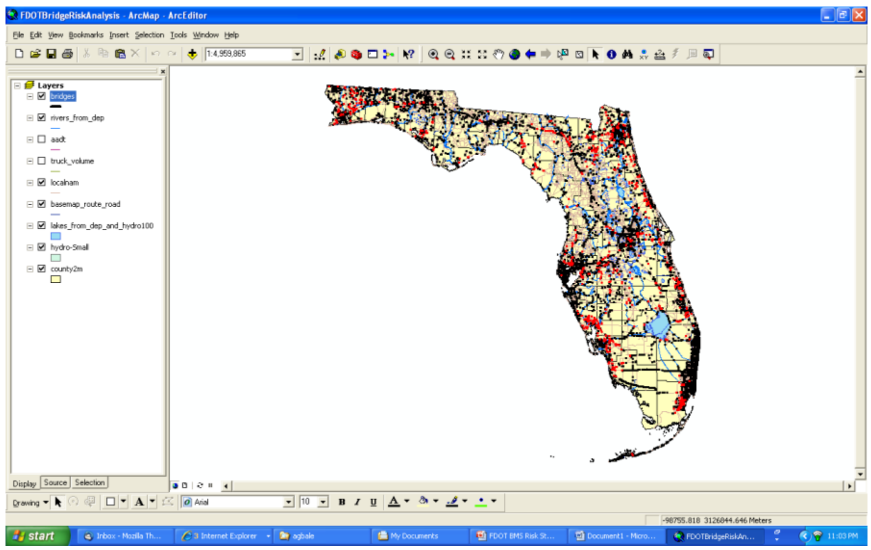


Assuming Poisson occurrence of tornadoes, the probability (likelihood) of occurrence by category was estimated for each bridge location. In the results, it was observed that many bridges (both coastal and inland) in Florida are exposed to the risk of tornadoes, with the mean annual probabilities (about 2% across the categories) being comparable to that of the occurrence of hurricane category 2. It should be noted also that tornadoes sometimes accompany hurricanes.

In terms of the consequences, there was limited data available for damage on Florida bridges due to tornadoes. But from damage reported elsewhere, it was identified that bridges with long spans should be considered very susceptible, particularly narrow, and high level truss. Also, movable bridge elements, and other bridge non-structural elements such as signs, railings, etc., were identified as being very susceptible to damage from the strong winds. Based on the methodology developed in this study, the top 20 bridges vulnerable to tornadoes were identified.

Risk models for wildfires

A detailed data set on historical wildfires (both GIS shapefile formats and databases), with date range of 1980 to 2010, was obtained from the Florida Department of Forestry. These data were analyzed to estimate the likelihood of wildfire occurrence within 1 mile of each Florida bridge. Another set of data from NOAA was also obtained, indicating wildfire occurrence in each Florida county for the period 1996 to 2010. Each set of data produced annual rates of wildfire occurrence near the bridges and in each county. Assuming Poisson process for the events, as for the hurricane and tornado hazards, the probability was calculated for the occurrence of wildfire near Florida bridges.



If a wildfire engulfs a bridge, the damage that occurs is dependent on the bridge material. Timber is the most vulnerable, followed by steel and then concrete. Since the wildfire intensity is much less than that of, say, a fuel tanker explosion, serious damage may be restricted to specific bridge elements, such as structural timber and non-structural elements such as railings, signs, lighting, etc. rather than total destruction. Though the data available for wildfire damage on Florida bridges are limited, it can be reasonably assumed that road closures constitute the biggest threat to roadways and bridges, rather than physical member damage. Based on the methodology developed in this study, the top 20 bridges vulnerable to wildfires were identified.

Wildfire							
Bridge key	Element vuln	Age vuln	Design vuln	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Elem VulnW	Age vulnW	Des vulnW	Haz probW	Risk indexW	Agcy costW	User costW
720343	1.095	1.571	1.667	20.263	0.5813	8.41	0.41
720249	1.062	1.571	1.667	20.263	0.5635	8.15	0.40
480035	1.067	1.857	1.667	17.625	0.5819	7.05	10.21
720076	1.126	2.057	1.667	20.263	0.7824	6.81	3.97
720114	1.035	1.657	1.667	20.263	0.5793	3.26	0.07
720107	1.095	1.657	1.667	20.263	0.6130	3.13	1.38
150107	0.970	1.886	1.333	9.243	0.2254	2.78	0.10
760043	1.139	1.371	1.667	20.263	0.5275	2.46	17.51
570091	1.039	0.914	1.333	17.625	0.2233	2.29	1.12
870592	1.188	1.029	1.667	9.268	0.1887	2.29	0.05
900101	0.843	1.229	1.333	9.268	0.1280	2.20	2.44
180940	0.972	1.686	1.333	15.700	0.3431	2.11	0.81
470029	1.014	2.486	1.667	17.625	0.7408	2.06	0.05
120002	1.107	1.800	1.667	15.308	0.5082	1.95	0.84
780056	1.107	1.829	1.667	20.263	0.6834	1.94	2.65
720153	1.098	1.886	1.667	20.263	0.6993	1.84	2.28
490100	0.967	0.629	1.333	17.625	0.1428	1.81	0.73
490032	0.961	1.057	1.333	17.625	0.2387	1.80	1.74
580174	1.020	0.743	1.333	17.625	0.1780	1.76	0.14
720518	1.097	1.029	0.333	20.263	0.0762	1.74	0.66

Risk models for floods

Much of Florida is at low elevations at or near sea level. Coupled with the state's frequent experience with hurricanes and tropical storms, flooding is a common occurrence at Florida bridges in riverine and tidal locations. To assess the likelihood of flooding at bridge locations, two sets of GIS data were acquired: one from the FEMA Map Office and the other from the Florida Geographic Data Library (FGDL). Using GIS tools, Florida bridges were assigned risk levels as follows: high risk zones with 100-year floods, i.e., their annual rate of occurrence is 1/100 or 1%; moderate risk or 500-year floods, with their annual rate of 0.2%; and the moderate-to-low-risk zones considered to be outside the flood plains, or in other words, have zero annual rate of occurrence. Estimating the likelihood of flooding is most accurately done on a bridge-by-bridge basis, where the detailed hydrology and hydraulics data for the specific location can be critically analyzed. For the objectives of this research, use of the FEMA flood data was deemed adequate. The consequences of flooding could not be well quantified in this study due to lack of historical data on flooding effects on bridges. But the observed damage to Florida bridges due to hurricanes was partially used to infer the damage expected from flooding. The vulnerability of bridge elements to damage during flooding varies, with channel elements being most vulnerable, followed by culverts, approach slabs, slope protection, walls, footings, and movable bridge elements. Based on the methodology developed in this study, the top 20 bridges vulnerable to floods were identified.

Risk models for scour

The occurrence of scour is associated with hurricanes and floods, thus scour may be classified as a secondary, or consequential hazard rather than a primary hazard. Nevertheless, the increased scour resulting from hurricanes and floods is a real hazard to Florida bridges and must be considered. Two approaches were considered in predicting scour at bridge locations. The first one is an elaborate mechanistic approach which is well described in the Florida Scour Manual and other publications where the soil properties, hydraulic data, bridge geometric attributes, and other pertinent data are utilized, through various equations to estimate the scour depth. The other approach is primarily empirical, where National Bridge Inventory (NBI) data are used, with a bit of theoretical consideration, to establish the likelihood of scour and the risks. The latter has been made popular by the FHWA, as evidenced in the HYRISK Software and also the Unknown Foundation Procedure Manual for Florida Bridges. Some historical data on river elevations, basically in the form of hydrographs (gauge heights and discharge) are available for some Florida locations at NOAA's National Weather Service website and linked USGS websites. In this study, it was demonstrated how this data can be used to predict the probability of scour by associating the overtopping frequency with scour vulnerability. Unfortunately, many of the data sites have incomplete data or data that are only provisional and not validated yet. This detailed type of information can be and are assumed to have been used by the bridge inspectors to assess the overtopping frequency at each bridge location for entry into the NBI Item 71 Waterway adequacy. Moreover, the FDOT Districts and State Drainage Office will have access to more accurate and complete flood data for assessing the overtopping frequency. Also the Pontis database's *userbrg* table has a field (*scrrating)* which is a good estimate of the likelihood of occurrence of scour.

Risk models for vehicle or vessel collisions and bridge overloads

Vehicular crashes on bridge roadways, including collisions with bridge elements constitute real hazards, and various studies have been conducted to identify reasons for vessels colliding with bridge members in the waterways. The former type of accident has been known to result in significant fire hazard, especially when trucks carrying flammable materials are involved, for example, fuel tankers. To estimate the likelihood of these occurrences, various models were developed and some suggested for future development. Vehicular crash rates were estimated, including annual crash

probabilities for trucks. The Florida vehicle classification scheme was applied to identify the proportion of the traffic stream that would be fuel tankers; this refines the probability of truck crashes that may result in fire at the bridge location. There is a limitation in this approach due to the unavailability of such vehicle classification data for all roadways in Florida. Collision risks were estimated for vehicles and trucks, as well as consequences expected based on historical records of crashes at Florida bridge locations. Specific types of collisions considered also included those due to roadway surface accidents, over-height vehicle collisions, and vessel impact collisions. Using the parameters suggested in the AASHTO Specifications for Vessel Collision Design of Bridges, a methodology was developed and suggested for predicting likelihood of vessel impacts on the Florida bridge substructures. Based on the methodology developed in this study, the top 20 bridges vulnerable to collisions and bridge over-height cases were identified.

Risk models for bridge advanced deterioration

A common and general concern of risk management is the unavoidable disruption of service due to the need to respond proactively to impending hazards. If bridge maintenance is deferred for a prolonged period, the condition of the structure reaches a point where the agency is forced to take action to ensure safe mobility. The action may be posting, closure, strengthening, or partial or complete replacement. All of these actions disrupt service, forcing road users to expend more time and fuel in congestion or detours. They also force the agency to expend public funds on the action. This study developed tools to identify opportunities where the agency can apply strategic preventive maintenance actions and postpone the need for more expensive forced activities. By analyzing Florida bridge inventory data, service disruptions were correlated to the deteriorated condition of the bridge to develop a disruption likelihood model. The condition of the bridge was represented by a decay index formulated from a two-stage process modeled on the concept of the bridge health index. By computing the life cycle costs of service disruption, a cumulative risk profile was formulated and used to modify the Project Level Analysis Tool (PLAT). The analysis also produced, as a by-product, the data necessary to compute failure probabilities for Pontis 4.5. Based on the methodology developed in this study, the top 20 bridges vulnerable to advanced deterioration were identified.

Risk models for bridge fatigue

Fatigue is a deterioration process where a material flaw, initially microscopic in size, develops into a larger defect and eventually a crack. This occurs because of a concentration of stress in the vicinity of the flaw, which is cyclically applied by the passage of live loads on the structure or by distortion of the structure. The crack grows, and this situation may quickly lead to complete failure of the structural member. For bridge management analysis, fatigue of structural steel is of great significance. In this study, risk was measured using the product of likelihood and consequence, and described as vulnerability to fatigue cracking as the applicable hazard. A risk mitigation action, crack repair, is developed in order to reduce the likelihood of member failure. Replacement of the superstructure (or of the entire bridge) is an action which can restore the fatigue life of the structure, while resilience is the remaining fatigue life of the bridge. Based on Florida data, 519 structures were identified as requiring fracture-critical inspections. The likelihood of cracking was estimated using an adaptation of the approach described in the National Cooperative Highway Research Program (NCHRP) Report 495, which is based on the AASHTO fatigue life model. Consequences were also computed in terms of agency costs and impact to road users. Based on the methodology developed in this study, the top 20 bridges vulnerable to fatigue were identified.

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1. Introduction

The FDOT has been actively implementing the American Association of State Highway Transportation Officials (AASHTO) Bridge Management Software (BrM) to support network-level and project-level decision making in the headquarters and district offices. The BrM was formerly known as Pontis BMS, and FDOT is still using the Pontis database for its BMS operations. Thus Pontis will be used to reference the FDOT BMS throughout this report. Pontis is an integral part of a FDOT-wide effort to improve the quality of asset management information provided to decision makers. The credibility and usefulness of this information is also essential for satisfaction of the requirements of the Government Accounting Standards Board Statement 34 (GASB 34) regarding the reporting of capital assets.

Previous FDOT research has identified analytical needs for implementation of the economic models of the Pontis BMS, and has made significant progress in the development of these models. With the success of these research efforts, it is now proposed to extend bridge management tools and processes to an area that is receiving increasing attention nationally: risk management.

The state of Florida is exposed to many natural and man-made hazards, including hurricanes, tornadoes, flooding, landslides, and wildfires. With highway bridges and other structural elements constituting important lifelines on the transportation network, it is very important to incorporate these risks into the decision-making processes at the FDOT. For example, in 2004, Hurricane Ivan damaged the Interstate-10 Escambia Bay Bridge, in Pensacola, Florida, resulting in enormous agency and user costs to the people of Florida and the nation as a whole. The Pontis inspection records also indicate some of the damages from this hurricane to sign structures. Hurricanes can produce violent winds, tornadoes, storm surge, and floods. Many bridges in Florida are vulnerable to both coastal and riverine flooding. Landslides, best described as earth flows on slopes due to gravity, may block or damage bridge channels and slope pavements, and worsen a flooding situation. Wildfires may also constitute a hazard to bridge structural elements, especially structural steel superstructures and substructures. Additional risks that are being considered include fatigue, scour, over-height truck impact, ship impact, and overloads. Generally, in addition to the natural and man-made hazards, two other types of risks are considered: vulnerability of the structure and users due to advanced deterioration; and also risk to users due to substandard roadway width. Also, incorporation of risk assessment and risk management is now being nationally recognized as an improvement to Pontis.

A review of the current management tools within the FDOT shows that risk management is not being implemented, except for a study done on the application of risks analysis to the project delivery system.

1.1. Research objectives

Risk assessment is a process to estimate the likelihood and consequences of an identified hazard, while risk management considers the warrants, costs, and benefits of mitigating actions. The main goal of the proposed research was therefore, to develop a framework and also implement risk assessment and risk management models in the Florida Pontis BMS.

An important deliverable of this research was a framework of a transportation asset risk model that can be applied to other transportation assets (pavements, culverts, guardrails, signs, etc.) and decision making cases in the FDOT, including the Offices of Planning, Design, Traffic, and to some extent, Structures. The research also provided useful data, analytical tools, and a report describing the methodology and updating procedures for future use by the FDOT. These are not available for use by the headquarters Maintenance Office and by the District Structures Maintenance Engineers (DSMEs) in the FDOT's maintenance planning processes, and will be of great interest to the entire national bridge management community beyond Florida.

The specific objectives are listed as follows:

- Conduct an extensive literature review, internationally, and national, including a review of the FDOT's management practices to identify any prior and current application of risk assessment and risk management techniques.
- Develop a risk assessment model, including formally identifying the types of hazards, the relevant adverse events that could occur, estimating the likelihood of these events, and also estimating the consequences of each adverse event.
- Develop performance measures based on a methodology that would scale the risk to a hazard value function.
- Identify risk mitigation actions for each type of hazard identified and modeled.
- Develop guidelines for a new risk assessment process based on the results of the study, including recommendations for any changes in the bridge inspection process.
- Modify the Florida Project Level Analysis Tool (PLAT) and Network Level Analysis Tool (NAT) to incorporate the new risk models and the utility function as a prioritization criterion.
- Prepare the project final report describing the study methods and results, as well as recommendations for implementation of the results and for future research.

1.2. Framework and definitions

FDOT has a variety of means at its disposal to manage risk in the structure inventory. The present study is concerned primary with the use of project selection and programming decisions to control risk. But risk can also be managed through design, maintenance, and operational decisions. For example:

- Modern bridges are often provided with structural redundancy as a design principle, so fracture of any one member is less likely to cause catastrophic failure (a design decision).
- If a bridge is found to have damage affecting its load-bearing capacity, certain emergency maintenance actions, such as cribbing or carbon fiber wrapping, can sometimes be used in order to restore its capacity (a maintenance decision).
- If it is not cost-effective to restore a bridge's load-bearing capacity, then the bridge may be posted to restrict the usage of the structure to ensure safety (an operational decision).
- If an extreme natural hazard event, such as a hurricane, is underway, a vulnerable bridge may be closed until the hazard has passed (an operational decision).

A complete and efficient risk management strategy combines all of these tools. Because of these and other available measures, the sudden failure of a bridge under traffic is extremely rare. Nonetheless, certain hazards still present a safety concern, including earthquakes, tornadoes, vehicular and vessel collisions, and sudden fracture or buckling on non-redundant structures.

1.2.1. Service disruption

A much more common and general concern of risk management is the unavoidable disruption of service due to the need to respond pro-actively to impending hazards. If bridge maintenance is deferred for a prolonged period, the condition of the structure reaches a point where the agency is forced to take action to ensure safe mobility. The action may be posting, closing, strengthening, or partial or complete replacement. All of these actions disrupt service, forcing road users to expend more time and fuel in congestion or detours. They also force the agency to expend public funds on the action. Of course, the sudden damage or destruction of a structure due to a natural extreme event is also a form of service disruption, having especially severe consequences and impacts.

One of the key life cycle tradeoffs in bridge management is the possibility of strategic preventive maintenance actions to postpone the need for more expensive forced activities. A purpose of Pontis and the PLAT is to identify these opportunities. Accurate evaluation of preventive activities requires the use of tools to quantify the negative impacts of allowing conditions to deteriorate.

1.2.2. Elements of risk

In general, a risk analysis model consists of three elements (Figure 1):

- Likelihood model, quantifying the probability that a hazard will arise and cause an actual disruption of service.
- Consequence model, quantifying the direct effect of the hazard on the structure, including the agency response, and its immediate agency cost, that is forced by the hazard.
- Impact model, quantifying the indirect effect of the hazard on the public and the environment. While the public may not be aware of deteriorated conditions that necessitate action, they are still impacted by congestion and detours that result from the agency response to the hazard.

In the framework introduced here, likelihood is expressed as a probability, in percent. Consequence is expressed as a choice of agency action, or a set of choices, with an estimate or expected value of cost. Impact will be expressed in the form of social cost, the sum of agency, user, and (when appropriate) non-user costs. An alternative to social cost is to use a unitless utility function as a means of setting priorities among alternative investments. Since Florida DOT has historically relied on user cost in its bridge management decision making processes, the preference is to continue to express project benefits in this way if possible.

The current memorandum, responding as it does to Tasks 2 and 3 of the study, focuses on the likelihood element of the risk model. Later tasks will address consequences and impact.

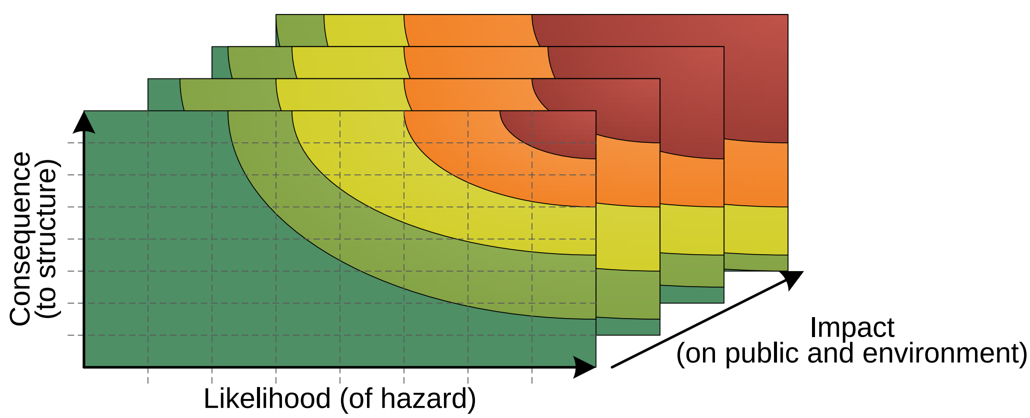

Figure 1.1. Risk as the product of likelihood, consequence, and impact of hazards

1.2.3. Alternative performance measures

When concepts of risk are used in communications with stakeholders and the public, they are easily and incorrectly confused with unsafe conditions. As a result, agencies rarely use the term "risk" directly when presenting assessments and needs, especially at the asset level. It is desirable to emphasize that operational policies and procedures are in place to prevent a hazard from becoming an unsafe condition. It is more accurate to present risk as the possibility of service disruption, which may be caused by deterioration, by exogenous hazards (hurricanes, fires, etc.), or by operational procedures necessary to maintain safety.

At the asset level, risk management is often measured using the product of likelihood and consequence, and described as vulnerability of an asset to exogenous hazards. Risk mitigation actions are developed in order to reduce either the likelihood, or consequence, or both. These reduce the vulnerability of the asset, or increase its resilience. Most performance measures in asset management are expressed as positive qualities of the asset, such that it is desired to increase the value of the performance measure or prevent the decline of the measure over time. Thus, it is common to use "resilience" as a measure of risk avoidance in asset management (Exhibit 2).

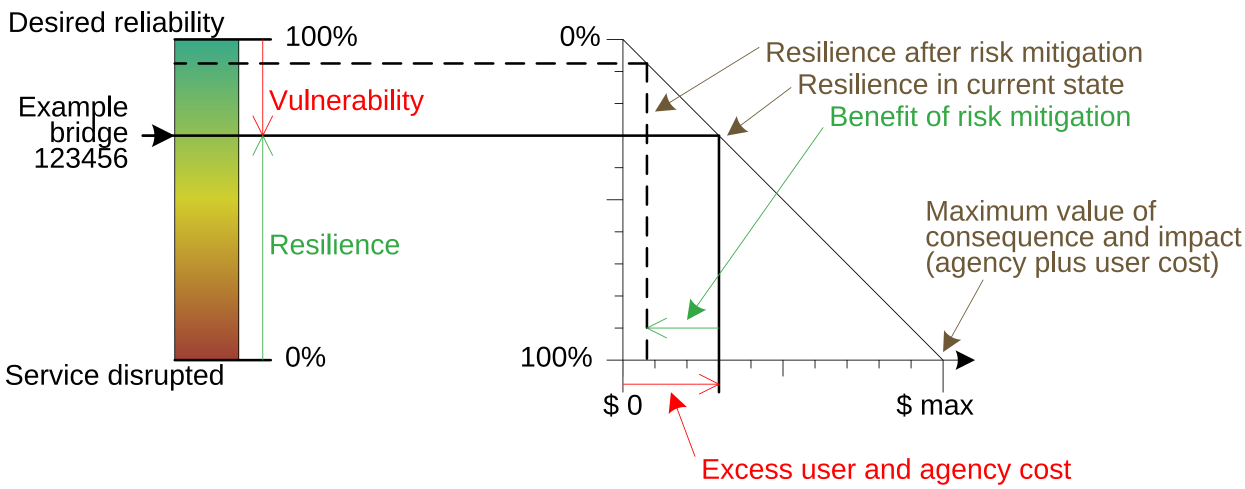

Figure 1.2. Framework for quantifying risk

At the network level, all three components of risk (likelihood, consequence, and impact) are necessary in order to fully describe the quantity to be managed, and to set priorities among alternative investments. "Risk" is less easily confused with "lack of safety" at the network level, and so is somewhat more commonly used at the network level. However, it is also common to speak of the vulnerability or resilience of the transportation network as a whole. "Resilience" is often preferred because it is defined in a positive direction in the same manner as most other network level asset management performance measures.

For the present study, it is desired to express performance in the form of economic measures if possible. Therefore, a lack of resilience, at either the asset level or the network level, will be expressed as an excess social cost. The social cost framework for risk management works in the same way as the functional improvement framework presently used in Florida (Thompson et al., 1999). It is not necessary to quantify total social costs of the transportation network, but only the excess social costs that arise because of resilience that is below desired levels.

When resilience of an asset is increased due to risk mitigation actions, the result is a marginal decrease in expected value of social cost. This marginal social cost may include travel time, vehicle operating costs, and accident costs. (It may also include non-user costs such as those related to air quality, but this is beyond the scope of the present study.) The positive contribution of a risk mitigation action will therefore be measured as user benefit, defined as a marginal reduction in expected social cost.

1.2.4. Available data

The risk model to be developed is derived as a network level model, in that it is meant to be a general model that can be applied to the wide range of structures in the inventory. Therefore the model must rely on comprehensive data sources, and not on anecdotal descriptions of hazards. This is especially problematic when structural failures are relatively uncommon and each failure is unique. There is no laboratory where a scientific sample of bridges can be allowed to deteriorate to failure under realistic conditions of weather and traffic. Similarly, highly deteriorated conditions in the Florida inventory are uncommon and are routinely avoided by active management processes.

Florida's Pontis database is the only comprehensive source of data on historical conditions and events of service disruption. Therefore a resourceful data mining of Pontis is necessary to develop the required models.

1.3. Literature review

Various documented studies and articles related to analysis of hazards on bridges are presented here, first in general terms, and then in details for some specific pertinent studies.

Using user costs and accident risk during the construction phases, Corotis et al. (2010) presented a risk-based analysis of Colorado bridges to identify key factors that would explain the differences between the various structure types. Adey et al. (2003) presented a methodology on determining the optimal intervention for inadequate levels of service due to multiple environmental hazards to be used for bridge management strategies. Zayed et al. (2007) proposed and developed a risk index (R) for risk assessment and prioritization of bridges with unknown foundations to assist bridge managers. Zayed et al. (2007) provided practitioners with risk parameters and factors for the evaluation and ranking of bridges. Primary risk parameters including but not limited to the bridge type, cost, bridge geometry, substructure system, bridge age, design life of bridge, type of bridge foundation, bridge conditions, potential loss of life, soil characteristics, average daily traffic (ADT) and average annual daily traffic (AADT), scour, seismic vulnerability, value of lost time, detour length, what the bridge passes over (water or land or both) were collected from ten bridges in the states of Florida, Indiana and New York for evaluation in the case study.

In the event of an emergency, the New Zealand Defense Emergency Management (CDEM) Act (2002) requires that road networks be functional to the fullest possible extent. Seville et al. (2005) carried out a research study which focuses on the challenge of assessing the risk of road closures for the State Highway network. They considered risk as a function of the following: the likelihood and magnitude of a hazard event; the vulnerability of the road network to damage from that event; and the social, environmental and economic impacts of any damage or disruption to the road network and subsequent traffic flows, summed over the full spectrum of hazards and hazard magnitudes capable of impacting on the road network.

Seville et al. (2005) suggests a framework that uses a walkthrough scenario approach, in which hazard events in New Zealand are randomly simulated over a period of time, using a Geographical Information

System (GIS) analysis. Ayyub et al. (2009) developed an analytic and probabilistic risk analysis methodology for protected hurricane-prone regions to assist decision and policy makers. Ideas on developing the risk and vulnerabilities of bridges in Florida due to both man-made and natural disasters were developed by Lachance (2005) and Lazlo (2008). The model used data including statistical bridge data, weather data compiled from hurricane and tornado history in Florida, and Flood data. Lazlo (2008) and Lachance (2005) applied Geographic Information System (GIS) analysis methods to transportation management while introducing methods of risk management to develop an infrastructural management system. The results from Lachance (2005) and Lazlo (2008) were very general and in some cases limited to data from one county.

Pinelli et al. (2004) presented a model for predicting damage after a hurricane but for residential buildings and not for highway bridges. The model is based on defined damage modes for buildings and the Monte Carlo simulations of hurricane wind speeds on engineering numerical models of the building types. Stewart (2010) reviewed risk-based approaches and describes risk acceptability (based on fatality risks, failure probabilities, and net benefit assessment) and cost-effectiveness of protective measures for infrastructure. The decision support framework considers hazard and threat probabilities, value of human life, physical and indirect damages, risk reduction, and protective measure costs. An example application is given for a bridge over an inland waterway where the hazard is ship impact.

The aging and deterioration of bridges was considered by Padgett et al. (2010) in the risk analysis of bridges subjected to earthquake and hurricane hazards where the bridge elements experience seismic and surge/wave loading respectively. Mackie (2010) described for bridges experiencing seismic events, the sensitivity of the probabilistic repair cost and time metrics to changes in repair quantities, unit costs, production rates, and correlation at the demand and damage levels. With focus on coastal bridges, Ataei (2010) presented the use of bridge fragility to assess the risk to the bridges posed by hurricane-induced storm surge and wave. Efforts are discussed on the development of probabilistic models of the bridge vulnerability subjected to hurricane scenarios, and sensitivity studies are presented on the significance of varying hazard and bridge parameters on the dynamic response of coastal bridges.

Fragility curves or functions are typically developed for specific bridge elements subjected to damage resulting from hazards such as earthquake (Choe et al., 2010; Sullivan and Nielson, 2010; Alipour et al., 2010; Ramanathan et al., 2010; Seo and Linzell, 2010). According to Ramanathan et al. (2010), fragility curves are condition probability estimates of the likelihood that a structure will meet or exceed a specified level of damage for a given ground motion intensity measure. These curves could be developed from expert opinions, empirical data, and analytical methods. Maconochie (2010) incorporates concepts from risk-based asset management, as well as reliability theories. The model was designed to be utilized by transportation agencies for effective management of their programs “by providing a systematic risk-based perspective to investment decision making.” Instead of creating a model for the failure of a bridge, Maconochie (2010) creates a model that predicts the mean time to a service interruption (situation that causes a bridge owner to perform emergency acts and/or restricts the use) of the bridge.

1.3.1. Thompson et al. (2012)

Thompson et al. (2012) described the development of a tool, the Bridge Replacement and Improvement Management (BRIM) system, to rank potential bridge projects by directly considering the risk of an interruption to service in Minnesota DOT’s long term planning process. The database in the tool will help project stakeholders to understand how the DOT prioritizes and programs bridge projects for future contracts. The database and tool contain a risk assessment model to provide a consistent rating and ranking of Minnesota bridges using the principles of risk management. The BRIM consists of a set of risk evaluation models that consider the major natural and man-made hazards affecting the bridge inventory:

advanced deterioration of deck, superstructure, and substructure; scour; fracture criticality; fatigue; overweight and over-height trucks.

1.3.2. Consolazio et al. (2010)

The focus of this article is further developing the probability of collapse expressions for bridge piers subject to barge impact loading. During the collision of a traveling water vessel, such as a large barge, with a structural component of a bridge, there is a large horizontal force transferred into the bridge superstructure that could possibly cause structural collapse. This article develops and improves expressions measuring the probability of structural collapse due to this type of collision. These probabilistic collapse expressions developed in this study are meant to serve as an aid in the design of bridges for vessel collision. Through the use of probability analysis, along with the aid of finite element analysis of barge-pier collision simulations, the authors propose new structural component designs to better withstand vessel impact at the critical impact locations they have determined.

Now although vessel impact is not an environmental hazard, it poses a very serious hazard to coastal bridges and needs to be addressed. Also, the methods for the development of the probability expressions for vessel collision can be very similar to those of environmental hazards. The paper quantifies the probability of vessel impact on structural elements of bridges that traverse waterways, specifically, the collapse of bridge piers from vessel impact, not just the general collapse of the entire structure like in the previous articles. The authors go into elaborate detail in developing various finite element models of the bridge piers and the barge-bow representing the vessel that induces the impact, providing detailed results of deformation and collision forces.

One of the major components of this paper is further developing the relationship between vessel impact deformation and the damage inflicted on the structural components of the bridge. This also provides insight into how much detail can be put into developing probability expressions, which under similar methodology, can assist in dealing with risk due to environmental hazards.

One very direct and important application of this paper's methodology is how the authors went about identifying which bridges to look at. In other words, the approach they took to select what spectrum of bridges to select to undergo study. In this article, it is shown that there is great importance in selecting a set of bridge cases that represent a wide range of bridge types. Many other articles in this literature review apply a method in which only bridges of a certain category are addressed. For example, there are other journals mentioned in this review that focus on addressing the risk a bridge is vulnerable to structural damage from storm surges experienced in hurricanes. The only bridges of interest in addressing that specific environmental hazard were low elevation, concrete slab and girder bridges that cross coastal waterways.

1.4. Risk assessment: hazards identification

In general, civil infrastructures are exposed to the following hazards: Scour, Flood (coastal and riverine); Earthquake; Fatigue/Fracture; Impact (vehicular and vessel); Advanced deterioration; Vehicular crash (substandard width or alignment); Hurricane; Tornado; Landslide; Wildfire; and Other (e.g. Terrorism). The first step taken in developing the risk assessment model was by formally identifying the types of hazards, the relevant adverse events that could occur, and the likelihood of these events.

1.4.1. Hazards in Florida

According to the FEMA (2011) website, the recent history of nationally-declared disasters resulting from natural hazards in Florida is listed below in Tables 1.1 to 1.5. Most are related to hurricanes, severe storms, tornadoes, and flooding while a few fire hazards have occurred in recent times. Table 1.6 shows a summarized breakdown of the various types of natural hazards as they affect bridges. Not shown in Table 1.6, is another set of hazards including those due to impact to the bridge superstructure by vehicles (e.g., fuel tankers) or to bridge substructures by vessels on waterways. These two types of hazard will be classified Man-made hazards (Unintentional). Nothing is mentioned about seismic-related (earthquakes) disasters in Florida, but this category of natural hazard will be discussed later in terms of its history in Florida.

Table 1.1. Recent history of Florida hurricanes (FEMA 2011)

Year	Date	Disaster	Classification
2008	10/27	Hurricane Gustav	Category 4
2005	10/24	Hurricane Wilma	Category 3
2005	08/28	Hurricane Katrina	Category 3
2005	07/10	Hurricane Dennis	Category 3
2004	09/26	Hurricane Jeanne	Category 3
2004	09/16	Hurricane Ivan	Category 3
2004	09/04	Hurricane Frances	Category 2
2004	08/13	Hurricane Charley and Tropical Storm Bonnie	Category 4
1999	10/20	Hurricane Irene	Category 2
1999	09/22	Hurricane Floyd	Category 2
1998	09/28	Hurricane Georges	Category 2
1998	09/04	Hurricane Earl	Category 2
1995	10/04	Hurricane Opal	Category 3
1995	08/10	Hurricane Erin	Category 2
1992	08/24	Hurricane Andrew	Category 5
1985	12/03	Hurricane Kate	Category 3
1985	09/12	Hurricane Elena	Category 3
1979	09/13	Hurricane Frederic	Category 3
1968	11/07	Hurricane Gladys	Category 1
1965	09/14	Hurricane Betsy	Category 3

Table 1.2. Recent history of Florida tornadoes (FEMA 2011)

Year	Date	Disaster
2007	02/08	Severe Storms, Tornadoes, and Flooding
2007	02/03	Severe Storms and Tornadoes
2003	04/25	Tornado
1998	02/12	Severe Thunderstorms, Tornadoes and Flooding
1998	01/06	Tornadoes
1994	11/28	Tropical Storm Gordon, Heavy Rain, Tornadoes, Flooding
1993	03/13	Tornadoes, Flooding, High Winds, Tides, Freezing
1992	10/08	Flooding, Severe Storm, Tornadoes
1979	05/15	Severe Storm, Flooding, Tornadoes

Table 1.3. Recent history of Florida fires (FEMA 2011)

Year	Date	Incident
2009	5/21	Martin County Fire Complex*
2008	5/21	Brevard Fire Complex*
2007	6/27	Okeechobee Fire Complex*
2007	5/9	Caloosahatchee Fire Complex*
2007	5/8	Black Creek Fire*
2007	5/7	Suwannee Fire Complex*
2007	5/2	Deland Fire Complex*
2007	3/26	53 Big Pine Fire*
2006	5/15	Volusia Fire Complex*
1998	06/18	Florida Extreme Fire Hazard

*Fire Management Assistance Declarations

Table 1.4. Recent history of Florida severe and tropical storms (FEMA 2011)

Year	Date	Disaster
2009	05/27	Severe Storm, Tornadoes, and Straight-line Winds
2009	04/21	Severe Storm, Tornadoes, and Straight-line Winds
2007	02/08	Severe Storm, Tornadoes, and Flooding
2007	02/03	Severe Storm and Tornadoes
2003	07/29	Severe Storm and Flooding
2001	09/28	Tropical Storm Gabrielle
2001	06/17	Tropical Storm Allison
2000	10/03	Tropical Storm
1998	11/06	Tropical Storm Mitch
1998	02/12	Severe Thunderstorms, Tornadoes and Flooding
1996	10/15	Severe Storm/Flooding
1995	10/27	Severe Storm, Flooding
1994	11/28	Tropical Storm Gordon, Heavy Rain, Tornadoes, Flooding
1994	07/10	Severe Storm, Flooding, Tropical Storm Alberto
1992	10/08	Flooding, Severe Storm, Tornadoes
1992	08/14	Flooding, Severe Storm
1990	04/03	Flooding, Severe Storm
1982	07/07	Severe Storm, Flooding
1979	09/29	Severe Storm, Flooding
1979	05/15	Severe Storm, Tornadoes, Flooding
1973	05/26	Severe Storm, Flooding

Table 1.5. Recent history of Florida flooding (FEMA 2011)

Year	Date	Disaster
2007	02/08	Severe Storm, Tornadoes, and Flooding
2003	07/29	Severe Storm and Flooding
2000	10/04	Heavy Rains And Flooding
1998	02/12	Severe Thunderstorms, Tornadoes and Flooding
1996	10/15	Severe Storm/Flooding
1995	10/27	Severe Storm, Flooding
1994	11/28	Tropical Storm Gordon, Heavy Rain, Tornadoes, Flooding
1994	07/10	Severe Storm, Flooding, Tropical Storm Alberto
1993	03/13	Tornadoes, Flooding, High Winds, Tides, Freezing
1992	10/08	Flooding, Severe Storm, Tornadoes
1992	08/14	Flooding, Severe Storm
1990	04/03	Flooding, Severe Storm
1982	07/07	Severe Storm, Flooding
1979	09/29	Severe Storm, Flooding
1979	05/15	Severe Storm, Tornadoes, Flooding
1975	09/26	High Winds, Heavy Rains, Flooding
1975	08/22	Flooding
1973	05/26	Severe Storm, Flooding
1970	07/03	Heavy Rains, Flooding
1953	10/22	Flood

Table 1.6. Breakdown of natural hazards in Florida based on recent history

Natural Hazard	**Specific Types**	**Potential effects on bridge structure**
1. Tropical Cyclone	Hurricanes/ Severe and Tropical Storms	Excessive wind forces
		Storm surge waves
	Tornadoes	Excessive wind forces
	Flooding (Coastal)/Landslides	Channel erosion/deposit
		Slope failure
		Scour
		Debris impact Approach slab undermining
2. Flooding (Riverine)/ Landslides		Channel erosion/deposit
		Slope failure
		Scour
		Debris impact
3. Tornadoes (Inland)		Excessive wind forces
4. Seismic (Earthquake/Tsunami)	N/A	N/A
5. Fire	Wildfire	Extreme material temperature

According to USGS (2011a) Florida is not usually a state that commonly experiences earthquakes, though some minor shocks have been recorded in the state. It was stated that only one of these shocks caused damage. There have been other recorded shocks but of doubtful seismic origin. In January 1879, a shock occurred near St. Augustine, in the northeast part of the state. This is the largest earthquake recorded to have been centered in Florida. The tremors and shocks were felt as far as Daytona Beach, Tampa, north and central Florida, and at Savannah, Georgia. In January 1880, two strong earthquakes were centered in Cuba that sent severe shock waves through Key West, Florida. A famous shock occurred in Charleston, South Carolina, in August 1886 that was felt throughout northern Florida, including strong aftershocks that occurred in September, October and November 1886. Jacksonville experienced a slight shock and a minor earthquake in June 1893 and October 1900, respectively. In Captiva Island, in the Gulf of Fort Myers, an apparent earthquake, though accompanied by sounds like distant heavy explosions, was experienced in November 1948. In November 1952, Quincy, near Tallahassee, experienced a slight tremor but with no serious effects noted. Some shocks of doubtful seismic origins are recorded for the Everglades-La Belle-Fort Myers area in July 1930, Tampa in December 1940, and the Miami - Everglades - Fort Myers area in January 1942. Most authorities attribute these incidents to some sort of explosion.

Also posted on the website is an article by Mott (1981) indicating that slight tremors have been felt in various parts of Florida as follows: November 1952 in Lake City and Quincy; March 1953 in Orlando; in 1973 in central Florida; in December 1973 in Seminole and Orange counties; in 1975 around Daytona Beach; and in October 1977, an Earthquake Seismograph Station became operational at the University of Florida, with another tremor recorded in November 1977 over the Florida Peninsular.

National Seismic Hazard Maps are developed by the U.S. Geological Survey (USGS) to display earthquake ground motions for various probability levels across the United States and are applied in seismic provisions of building codes, insurance rate structures, risk assessments, and other public policy (USGS 2011b). The maps are derived from seismic hazard curves calculated on a grid of sites across the United States that describe the frequency of exceeding a set of ground motions. The most recent version available on the USGS website is for 2008 and is shown in Figure 1.3 for the entire United States, and specifically for the state of Florida in Figure 1.4, including the probabilities of exceeding given ground motion accelerations.

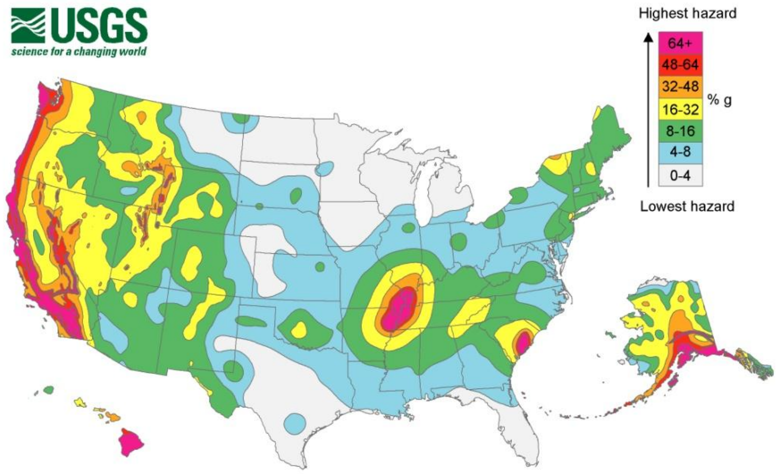

Figure 1.3. National seismic hazard map 2008 (USGS 2011b)

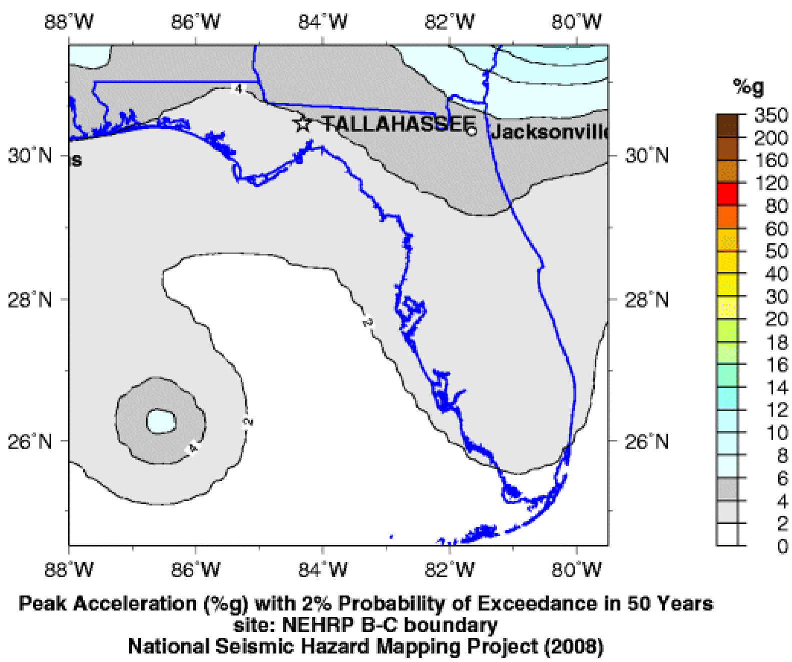

Figure 1.4. Seismic hazard map for Florida (USGS 2011b)

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2. Hurricanes

Over half of the hurricane-related damage in the United States occurs in the state of Florida, and with approximately 85% of the rapidly increasing population situated on or near the 1,900 km of coastline, Florida losses will continue to mount in proportion to coastal population density (Pinelli et al., 2004). Florida has quite a history with hurricanes and thus gets the majority of the news and appears that it is a major state in the path of Hurricanes. The typical classification scheme for hurricanes is on the Saffir–Simpson Hurricane Wind Scale shown in Table 2.1, based on the 3-sec. sustained wind speeds, and relating the speeds to damage to physical properties. This classification scheme is described in more details later in this report.

Table 2.1. Hurricane Classification – The Saffir–Simpson Hurricane Wind Scale (NOAA 2012)

Category	Wind Speed mph (km/h)	Storm surge ft (m)	Type of Damage
1	74 – 95 (119-153)	4 - 5 (1.2-1.5)	***Very dangerous winds will produce some damage:*** Well-constructed frame homes could have damage to roof, shingles, vinyl siding and gutters. Large branches of trees will snap and shallowly rooted trees may be toppled. Extensive damage to power lines and poles likely will result in power outages that could last a few to several days.
2	96 – 110 (154-177)	6 - 8 (1.8-2.4)	***Extremely dangerous winds will cause extensive damage:*** Well-constructed frame homes could sustain major roof and siding damage. Many shallowly rooted trees will be snapped or uprooted and block numerous roads. Near-total power loss is expected with outages that could last from several days to weeks.
3 (Major)	111-129 (178-208)	9 – 12 (2.7-3.7)	***Devastating damage will occur:*** Well-built framed homes may incur major damage or removal of roof decking and gable ends. Many trees will be snapped or uprooted, blocking numerous roads. Electricity and water will be unavailable for several days to weeks after the storm passes.
4 (Major)	130 – 156 (209-251)	13 – 18 (4.0–5.5)	***Catastrophic damage will occur:*** Well-built framed homes can sustain severe damage with loss of most of the roof structure and/or some exterior walls. Most trees will be snapped or uprooted and power poles downed. Fallen trees and power poles will isolate residential areas. Power outages will last weeks to possibly months. Most of the area will be uninhabitable for weeks or months.
5 (Major)	>156 (>251)	>18 (>5.5)	***Catastrophic damage will occur:*** A high percentage of framed homes will be destroyed, with total roof failure and wall collapse. Fallen trees and power poles will isolate residential areas. Power outages will last for weeks to possibly months. Most of the area will be uninhabitable for weeks or months.

During a hurricane, bridge structures suffer from a variety of damage, impact damage from floating debris, as well as erosion around the substructure due to storm surge and increased levels of floodwaters at a bridge site and scour, which is considered the most common form of hurricane damage to bridges. There are two types of scour hazards: scour as an extreme event hazard (from floods and hurricanes); and scour as a long-term cumulative hazard (from steady stream flow especially during the spring runoff season). The extreme event type of scour is usually associated with the cause of the event such as hurricane damage), while the long-term type of scour is merely called “scour” in common usage. The combined effects of these damages often lead to “the deterioration of rip rap by the abutments, undermining of bridge approaches, and damage to the areas behind the wing walls of many structures.” (Stearns and Padgett, 2011). Impact Damage on bridges is usually caused as storm surge and floodwaters collect large amount of debris, causing considerable amounts of damage to local bridges. Many bridges had debris resting against the superstructure which affected post-event functionality, while others suffered visible structural damage due to debris impacts (Stearns and Padgett, 2011).

Storm surge and wave loading also accompany hurricanes when the combined storm surge and waves rise to a level at or above the bottom of the bridge superstructure, the deck is subjected to uplift and transverse forces that can cause severe damage to the structure. The forces acting on the bridge are comprised of both a hydrodynamic and a hydrostatic component. The drag force, inertial force, and buoyancy force make up the hydrostatic component of the force, and the slamming force caused by trapped air effects makes up the hydrodynamic component (Sheppard and Marin, 2009). Given the limited connection capacity provided between many bridge superstructures and substructures, this loading can shift or even completely displace the deck of the bridge.

The following section presents some notable documented studies related to hurricanes and their impacts on bridges.

2.1. Hurricane-related studies

Stearns and Padgett (2011) presented some analyses of the damage to bridges in the Houston/Galveston region of Texas observed after Hurricane Ike. This hurricane originated off the coast of Africa, impacting islands in the Bahamas and Cuba before making landfall in the upper Texas coast on September 13, 2008 as a category 2 hurricane. In addition to post-event inspections, data were collected from consultants, Texas DOT, and through interviews of bridge owners. Using hindcast data developed by other researchers at University of Texas at Austin, for Hurricane Ike, surge and wave heights generated by the hurricane were identified at the damaged bridge locations. It was revealed that the peak storm surge level and wave heights were significant at various locations. Some non-coastal rural (inland) bridges not in the surge zone also suffered scour/impact damage, probably due to flooding associated with the hurricane. It was observed that many of the damaged bridges were either timber bridges or low-clearance water-crossing bridges. About half of these bridges were damaged or completely destroyed by the storm surge and wave loading, while the other half experienced damage due to scour around abutments and wing walls, as well as impact damage from debris (shown in the form of spalling and chipping of concrete).

In terms of the scour damage, it was noted that the combined effect of the storm surge, flooding, increased water levels and flow often led to the deterioration of rip rap by the abutments, undermining of bridge approaches, and damage to the areas behind the wing walls of many structures. As shown Figure 2.1, the damage to the approach also displaced the superstructure on a particular bridge multiple inches below its original height.

Figure 2.1. Bridge approach undermined by scour due to Hurricane Ike (Stearns and Padgett, 2011)

As an example of the impact damage, debris was observed to have bent guardrails on a bridge. Due to storm surge height, estimated as 12.9 feet with 3.1-foot waves at a timber bridge location, the bridge deck was displaced completely off the timber support structure, as shown in Figure 2.2. The bridge deck and the support bent connections were inadequate to resist the uplift forces, and they were ripped from the support columns. Another location with storm surge of 14.2 feet with 5.5-foot waves, had the deck and support bents missing, with only the support piers from the structure remaining (Figure 2.3).


Figure 2.2. Deck displacement due to storm surge and waves (Stearns and Padgett, 2011)

Figure 2.3. Deck displacement due to storm surge and waves (Stearns and Padgett, 2011)

Three major highway bridges were affected by Hurricane Ike. One of these bridges was a concrete box girder bridge with five spans on concrete pile substructure constructed in 1984. The bridge deck, 72.5 feet wide, consisted of 18 pre-stressed concrete box girders, each 1.67 feet thick with a 3-inch finish grade of asphalt on top. The end spans of the bridge were 45 feet long, and the middle three spans were 50 feet long. The support bents and abutments for the structure rested on 12 spiral-bound concrete piles. The NBI condition ratings in 2008 were “Very Good” for deck, “Good” for superstructure and substructure, and “Satisfactory” for the channel. The bridge had a deck clearance of approximately 5.3 feet over mean water elevation, and during Hurricane Ike the bridge was subjected to a 15-foot surge with 5-foot waves. The bridge suffered major shifting of many of the sections of the bridge and complete loss of some of the sections, yielding displacement in four of the five spans. It was revealed during site visits that minimal spalling had occurred on the bent beams along with complete loss of most connections between superstructure and substructure. The other major bridge had concrete slab on steel girders for some spans and timber girders for others. This bridge experienced shifting of some spans due to the storm surge and wave loading. The third major bridge was a concrete slab bridge with a movable bascule section in the main span. The major damage observed were due to erosion to the approach roadway, riprap, and support columns. The bascule also suffered severe damage to its fenders.

The levels of damage were defined, as shown in Table 2.2 where for example, “Light damage” implies when there is visible repairable damage that does not affect structural strength. As shown partially in Table 2.3, the damage observed include the following: approach slab/erosion; deck unseated; deck displaced; electrical failure; spalling; adjacent roadway: inundation and flooding.

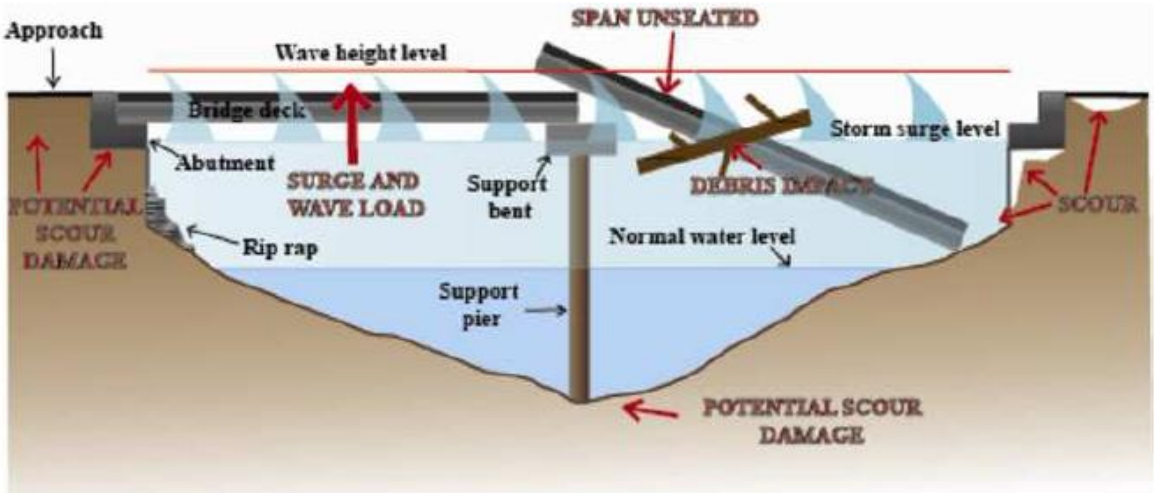


Figure 2.4. Illustration of common failure modes in bridge damage during hurricane event (Source Stearns and Padgett, 2011)

Table 2.2. Damage state definitions and descriptions (Stearns and Padgett, 2011).

Damage state	Description
Light	Some repairable damage to the superstructure. No immediate danger.
Medium	Minor damage to the superstructure and possibly substructure of the bridge. Possible loss of structural integrity.
Heavy	Major damage to entire bridge structure. Severe loss of structural integrity, posing public danger.
Destroyed	Bridge structure unusable or missing.

Table 2.3. Partial listing from summary of damaged bridges in Houston/Galveston region (Stearns et al., 2011).

Bridge Name	Damage State	Failure Mode	Spans	Substructure Material	Superstructure Material	County	Function
Wayne Morris #1	Destroyed	deck unseated	3	Timber	Timber	Chambers	Service Road
Wayne Morris #2	Destroyed	deck unseated	2	Timber	Timber	Chambers	Service Road
Wayne Morris #3	Destroyed	deck unseated	2	Timber	Timber	Chambers	Service Road
Wayne Morris #4	Destroyed	deck unseated	4	Timber	Timber	Chambers	Service Road
Rollover Pass Bridge	Destroyed	deck unseated	5	Concrete	Concrete	Galveston	Major Arterial
Jay Matthews	Heavy Damage	scour damage	3	Timber	Timber	Chambers	Service Road
Rodeo Bend	Heavy Damage	scour damage	1	Concrete	Concrete	Galveston	Local
Frenchtown Rd.	Heavy Damage	scour damage	1	Concrete	Concrete	Galveston	Local
Boondocks Bridge	Heavy Damage	scour damage	6	Steel	Steel	Jefferson	County Road
Humble Camp Bridge at Hildebrandt Bayou	Heavy Damage	deck displaced	24	Concrete, Steel, Timber	Steel, Concrete	Jefferson	County Road
Craigen Bridge	Heavy Damage	scour damage	5	Steel	Steel	Jefferson	County Road

Padgett et al. (2008) presented a very detailed review of bridge damage and repair efforts resulting from the Hurricane Katrina, collecting data related to 44 bridges, with an overall cost to repair or replace the bridges totaling about $1 billion. In general, it was observed that storm surge was severe in the coastal areas, with most damage done to superstructures but also cases of scour damage, inundation of electrical and mechanical equipment, and some wind damage (Padgett et al., 2008).

Starting with the damage due to surge-induced loading, the most severe failure mode was unseating of individual spans, particularly low-elevation spans. Specifically, this includes the following: deck displacement; spans shift but with no complete loss of support, but leading to pounding and damage to abutments, bent caps, and girders; bearing damage along with the deck displacement and span shifting; and loss of or damage to parapets on the bridge decks due to a combination of the storm surge and wind loading. The damaged spans were those with elevation at or below estimated peak storm surge levels. The types of bridges affected were traditional fixed bridges, with continuous spans, simply-supported spans, and movable bridges with fixed portions experiencing storm surges.

Impact damage was also reported, due to barges, oil drilling platforms, tug boats, and other debris. The damage modes were described as span misalignment, damage to fascia girders (spalling of concrete and breaking of prestressing strands), damage to fenders, and pile damage. Scour damage was presented in terms of scour and erosion of abutment, slope failure, and undermining of bridge approach. Damage due to water inundation primarily affected movable bridges with damage in the flowing modes: on submerged electrical and mechanical equipment (not designed for extended wetting or submersion in rushing flood water); debris accumulation affecting functioning of mechanical gears; bent pivots; fractured mechanical parts; and damage to traffic control gates. The water inundation destroyed lift motors and electrical systems, disabling the bridge. It was stated that 80% of emergency repair costs on a particular bridge was for replacement of the electronics damaged by water inundation.

High winds from the hurricane may actually worsen the damage modes described above, increasing the potential for impact and debris; facilitating larger surges, waves, and horizontal pounding. The high winds would also cause structural damage to operator houses and machinery housing in movable bridges, as well as cause damage to the electrical cables on the towers of the movable bridge.

Figure 2.5. US-90 Biloxi-Ocean Springs Bridge showing the primary mode of failure in severely damaged bridges: span unseating due to storm surge-induced loading (Padgett et al., 2008)

a.

b.

Figure. 2.6. Damage to bent caps (a); bridge parapet (b) due to storm surge (Padgett et al., 2008)

a.

b.

Figure 2.7. Misaligned span due to barge impact on the I-10 Bridge at Pascagoula, Miss. (a), resulting pier damage (b) (courtesy of MDOT) (Padgett et al., 2008)

a.

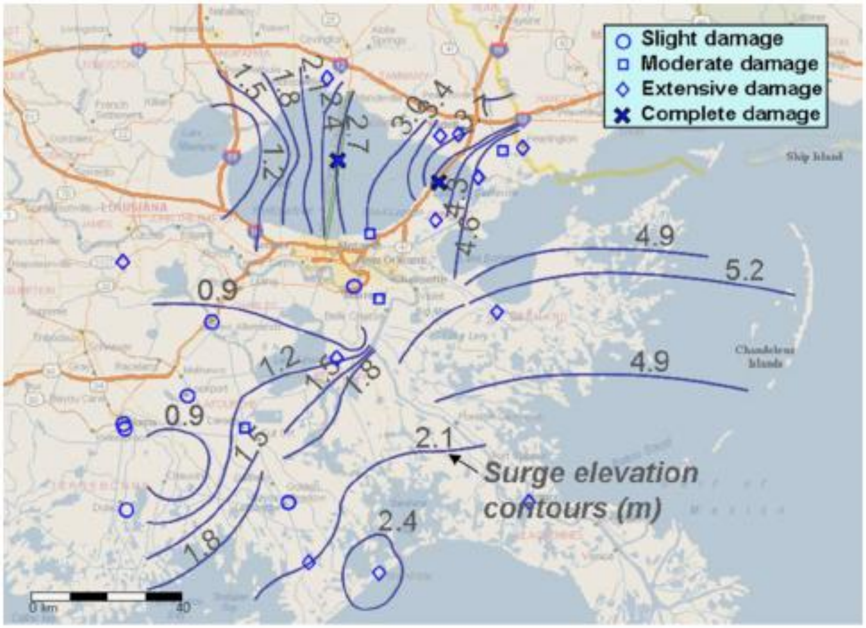
b.

Figure 2.8. Abutment and approach damage from scour and erosion (a) courtesy of MDOT; (b) courtesy of LADOT (Padgett et al., 2008)

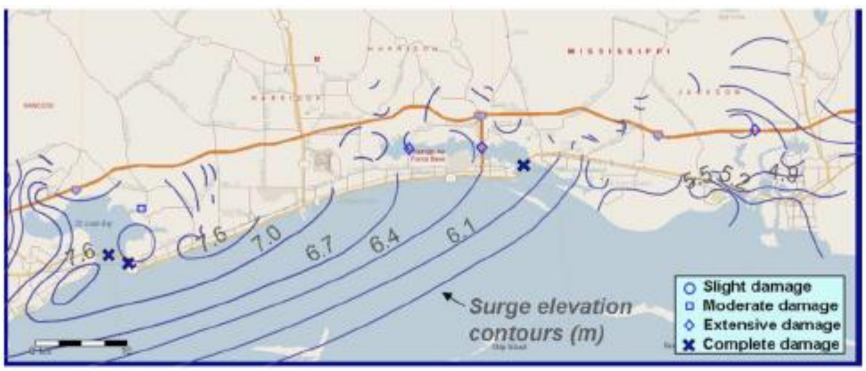
In describing the damaged states of the bridges, the FEMA table is shown in Table 2.4. The qualitative descriptions used for discriminating slight, moderate, extensive, and complete damage for bridges presented in HAZUS from seismic events are used in this study with additional descriptions included for more hurricane-specific damage (FEMA 2003).

Table 2.4. Qualitative damage state descriptions defined by amending HAZUS for typical hurricane-induced bridge damage Stearns and Padgett, 2011)

Damage state	Description
Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair), minor cracking to the deck, or slight damage to operator house.
Moderate	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2 in.), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure, moderate settlement of the approach, moderate scour of the abutment or approach, damage to guardrails, wind and/or water damage to operator house resulting in switchboard or content damage.
Extensive	Any column degrading without collapse—shear failure (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments, extensive scour of abutments, or submerged electrical or mechanical equipment.
Complete	Any column collapsing or connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.



a.



b.

Figure 2.9. Damaged bridges relative to storm surge contours in (a) Louisiana; (b) Mississippi (Padgett et al., 2008)

The repair and replacement cost was initially expensive for emergency repairs needed to restore immediate functionality of the bridges, especially with a higher priority on the Interstate-10 bridges before addressing the local bridges. Major damage and replacement projects were let out as contract bids while minor damage was often repaired in house (by state DOTs). Repair costs were reported as ranging from an estimated $275 million for replacement of the Biloxi-Ocean Springs Bridge carrying US-90 in Mississippi, to less than $1,000 for minor repairs of damaged operator houses on movable bridges in Louisiana. The cost estimates were assumed based on the findings of the TLCEE (2006) reconnaissance using preliminary DOT inspection reports and estimates, costs of work completed to date, and bid estimates.

It was found that slightly damaged bridges (all movable bridges found in Louisiana that typically suffered slight damage to the operator house and to gates and signals) had repair costs of less than $10,000. For bridges in the extensive damage state, there was more variation in the repair costs, ranging from $25,000 to nearly $7.7 million. The repair or replacement costs of completely damaged bridges ranged from $1.9 million to $275 million, depending upon the size of the bridge, number of the spans, etc. Though the size of the bridge may have also been a factor, it was shown that the repair or replacement costs are directly related to the level of damage on the bridge.

The main focus of Padgett et al. (2009)'s article is to use empirical data from bridge damage during recent Hurricanes to develop probabilistic bridge vulnerability estimates through statistical analysis. This article is intended to be an initial step towards risk assessment of coastal bridges and highway systems subjected to hurricane or storm surge events. The dataset collected in order to perform the statistical analysis of the bridge damage during hurricane uses the 2005 Hurricane Katrina reconnaissance which the results are presented in Padgett's Bridge Damage and Repair Costs from Hurricane Katrina. Data was also collected from the national bridge inventory database for locating bridge numbers, characteristics, and geographic coordinates for the damaged bridges as well as for the total bridge set within the surge region. Surge estimates developed by FEMA were also used. Surge elevations were interpolated from GIS from the surge contours. The last dataset employed in this study is the output of a Katrina hindcast developed by researchers at the University of Notre Dame which was used to estimate water velocity and wind speed estimates experienced during the storm.

With this empirical data, the author develops estimates of the conditional probability of meeting or exceeding different damage states, given the surge elevation at that location. Padgett then makes point estimates of exceedance probabilities for the undamaged bridges in the region by directly comparing the hazard at the damaged bridge locations relative to the undamaged bridges. In order to make the comparison between the damaged bridges and the undamaged ones, a homogeneous bridge sample is required. In this article all low-elevation, multi-span, simply supported, concrete water crossing bridges were chosen. By doing this, the author develops damage probability matrices which are then presented in the form of fragility curves. The author states that the fragility curves developed can be used in validation of future analytical studies that simulate bridge damage from surge and wave action in hurricanes. Possible uncertainty in the fragility curves can be due to both the limited damage data available from the Katrina bridge inventory data and also bias from the specific storm and characteristics. The author states "Regression analysis using this information provides the first set of fragility curves for bridges subjected to hurricane induced storm surge hazards. These statements of conditional probability of failure can be used in estimating damage potential for similar coastal bridges, validating their use for an estimate for future coastal bridge damage in other areas, given the predicted storm surge heights are known.

Hayes (2008) addresses risk based hazard to Delaware's coastal bridges based on the coastal bridge hurricane storm surge model developed by Max Sheppard (Sheppard and Miller, 2003). The journal assesses vulnerability of coastal bridges to storm surge and wave forces of Delaware's bridge inventory based on the work of Florida hurricane studies. The study performs trial assessments on 3 Delaware coastal bridges based on the three-level vulnerability analysis procedure developed by Max Sheppard and Ocean Engineering Associates, Inc (OEA) for Florida's coastal bridges and to see how applicable this procedure is for coastal bridges on the Middle Atlantic coast. In this analysis method, initially, a level I analysis (most conservative and simple) is performed, and if there exists a significant threat, a higher level (II or III) analysis would be warranted and performed (according to the article, Level II and III analyses shall be conducted by a qualified coastal engineer).

After implementing this method on the three case study bridges chosen along Delaware's coastline, it was shown that all of these bridges were assumed to be non-vulnerable to damage from storm surge waves from hurricanes according to the criteria presented in Max Sheppard's method. The article will prove a useful example for applying this storm surge vulnerability assessment on Florida's coastal bridges and may also prove insightful in developing a method to apply this analysis approach to a network system of coastal bridges in Florida's bridge database.

The relevant applicable codes used in this article are AASHTO LRFD Bridge Design Specifications, Shore Protection Manual of the US Army, Coastal Engineering Manual, and Max Sheppard's Guide Specifications for Bridges Vulnerable to Coastal Storms. An important criterion for bridges at risk defined by this method is that "Vertical clearances of highway bridges should be sufficient to provide at least 1 ft of clearance over the 100-yr design wave crest elevation, which includes the design storm water elevation."

2.2. Risk assessment: likelihood estimates for hurricanes

It is important to be able to forecast the future occurrence of these hazards near bridge locations. Towards this goal, historical data is needed in order to develop probabilistic models for such forecasts or prediction. This section presents results of the efforts to collect the pertinent data and also analyze them as inputs towards the development of risk assessment models for the hurricane hazard.

2.2.1. The Saffir-Simpson hurricane wind scale

Though mentioned briefly in the previous section, the Saffir-Sampson scale is described here again, based on NOAA (2012), as it relates to the hurricane categories. It is a 1 to 5 categorization based on the hurricane's intensity at the indicated time. The scale has been an excellent tool for alerting the public about the possible impacts of various intensity hurricanes, providing examples of the type of damage and impacts in the United States associated with winds of the indicated intensity. The maximum sustained surface wind speed (peak 1-minute wind at the standard meteorological observation height of 10 m [33 ft] over unobstructed exposure) associated with the cyclone is the determining factor in the scale. The historical examples provided in each of the categories correspond with the observed or estimated maximum wind speeds from the hurricane experienced at the location indicated. These do not necessarily correspond with the peak intensity reached by the system during its lifetime. It is also important to note that peak 1-minute winds in a hurricane are believed to diminish by one category within a short distance, perhaps a kilometer of the coastline. For example, Hurricane Wilma made landfall in 2005 in southwest Florida as a Category 3 hurricane. Even though this hurricane only took four hours to traverse the peninsula, the winds experienced by most Miami-Dade, Broward, and Palm Beach County communities were Category 1 to Category 2 conditions. However, exceptions to this generalization are certainly possible.

The scale does not address the potential for other hurricane-related impacts, such as storm surge, rainfall-induced floods, and tornadoes. It should also be noted that these wind-caused damage general descriptions are to some degree dependent upon the local building codes in effect and how well and how long they have been enforced. For example, building codes enacted during the 2000s in Florida, North Carolina and South Carolina are likely to reduce the damage to newer structures from that described below. However, for a long time to come, the majority of the building stock in existence on the coast will not have been built to higher code. Hurricane wind damage is also very dependent upon other factors, such as duration of high winds, change of wind direction, and age of structures.

Earlier versions of this scale – known as the Saffir-Simpson Hurricane Scale – incorporated central pressure and storm surge as components of the categories. The central pressure was used during the 1970s and 1980s as a proxy for the winds as accurate wind speed intensity measurements from aircraft reconnaissance were not routinely available for hurricanes until 1990. Storm surge was also quantified by category in the earliest published versions of the scale dating back to 1972. However, hurricane size (extent of hurricane-force winds), local bathymetry (depth of near-shore waters), topography, the hurricane's forward speed and angle to the coast also affect the surge that is produced. For example, the very large Hurricane Ike (with hurricane force winds extending as much as 125 mi from the center) in 2008 made landfall in Texas as a Category 2 hurricane and had peak storm surge values of about 20 ft. In contrast, tiny Hurricane Charley (with hurricane force winds extending at most 25 mi from the center) struck Florida in 2004 as a Category 4 hurricane and produced a peak storm surge of only about 7 ft. These storm surge values were substantially outside of the ranges suggested in the original scale. Thus to help reduce public confusion about the impacts associated with the various hurricane categories as well as to provide a more scientifically defensible scale, the storm surge ranges, flooding impact and central pressure statements are being removed from the scale and only peak winds are employed in this revised version -- the Saffir-Simpson Hurricane Wind Scale as shown in Table 2.1

2.2.2. Hurricane map data

The primary source of data for the hurricane winds was the Federal Emergency Management Agency's (FEMA) HAZUS software program. The HAZUS has extensive national data for three natural hazards – earthquake, hurricane, and floods. A request was made to FEMA, and the software was obtained (free), along with the pertinent historical data. Running the program generates hurricane maps in a Geographic Information System (GIS) format for desired regions in the U.S., in this case, Florida. The map layers show hurricane wind speeds corresponding to various return periods for specific census tracts in the state. A return period simply means the time between reoccurrence of the wind speed. For example, a wind speed of 98 mph at a return period of 20 years in a census tract implies that the tract will experience such a wind once every 20 years.

Using ESRI ArcGIS 9.3, simple spatial join commands were used in the ArcToolBox to query the overlay between the Hurricane GIS layer and the Florida DOT bridge GIS layer. The intersection of the layers yielded the bridges (Structure ID, roadway ID, Roadside (C, L, or R),) lying in each census tract, acquiring the listed hurricane wind attributes (wind speeds at return periods ranging from 10 years to 1000 years). A sample of merged data is shown in Table 2.5 below. For some few cases such as Structure IDs 150055, and 156701 in Table 2.51, where two or three tracts, contribute to the wind data at a bridge location, an average is taken of the wind speeds at each return period. Typically, these are adjoining tracts and the values are very close.

Table 2.5. Sample hurricane map data (wind speeds and return periods) for bridges

				wind speeds (mph) at return periods (yr.)						
ROADWAY	ROAD_SIDE	STRUCTURE_	tract	10	20	50	100	200	500	1000
15150000	C	150055	12103024902	68.6	83.3	100.3	111.4	121.7	135.6	143.4
15150000	C	150055	12103024905	68.7	83.2	100.3	111.3	121.4	134.9	143.3
15240000	C	150125	12103024902	68.6	83.3	100.3	111.4	121.7	135.6	143.4
15061000	C	150177	12103024904	68.8	83.4	100.3	111.4	121.5	134.8	143.6
15008500	C	154203	12103025401	68.6	82.8	99.5	110.3	119.7	132.6	142.2
15514000	C	154207	12103025401	68.6	82.8	99.5	110.3	119.7	132.6	142.2
15504000	C	154403	12103024901	68.8	83.5	100.5	111.6	121.9	135.4	143.4
15000114	C	154406	12103024901	68.8	83.5	100.5	111.6	121.9	135.4	143.4
15000444	C	156701	12103024904	68.8	83.4	100.3	111.4	121.5	134.8	143.6
15000444	C	156701	12103024905	68.7	83.2	100.3	111.3	121.4	134.9	143.3
15000444	C	156701	12103024906	68.7	83.3	100.4	111.5	121.7	135.3	143.3

2.2.3. Analysis of hurricane wind data

Li and Ellingwood (2006) stated that the two-parameter Weibull distribution provided a reasonable and appropriate fit to the probability distribution of hurricane winds, making validating to the earlier similar results from Vickery et al. (2000), Peterka and Shahid (1998), and Batts et al. (1980). The Weibull cumulative distribution function (CDF), defined based on the scale and shape parameters, u and α, respectively, is given as

$$F_V(v) = P(V < v) = 1 - \exp\left[-\left(v/u\right)^{\alpha}\right] \qquad 2.1$$

According to Li and Ellingwood (2006), using wind maps such as provided in the FEMA's HAZUS data, the parameters u and α are location-specific and can be determined from the relationship between wind speed, v_T , and return period, T , where

$$v_T = u\left[-\ln\left(1/T\right)\right]^{1/\alpha} \qquad 2.2$$

Using two pairs of variables wind speed, v_T , and return period, T, for example v_{20} and v_{1000} corresponding to return periods 20 years' and 1000 years respectively, the Weibull parameters can be estimated using equation 2.2 above. First the shape parameter α is estimated as follows.

$$\frac{1}{\alpha} = \frac{\ln\left(\frac{v_{1000}}{v_{50}}\right)}{\ln\left(\frac{-\ln\left(1/1000\right)}{-\ln\left(1/50\right)}\right)}$$

Then the scale parameter u is estimated using one of the pairs of variables, for example,

$$u = \frac{v_{1000}}{\left(-\ln\left(1/1000\right)\right)^{1/\alpha}}$$

The main use of such Weibull functions will be for simulation of probable wind speeds at the specific locations. For example, based on the HAZUS map data, the variability of hurricane winds at structure ID 154403 with the data shown in Table 2.5 was estimated as a Weibull distribution with shape and scale parameters equal to 1.495 and 39.433 respectively. The distribution plots are shown in Figures 2.10 and 2.11. The probability of exceeding 110 mph wind speed at this bridge location is approximately 0.01.

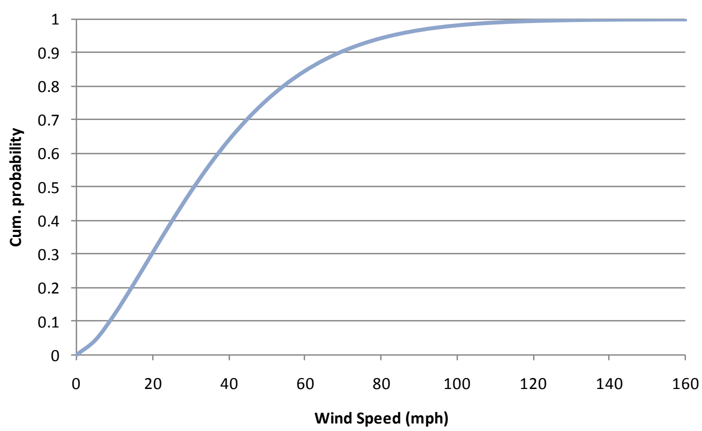


Figure 2.10. Cumulative probability distribution of wind speeds for Structure ID 154403

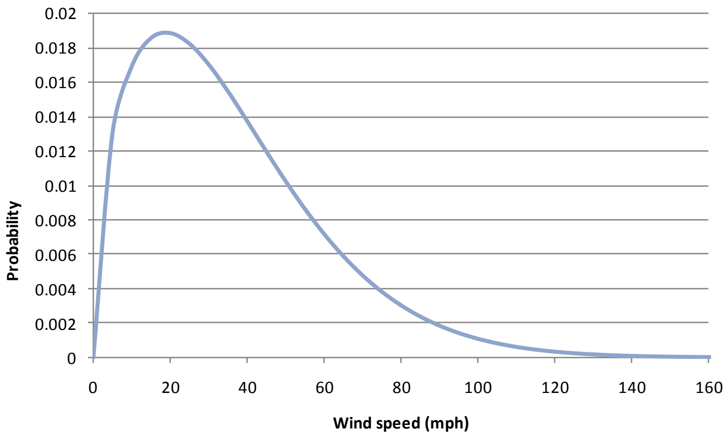


Figure 2.11. Probability distribution of wind speeds at Structure ID 154403

In this study, the wind speed will not be simulated. Rather the interest is in estimating future probability of hurricane occurrence. Regarding timed-based predictions, it can be assumed that exponential distributions will reasonably fit the time between occurrences of hurricane category winds at each location. The number of hurricane storms arriving at a specific coastal location in one year can be assumed to follow the Poisson distribution (Kobayashi et al., 2003; Le and Brown 2008), with Kriebel (1982) indicating from hurricane data, an example of λ, the average number of storms per year being 0.7 at Panama City, Florida. Le and Brown (2008) reported that the average annual hurricane occurrence varied from about 0.04 in the northeastern part of Florida to about 0.3 in the northwest region.

Considering Florida in terms of four regions: northwest, southwest, southeast, and northeast, the hurricane occurrences in these regions can be quite different. Based on land falling hurricanes during the period from 1851 to 2005, Le and Brown (2008) reported the occurrences in the Florida regions as shown in Table 2.6. It can be seen that the northwest region of Florida has experienced the most while northeast region has experienced the fewest, with southwest and southeast experiencing about the same and more likely to see categories 4 and 5 storms than any other region.

Table 2.6. Hurricane occurrence in different regions of Florida (Le and Brown 2008)

Region	Northwest	Southwest	Southeast	Northeast	Total
Category 1	20	12	8	3	43
Category 2	13	7	9	3	32
Category 3	13	6	10	0	29
Category 4	0	3	3	0	6
Category 5	0	1	1	0	2
Total	46	29	31	6	112

Based on the previous studies (Kobayashi et al., 2003; Le and Brown 2008;Kriebel 1982, and Le and Brown 2008) just described in the last two paragraphs, the number of storms arriving at a location in one year can be assumed to follow the Poisson distribution defined as

$$P_n = \frac{\lambda^n \exp(-\lambda)}{n!} \qquad 2.3$$

where P_n = the probability of having n storms per year, and
λ = mean rate or an average number of storms per year.

The recurrence time or time between two consecutive occurrences of an event in a Poisson process can be expressed in a probability distribution of function of T, such that

$$F_T(t) = P(T \leq t) = 1 - \exp\left[-\lambda t\right] \qquad 2.4$$

Where λ = mean occurrence rate as defined earlier, and the mean recurrence time, is referred to as the return period = 1/ λ.

From published HAZUS hurricane maps, the return times (years) are provided for various hurricane wind speeds. By classifying these speeds into hurricane categories (according to the Saffir-Simpson scale) at various locations, the return periods can be assigned to each specific hurricane category at each bridge location. Using the return period, the exponential distributions can be developed as briefly explained above. The probability of having a designated hurricane category wind within a specified period of time can therefore be estimated using equation 2.4.

As an example, considering the Structure ID 154403 from Table 2.5, the average annual rate λ for hurricane categories 1, 2, 3, and 4 are estimated as 0.05, 0.02. 0.01 and 0.002 respectively. Using equation 2.4, the probability of the bridge experiencing a hurricane of category 1 within the next one year is estimated as

$F_T(1) = P(T \leq 1) = 1 - \exp\left[-0.05 * 1\right]$ or 0.04877.

Similarly, the probabilities for hurricane categories 2, 3, and 4 within one year at this bridge can be shown to be equal to 0.0198, 0.00995, and 0.00200 respectively, while category 5 has zero probability of occurrence. If desired for long-term planning, the probabilities of hurricanes within next 10 years can also be estimated, for example, for category 1,

$F_T(10) = P(T \leq 10) = 1 - \exp\left[-0.05 * 10\right]$ or 0.3935.

Similarly, the probabilities for hurricane categories 2, 3, and 4 within 10 years at this bridge can be shown to be equal to 0.1813, 0.0952, and 0.0198 respectively, while category 5 has zero probability of occurrence.

Estimates of the likelihoods of hurricane occurrence near Florida bridges are presented in more details in Appendix A1 but summarized here for hurricane categories in Figure 2.12. The mean annual probabilities of hazard events decrease with increase in the hurricane intensity (category).

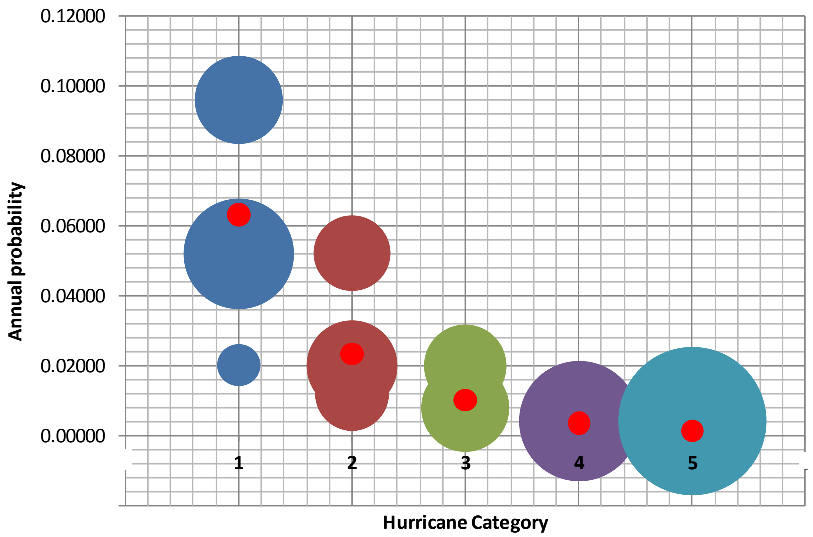


Figure 2.12. Annual probabilities of hurricanes (bubble plot) at Florida bridge locations

2.2.4. Hurricane models – special case of coastal bridges

The prediction model for hurricane occurrence at a bridge location has been presented earlier but of interest is also the effect of storm surge and wave loading that accompany hurricanes. This is a secondary effect of hurricane events that cannot be realistically predicted at coastal bridge locations. The closest to a prediction is to use the surge elevation data of coastal areas where these bridges are located. Sheppard and Miller (2003) developed design storm surge hydrographs for the Florida coast. This report listed recommended values for peak storm surge heights and corresponding likelihoods (50 year, 100 year, and 500 years occurrence) at various locations as summarized in Table 2.7, along with the GPS coordinates. Though the precision of the coordinates cannot be ascertained, it can be assumed reasonable. Using the latitude/longitude information from this list, a shape layer was developed in the GIS and overlaid on the Florida state-maintained coastal bridges. Shown in Figures 2.13 to 2.15 are the storm surge heights at selected coastal locations with the labels showing the 100 yr frequency estimates in feet. The bridges in the vicinity of these height locations can be easily identified.

Figure 2.16 shows (highlighted in lighter color) the coastal locations with estimated surge heights equal or greater than 13 feet. It should be noted that knowing the estimated storm surge height is not sufficient to predict the effects of surge-related hurricane events. These estimates have to be used in analyses to evaluate the adequacy of the available clearance below the lowest superstructure member of the bridge and the underlying water. In other words, information on storm surge heights can provide a rough knowledge on the expected intensity at certain bridge locations and it will be more useful in estimating the consequences and impacts of hurricanes, rather than in predicting their storm surge-related occurrence or magnitude.

A preliminary study on the vulnerability of coastal bridges to storm surges and wave loading is presented in Appendix A6.

Table 2.7. Estimated peak storm surge heights in Florida (Sheppard and Miller, 2003)

Ref No.	Location	Latitude (deg N)	Longitude (deg W)	Peak Storm Surge Heights (ft, NGVD) 50-yr	100-yr	500-yr
101	Escambia W, Esc.	30.28	87.52	9.8	11.4	15.3
102	Pensacola Bay, Esc.	30.32	87.27	9.7	11.0	14.3
103	Pensacola Bch, Esc.	30.35	87.07	9.4	10.8	13.9
104	Eglin AFB, Esc.	30.38	86.87	9.2	10.7	13.8
301	Eglin AFB, Oka.	30.40	86.63	9.9	11.2	13.0
302	Destin W, Oka.	30.39	86.60	10.2	11.4	13.7
303	Destin E, Oka.	30.38	86.40	10.2	11.4	13.7
401	Miramar Bch, Wal.	30.37	86.35	9.8	11.4	14.2
402	Grayton Bch, Wal.	30.33	86.16	9.4	11.2	14.1
403	Inlet Bch, Wal.	30.29	86.05	8.9	10.5	13.6
501	Hollywood Bch, Bay	30.27	85.99	10.6	11.9	14.8
502	Panama City, Bay	30.10	85.69	11.0	12.2	15.1
503	Mexico Bch, Bay	29.93	85.39	10.7	12.0	15.0
600	Beacon Hill, Gulf	29.92	85.38	10.1	11.7	16.3
601	St Joseph Pt, Gulf	29.85	85.41	8.0	9.3	12.6
602	St Joseph Park, Gulf	29.76	85.40	7.7	8.8	11.9
603	Cape San Blas, Gulf	29.68	85.37	9.3	11.1	16.0
604	McNeils, Gulf	29.68	85.30	10.8	12.4	16.9
605	Indian Pass, Gulf	29.68	85.25	11.0	12.6	17.0
701	St Vincent Is, Fra.	29.59	85.05	10.1	12.0	14.7
702	West Pass, Fra.	29.63	84.93	10.2	12.1	15.1
703	Sikes Cut, Fra.	29.68	84.81	10.2	12.3	15.4
704	St George Is, Fra.	29.75	84.71	10.5	12.6	16.0
705	Dog Is, Fra.	29.80	84.59	11.5	13.0	16.4
706	Alligator Hbr, Fra.	29.90	84.35	12.2	14.7	18.7
801	Lighthouse Pt, Wak.	29.93	84.29	13.1	14.7	17.3
802	Shell Pt, Wak.	29.96	84.23	13.3	15.1	17.3
803	Goose Creek Bay, Wak.	30.00	84.17	13.5	15.3	17.8
804	Whale Is, Wak.	30.03	84.12	13.9	15.3	18.1
805	Palmetto Is, Wak.	30.07	84.06	14.2	15.5	18.3
806	Little Redfish Pt, Wak.	30.10	84.00	13.9	15.2	17.9
1001	Stake Pt, Tay.	30.00	83.80	13.6	14.9	17.7
1002	Deadman Bay, Tay.	29.60	83.50	13.4	14.6	17.5
1101	Horseshoe Bch, Dix.	29.40	83.25	13.1	14.3	17.2
1102	Suwannee River, Dix.	29.30	83.10	12.8	14.0	17.0
1201	Cedar Key, Levy	29.15	83.00	12.6	13.7	15.5
1202	Waccasassa River, Levy	29.15	82.83	12.3	13.3	16.0
1301	Crystal River, Cit.	28.88	82.64	12.0	13.0	15.9
1302	Homasassa Bay, Cit.	28.75	82.64	11.7	12.7	15.8

Table 2.7. Estimated peak storm surge heights in Florida (Sheppard and Miller, 2003) Cont'd)

Ref No.	Location	Latitude (deg N)	Longitude (deg W)	Peak Storm Surge Heights (ft, NGVD) 50-yr	100-yr	500-yr
1303	Chassahowitzka Bay, Cit.	28.65	82.64	11.5	12.4	15.6
1401	Little Pine Is Bay, Her.	28.50	82.64	11.2	12.1	15.5
1501	Port Richey, Pas.	28.25	82.75	10.9	11.8	15.4
1601	Anclote River, Pin.	28.08	82.83	9.5	11.5	15.3
1602	Hurricane Pass, Pin.	27.89	82.85	8.5	10.1	13.4
1603	St Pete Bch, Pin.	27.73	82.74	9.9	11.5	14.7
1604	Bunces Pass, Pin.	27.62	82.72	8.5	9.9	13.1
1701	Tampa Bay, Man.	27.54	82.74	11.0	12.3	15.0
1702	Bradenton Bch, Man.	27.46	82.70	11.1	12.5	15.0
1703	Longboat Key, Man.	27.39	82.64	11.3	12.8	15.7
1801	Longboat Key, Sar.	27.38	82.64	11.4	12.9	16.0
1802	Venice Inlet, Sar.	27.17	82.49	11.3	12.6	15.6
1803	Manasota, Sar.	26.95	82.38	11.7	13.1	15.5
1901	Manasota, Cha	26.95	82.38	11.7	13.1	15.5
1902	Don Pedro Is, Char.	26.89	82.33	11.5	12.9	15.0
1903	Gasparilla Pass, Char.	26.81	82.28	11.4	12.7	15.0
2001	Gasparilla Is, Lee	26.79	82.27	10.7	12.5	15.4
2002	Captiva Pass, Lee	26.65	82.25	10.6	12.2	14.7
2003	Captiva, Lee	26.52	82.19	10.6	12.2	14.9
2004	Sanibel Is, Lee	26.42	82.09	11.6	13.4	16.2
2005	Ft Myers Bch, Lee	26.43	81.91	12.9	14.8	17.4
2006	Bonita Bch, Lee	26.34	81.85	12.9	14.7	17.9
2101	Wiggins Pass, Col.	26.32	81.84	13.1	15.2	18.9
2102	Doctors Pass, Col.	26.19	81.82	12.2	14.1	17.5
2103	Keewaydin Is, Col.	26.06	81.79	11.5	13.1	16.3
2104	Naples, Col.	25.92	81.73	11.5	12.9	15.1
2201	Highland Pt., Mon.	25.50	81.20	11.6	13.0	15.5
2202	Shark Pt, Mon.	25.30	81.20	11.7	13.2	15.8
2203	Key West, Mon.	24.70	81.40	11.7	13.3	16.2
2204	Big Pine Key, Mon.	24.80	80.80	11.8	13.5	16.5
2205	Long Key, Mon.	25.10	80.40	11.9	13.6	16.9
2206	Key Largo, Mon.	25.25	80.30	12.0	13.7	17.3
2207	N. Key Largo, Mon.	25.10	80.40	12.1	13.9	17.6
2301	Key Biscayne, Dade	25.68	80.16	12.1	14.0	18.0
2302	Miami Bch, Dade	25.83	80.12	10.8	13.6	17.7
2303	Bakers Haulover, Dade	25.95	80.12	11.4	13.5	17.6
2401	Hollywood, Bro.	26.03	80.11	11.4	13.6	16.9

Table 2.7. Estimated peak storm surge heights in Florida (Sheppard and Miller, 2003) (Cont'd)

Ref No.	Location	Latitude (deg N)	Longitude (deg W)	Peak Storm Surge Heights (ft, NGVD)		
				50-yr	100-yr	500-yr
2402	Ft Lauderdale, Bro.	26.06	80.11	11.2	13.1	17.2
2403	Pompano Bch, Bro.	26.22	80.09	10.9	12.5	17.1
2501	Boca Raton, Palm.	26.33	80.07	9.9	11.6	14.6
2502	Boynton Inlet, Palm.	26.53	80.05	9.9	11.5	15.0
2503	Lake Worth Inlet, Palm.	26.76	80.04	9.7	11.1	15.0
2504	Jupiter Inlet, Palm.	26.96	80.08	9.8	11.2	15.4
2601	Blowing Rocks, Mar.	27.01	80.09	10.3	11.2	12.6
2602	St. Lucie Inlet, Mar.	27.15	80.15	10.8	11.6	13.0
2603	Jensen Bch, Mar.	27.26	80.20	11.1	11.9	13.5
2701	Jensen Bch Park, StL.	27.27	80.20	10.4	11.4	13.3
2702	Ft Pierce Inlet S, StL.	27.42	80.27	10.8	12.1	13.9
2703	Ft Pierce Inlet N, StL.	27.54	80.32	11.1	12.3	14.7
2801	Vero Bch, Ind.	27.58	80.33	10.2	11.5	13.9
2802	Indian R Shores, Ind.	27.74	80.38	10.0	11.3	13.4
2803	Sebastian Inlet, Ind.	27.84	80.44	9.9	11.2	13.4
2901	Sebastian Bch, Bre.	27.91	80.47	10.2	11.6	14.2
2902	Satellite Bch, Bre.	28.18	80.59	9.8	11.1	13.7
2903	Cocoa Bch, Bre.	27.58	80.33	9.4	10.7	13.3
2904	Cape Canaveral, Bre.	28.50	80.50	9.4	10.9	14.0
2905	N Cape Canaveral, Bre.	28.80	80.65	9.5	11.0	14.7
3001	New Smyrne Bch, Vol.	28.88	80.79	9.5	11.2	15.4
3002	Daytona Bch, Vol.	29.15	80.97	8.8	10.6	15.8
3003	N. Penisula Rec., Vol.	29.43	81.10	9.2	11.3	15.7
3101	Flagler Bch, Flag.	29.44	81.10	8.7	10.7	15.2
3102	Painters Hill, Flag.	29.54	81.16	9.4	11.8	16.7
3103	Marineland, Flag.	29.67	81.21	9.8	12.6	18.3
3201	Matanzas Inlet, StJ.	29.70	81.22	9.2	12.3	16.3
3202	St. Augustine Inlet, StJ.	29.96	81.31	9.6	12.3	16.9
3203	Ponte Vedra Bch, StJ.	30.23	81.37	10.4	13.1	18.9
3301	Lake Duval, Duv.	30.26	81.38	10.5	13.2	17.8
3302	Manhattan Bch, Duv.	30.36	81.40	10.5	13.2	17.9
3303	Little Talbot Is, Duv.	30.48	81.41	10.6	13.1	17.8
3401	Nassau Sound, Nas.	30.54	81.44	11.1	13.2	18.8
3402	Fernandina Bch, Nas.	30.70	81.43	11.6	13.7	19.9
3403	St. Marys Ent., Nas.	30.71	81.43	11.9	13.9	20.2

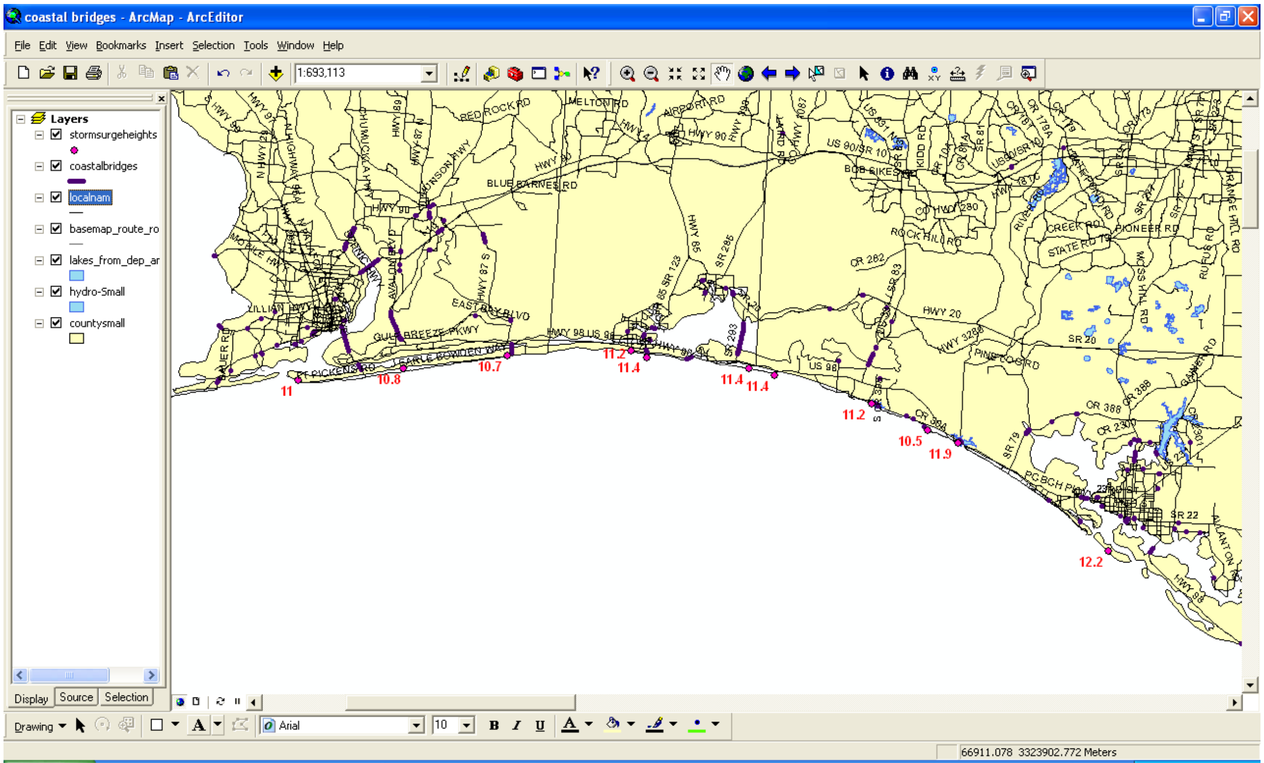


Figure 2.13. Storm surge heights in coastal areas of Escambia, Santa Rosa, Okaloosa, Walton, and Bay counties

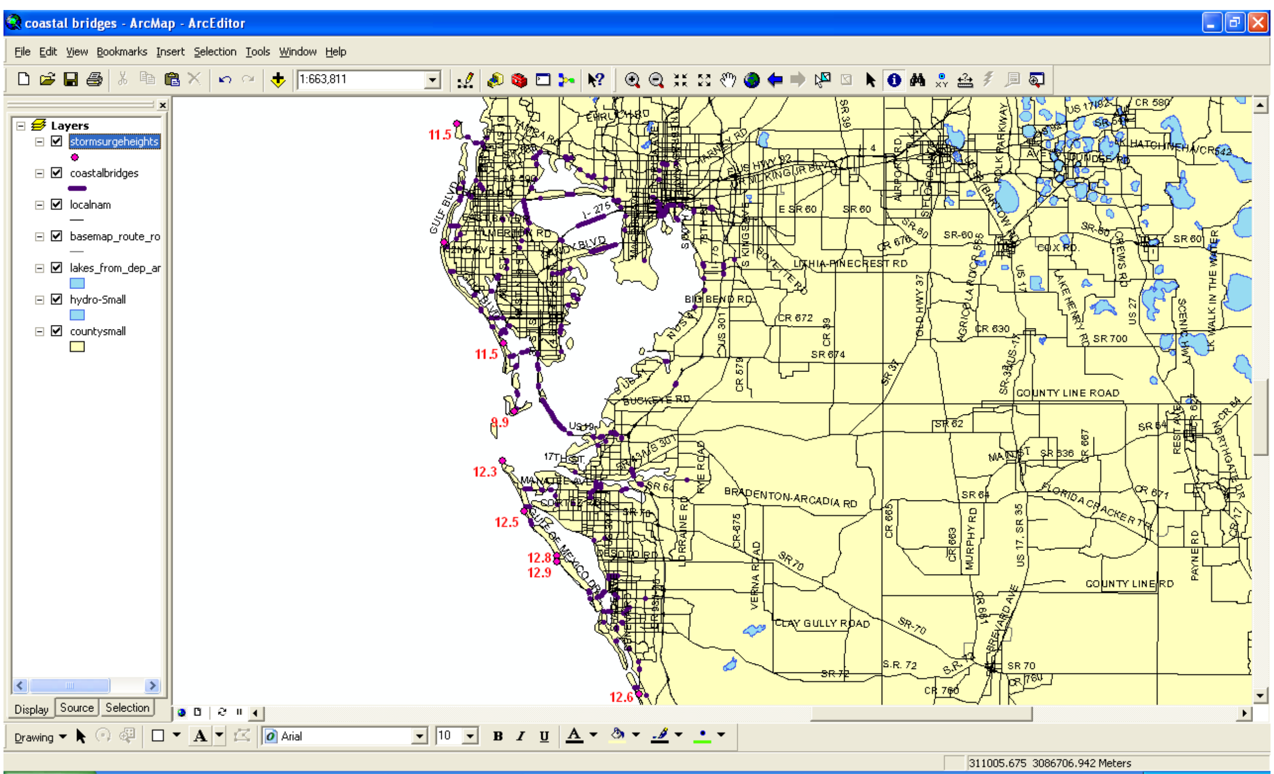


Figure 2.14. Storm surge heights in coastal areas of Pinellas, Hillsborough, Manatee, and Sarasota counties

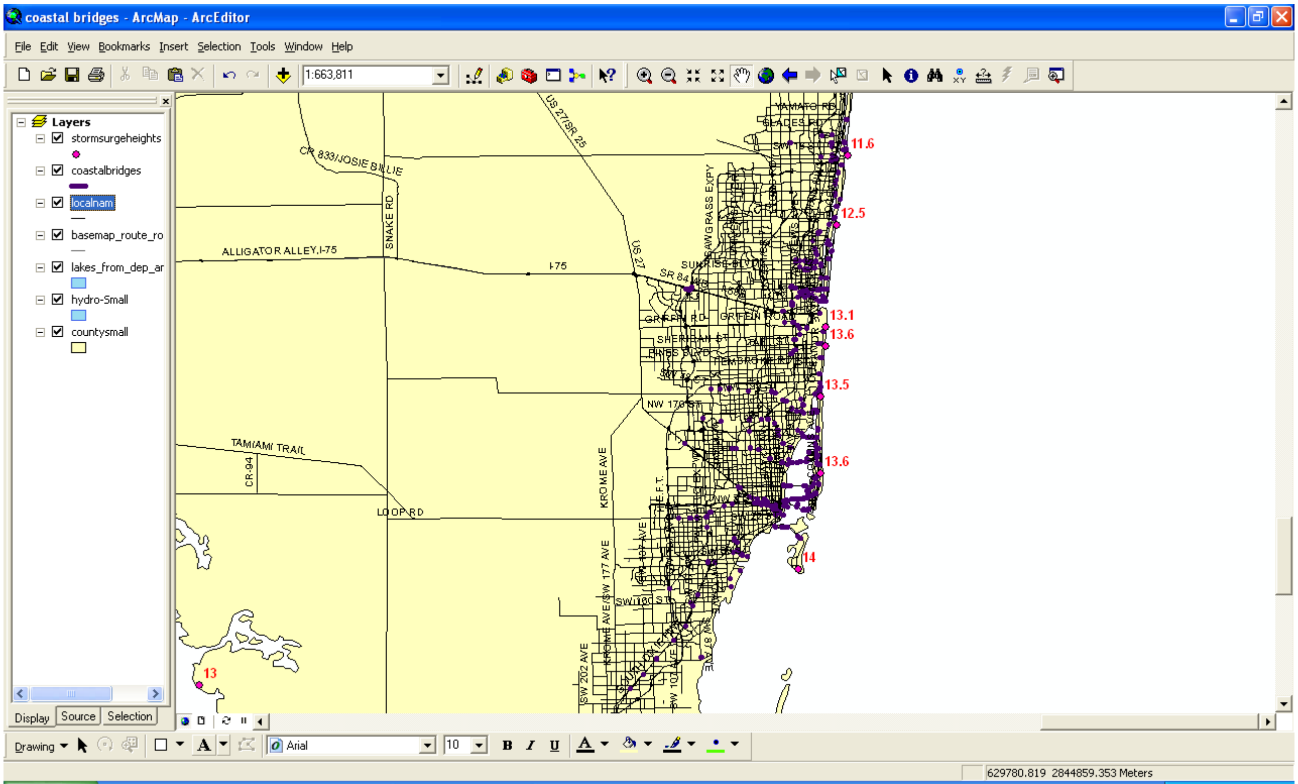


Figure 2.15. Storm surge heights in coastal areas of Miami-Dade, Broward, and Palm Beach counties

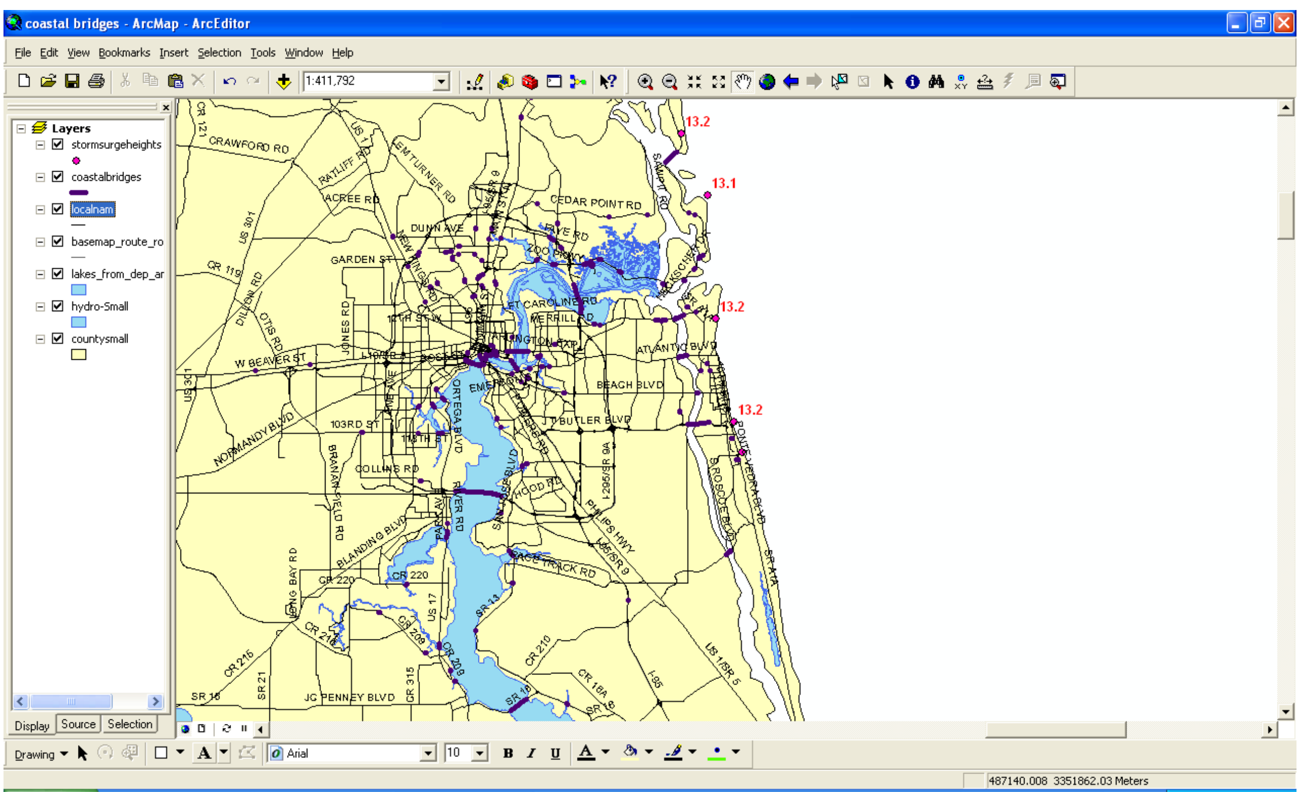


Figure 2.16. Storm surge heights in coastal areas of Nassau, Duval, and St. Johns counties

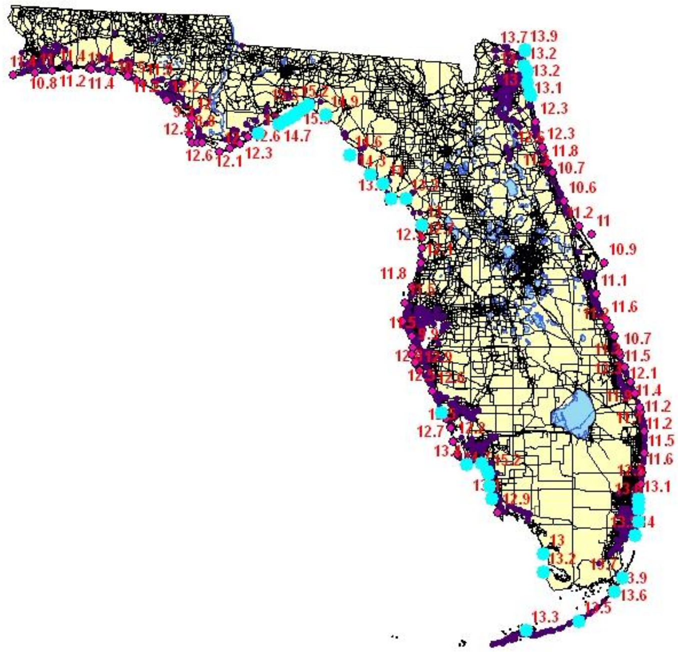


Figure 2.17. Coastal locations with estimated storm surge heights in Florida

2.3. Risk assessment: consequences of hurricane hazard

When hazards occur on bridges, the consequences consist of damage for which the agency is directly responsible (i.e., physical damage to the bridge, which will require repairs or replacement efforts) and other types of consequences related to public user costs (user delays, vehicle operations, and accidents). This section will address only the agency costs-related consequences. But first, it is important to note that the extent of the consequences or impact of hazards on bridges is dependent on attributes related to the bridge and its location. This can be referred to as the vulnerability or susceptibility of the bridge to damage from the particular hazard. For instance, bridges located on the coast are more vulnerable to damage from flooding, hurricane storm surge and wave loading than those bridges located inland, away from the coasts. This measure of vulnerability can be used to enhance the overall estimate of risk of the bridge to a particular hazard. In the examples given above, a bridge that is already very vulnerable will have a higher overall risk than another (less vulnerable) bridge that is exposed to the same likelihood estimate of occurrence of the particular hazard

2.3.1. Lessons learned from previous hurricanes

From the historical records reviewed, the year 2004 was the busiest in Florida for hurricanes, with four hurricanes making landfall in Florida (Figure 2.18). Damages to bridges and sign structures were observed in Florida as demonstrated in the two examples shown in Figures 2.19 and 2.20.

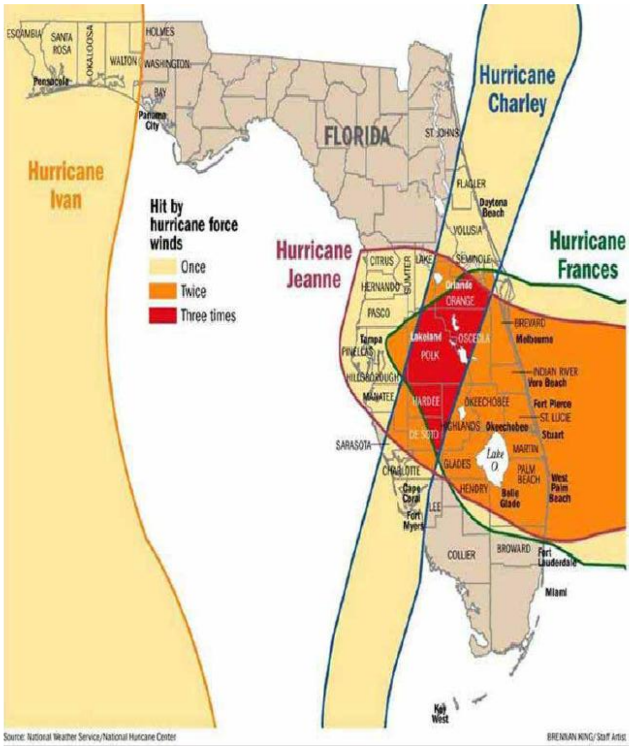


Figure 2.18. Hurricane experience for Florida in the year 2004 (Pavlov 2005)

Figure 2.19. Damage to bridge approach slabs in Florida during 2004 hurricane season (Pavlov 2005)

Figure 2.20. Damage to sign structures in Florida during the 2004 hurricane season (Pavlov 2005)

Maxey (2006) and Sheppard and Dompe (2006) described the damage to the I-10 Escambia bay bridge due to Hurricane Ivan, with. Maxey (2006) describing the damages as follows: Westbound bridge (Phase 1): 12 spans destroyed; 19 spans misaligned; and 7 bents replaced; Eastbound bridge (Phase 2): 51 spans destroyed; 33 spans misaligned; and 25 bents replaced. Some of the damages are illustrated in Figure 2.21.

After Hurricane Francis in 2004, according to Danielsen (2012), Mud Creek Bridge (ID 940005) suffered damages including 20 feet of scour under bridge, and the approach slab and bulkhead were undermined and destroyed (Figure 2.22). After hurricane Jeanne, slope failures were observed around end bents at three bridges (E. Lyons Bridge; Lake Worth Bridge; and Roosevelt Bascule Bridge) with example shown in Figure 4.3.

Figure 2.21. Damage to the decks, superstructure, and substructures on I-10 Escambia Bay Bridge after Hurricane Ivan (Maxey 2006)

Figure 2.22. Damage to the Mud Creek Bridge after Hurricane Frances (Danielsen 2012)

Figure 2.23. Slope failures at bridge bents after Hurricane Jeanne (Danielsen 2012)

As discussed in the literature review sections of this report, some lessons were also learned from other hurricane events that occurred outside Florida. These are presented in the following paragraphs.

According to Stearns and Padgett (2011), Hurricane Ike landed on September 13, 2008 in Texas, causing severe damage to the infrastructure in Houston/Galveston area. Data were collected for timber and major bridges from post-assessment surveys and reconnaissance reports, and a study of the failure modes of structures. Generally, the damage is described as being due to the following processes: inundation of bridge decks and superstructures; debris impact; erosion of abutment support; and erosion of approaches. Most severely damaged were timber bridges with low clearance over water. Of the 53 damaged bridges evaluated, 26 were observed to be damaged due to storm surge and wave loading, 25 bridges experienced scour around abutments and wing walls, while impact damage from debris affected four bridges. Peak storm surge level was found to be greater than 14 feet and wave heights more than 5 feet at most damaged locations. It was also observed that 17 rural bridges located inland, away from the surge zones, also experienced damage, primarily in the form of scour and impact damage. This is due to increased water flow rates and flooding.

For the timber bridges, various modes of failure were observed. Scour led to degradation of riprap on the abutments, undermining of the bridge approaches, and damage to areas behind the wing walls. Debris impact damages were seen on guardrails while tree trucks also impacted bridge piers and bents. Storm surge and wave loading caused shifting and complete unseating of bridge decks. One of the failures observed on the major bridges was loss of connection between the superstructure and substructure due to storm surge and wave loading, accompanied by failure of post-tensioning cable between girders. It should be noted though that the cables were already corroded before the hurricane. The adjacent roadway was also undermined and swept away. A major bridge, consisting of concrete slab and movable sections, was observed to be damaged at the approaches, fender (near bascule), and with severe erosion around the support columns. The damages are briefly summarized and categorized in Table 2.8, and levels of damage explained in Table 2.9.

Table 2.8. Reported bridge damages from Hurricane Ike

Bridge Type		Level of Damage #			
Subsctructure	Superstructure	Destroyed	Heavy	Medium	Light
Timber	Timber	Deck unseated (24)	Scour (8), Scour and Impact (2)	Scour (1), Scour and Impact (1)	
Timber	Steel		Scour (2), Scour and Impact (1)		Impact (1)
Timber and Steel	Timber			Impact (1)	
Concrete, Steel, and Timber	Steel and Concrete		Deck displaced (1)		
Steel	Steel		Scour (2)		
Concrete	Concrete	Deck unseated (1)	Scour (3)	Scour (1), Scour and Electrical (1)	Scour (2), Spalling (1), Scour and spalling (1)

No. of bridges indicated in parenthesis (e.g., 24)

Table 2.9. Levels of bridge damages from Hurricane Ike

Damage State	Description
Light	Some repairable damages to the superstructures. No immediate danger.
Medium	Minor damage to the superstrucure and possibly to the bridge. Possible loss of structural integrity.
Heavy	Major damage to entire bridge structure. Severe loss of structural integrity, posing public danger.
Destroyed	Bridge structure unusable or missing.

Potential risk mitigation measures include the following: replacing timber bridges with concrete bridge structures that have increased capacity and higher clearance; use of grated decks; use of air vents in bridge diaphragms; improved connectivity or vertical restraint elements between superstrucure and substrucure; use of transverse/shear keys to prevent lateral shifting; continuous span designs; increased span elevation; and erosion mitigation at approaches and abutments. In case of applying shear keys or restrianing cables to existing bridges, the design of the susbstructure has to be verified to carry the applied loads or upgraded as necessary. One suggested method of upgrade is to use fuses in new connection such that the connection would fail in a moderate event, with the deck and superstructure washed away but saving the substructure for reuse. Bridge approaches, abutments, and foundations should be evaluated for vulnerability to damage from erosion and scour due to flooding. Adequate mitigation may be needed in the form of riprap, wingwalls, or other protective measures.

For coastal bridges with low clearance over water, the lessons learned are as follows:

- *Timber* bridges are vulnerable to complete destruction from storm surge and wave loading, with the main mode of failure being deck unseating.
- *Timber* bridges in *coastal vicinity (inland)* are vulnerable to medium and heavy levels of damage from *scour and impact*.
- Bridges with *steel and timber members* are subject to *scour and impact* damage from light to heavy levels of damage.

- Bridges with *concrete, steel and timber members* are subject to heavy damage involving *deck displacement*.
- *Steel* bridges may suffer heavy damage from *scour*.
- *Concrete* bridges are vulnearable to complete damage from storm surge and wave loading, with the mode of failure being *deck unseating*. They are also subject to light to heavy damage from *scour*, medium damage to *electrical* components, and light damage in the form of *spalling*.

From a similar report on Hurricane Katrina by Padgett et al. (2008), described earlier, the following information can be summarized:

- Storm surge was severe in the coastal areas, with most damage done to superstructures due to unseating of individual spans, particularly low-elevation spans.
  - deck displacement
  - span shift without complete loss of support, but leading to pounding and damage to abutments, bent caps, and girders
  - bearing damage along with deck displacement and span shifting
  - loss of or damage to parapets on the bridge decks due to a combination of the storm surge and wind loading
- Scour damage
  - scour and erosion of abutment
  - slope failure
  - undermining of bridge approach.
- Impact damage from barges, oil drilling platforms, tug boats, and other debris.
  - span misalignment
  - damage to fascia girders (spalling of concrete and breaking of prestressing strands)
  - damage to fenders
  - pile damage.
- Inundation of electrical and mechanical equipment, primarily affecting movable bridges.
  - damage on submerged electrical and mechanical equipment (not designed for extended wetting or immersion in rushing flood water)
  - debris accumulation affecting functioning of mechanical gears
  - bent pivots
  - fractured mechanical parts
  - damage to traffic control gates
  - damage to lift motors and electrical systems, disabling the bridge. It was stated that 80% of emergency repair costs on a particular bridge were for replacement of the electronics damaged by water inundation.
- High winds from the hurricane may increase the potential for impact and debris, facilitating larger surges, waves, and horizontal pounding.
  - structural damage to operator houses and machinery housing in movable bridges
  - damage to the electrical cables on the towers of the movable bridge.
- The damaged spans were those with elevation at or below estimated peak storm surge levels.
- The types of bridges affected were traditional fixed bridges, with continuous spans, simply-supported spans, and movable bridges with fixed portions experiencing storm surges.

From this report by Padgett et al. (2008), the following criteria can be set and applied to Florida bridges:

- Extensive damage can result from impact to fixed and movable bridges, in this case, the moving object may be a barge. Railings/guardrails are suceptible to debris impact damage.
- Deck movements (from storm surge and wave loading) can cause moderate, extensive, or complete damage to fixed and movable bridges.
- Scour can cause moderate, extensive or complete damage to fixed and movable bridges.
- Winds are associated with slight, moderate, and extensive levels of damage in movable bridges.
- Damage to electrical and mechanical components accompany moderate, extensive and complete levels of damage in movable bridges.

Gilberto et al. (2007) reported that bridges were subjected to direct vertical and horizontal hydraulic forces, as well as being hit by large surge-borne debris such as vessels and large containers. Fast-moving water undermined bridge piers and caused flooding of mechanical and control rooms of moveable bridges. It was shown that provision of lateral support such as shear blocks prevented significant damage, as illustrated in Figure 2.24 where the railroad bridge piers with shear did not lose any spans across Biloxi Bay, but the nearby two parallel bridges (without lateral supports for spans) lost several spans.

a. b.

Figure 2.24. Bridges over Biloxi Bay: (a) two bridges severely damaged but parallel railroad bridge remained standing (b) lateral restraints (Gilberto et al., 2007).

Although a bridge was several miles inland, the impact from barges displaced the bridge superstructure about four feet to the north and tipped piers, causing a vertical drop of several inches. Spans of a particular bridge were pushed laterally off their bearings, evidently by a blockage of surge borne debris. There was also a case of a bridge suffering impact damage from an oil platform which broke loose from its mooring.

Also on movable bridges, the control and mechanical rooms were typically flooded, and the highly conductive and corrosive salt water damaged electrical and mechanical bridge controls, leaving them stuck either in the open or closed position.

Lessons learned from Gilberto et al. (2007) can be summarized as follows:

- Bridges with lateral restraints have a better chance of avoiding deck and superstructure damage (unseating or shifting) due to hurricane storm surge and wave loading.

- Impact damage, especially from heavy objects such as barges and loose oil platforms, can be severe enough to damage or displace the superstructure and tip the piers.
- Surge borne debris can also displace or shift superstructure spans
- Flooding on movable bridges may lead to highly conductive and corrosive salt water damaging electrical and mechanical bridge controls, leaving them stuck either in the open or closed position.

Robertson et al. (2007) reported that while Hurricane Katrina may have made landfall as a Category 3 hurricane in the Louisiana/Mississippi border, the storm surge and wave action developed while it was Category 5 in the Gulf of Mexico may not have lost strength rapidly as assumed. For bridges completely submerged in the storm surge, the superstructure's self weight was reduced, as well as development of buoyancy due to trapped air. This led to significant hydrostatic uplift forces. Buoyancy is greatly influenced by the deck/superstructure geometry (bulb tee girder, double tee girder, etc.) because of the capability to trap air. It was shown that immersion in seawater reduces the effective self-weight of concrete members from around 23.54 kN/m3 (150 lb/ ft3) to 13.49 kN/m3 (86 lb/ ft3), and flotation was generated on the spans from air trapped below the bridge deck or slab system, between the girders, transverse bridging, and end bulkheads.

For a particular bridge, the bearings supporting the girders at the pier bents provided no restraint against uplift and only nominal resistance against lateral movement (Figure 2.25). Being a low seismic zone, the region was not expected to make design provisions for lateral restraint or ties to prevent uplift. For some bridges, the friction induced by gravity load, and small steel angles, were the only physical restraint against lateral movement.

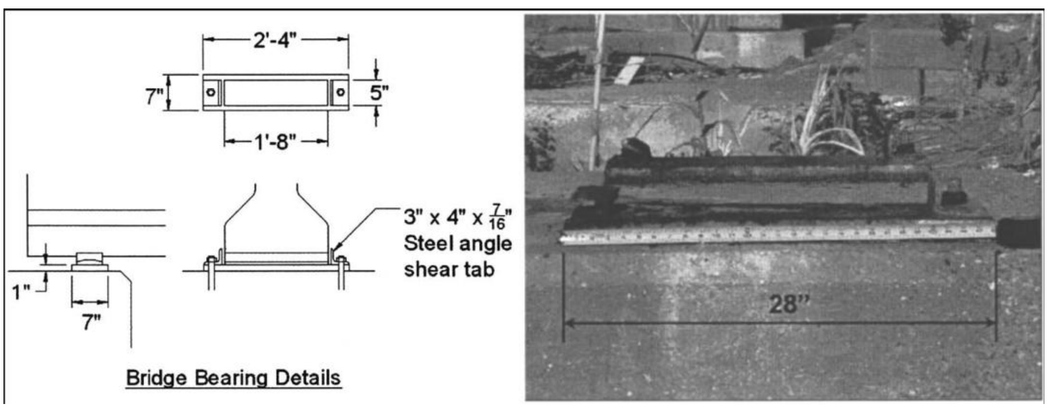


Figure 2.25. Bearing restraint details for US 90 Highway Bridge over Biloxi Bay (Gilberto et al., 2007).

A particular railroad bridge lost its entire railway's tracks, sleepers, and ballast, but the prestressed concrete bridge girders and deck remained intact (Figures 2.26 and 2.27). This performance was attributed to the reduced hydrodynamic uplift due to the small width of the bridge deck and the relatively small volume of entrapped air because of the closely spaced girders. The estimated wave-induced hydrodynamic uplift is estimated and the hydrostatic uplift (buoyancy) on the submerged deck was less than the deck self-weight. This bridge also had lateral restraint provided by concrete shear keys on either side of the girders at each support pier. This was adequate to resist the lateral hydrodynamic loads.

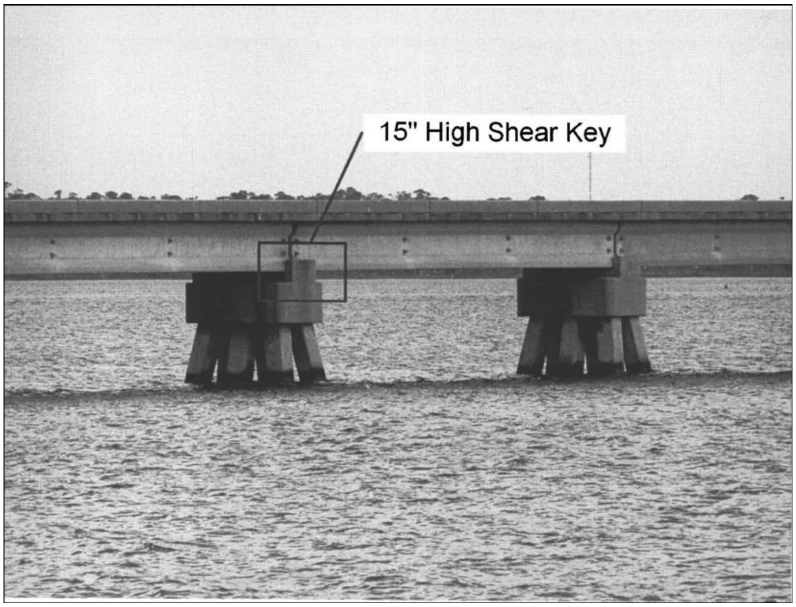


Figure 2.26. Railroad Bridge over Biloxi Bay with close-spaced girders and large concrete shear keys (Gilberto et al., 2007).

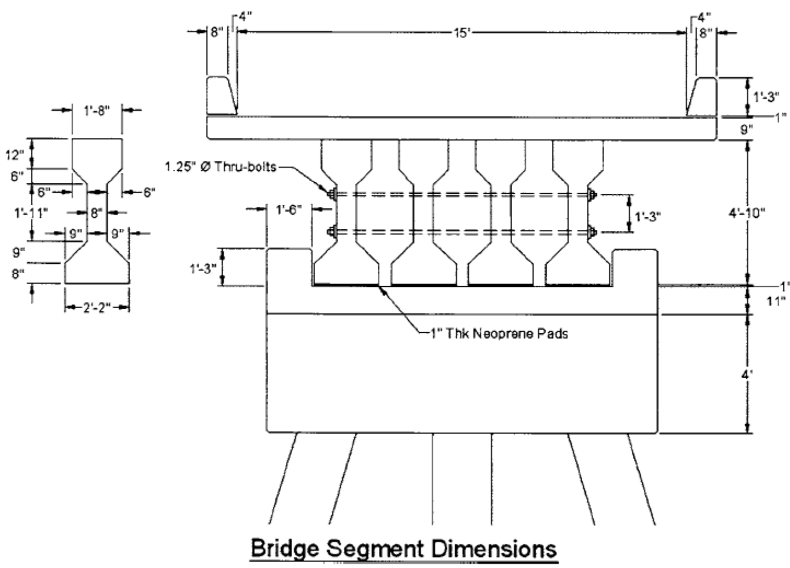


Figure 2.27. Cross-section details of Railroad Bridge over Biloxi Bay (Gilberto et al., 2007).

Extensive scour was observed around bridge abutments and piers, with two types of mechanisms identified: shear-induced scour due to pickup and transport of sediments by the flowing water and debris; and liquefaction-induced scour due to soil instability as a result of pore pressure gradients within the sediment bed. The latter occurs during a storm surge event, when the vertical effective stress between soil particles is reduced to nearly zero due to phase difference (i.e., time lag) between pore-pressure variation in the soil and water pressure variation on the bed surface. This mechanism is enhanced by the rapid drawdown as the surge water recedes. Scour occurs very rapidly under such conditions because the soil loses almost all of its shear strength and thus behaves like a viscous liquid, which can be transported easily by the flowing fluid. Wave-induced liquefaction of the soil may occur more in sandy soils. Provision of thin slope pavement may help by preventing shear-induced scour and postponing occurrence of the liquefaction. But the sandy soil is still susceptible to liquefaction-induced scour particularly during wave drawdown or ebb-surge.

There were signs of damage from debris impact, including those from floating casino barges, industrial barges, shipping containers, and 18-wheeler trucks. It was demonstrated that a 4,600 ton barge with a modest speed of 5 mph, and a 0.03 sec. impact duration will have an impulsive force acting on the structure of approximately 120,000 kips.

Water-damming by floating barriers was also identified as a source of damage to piers and columns, but demonstrated in Figure 2.28 for columns in a building. The process involves significant drag and inertia forces developed when large debris such as a shipping container becomes lodged between columns, creating disruption in the flow field.

Figure 2.28. Empty steel shipping container lodged against columns supporting roof (Gilberto et al., 2007)

Lessons learned and applicable to this FDOT bridge risk study include the following:

- For low-lying bridges, the superstructure can be lifted up by a combination of hydrostatic (from submersion) and hydrodynamic (from waves) forces and then displaced laterally by the hydrodynamic forces from surge and waves.
- The design type of bridge deck and superstructure will dictate the effects mentioned above (girders, deck slab, shear keys, etc.). Simply supported spans would be more vulnerable than continuous ones; Bulb tee beams will more vulnerable than deck slabs or box girders; existence of lateral restraints (shear keys) will improve resistance of the bridge superstructure.
- Using knowledge on the mechanism behind the scour process (shear-induced scour and wave-induced liquefaction of the soil), the extent of scour damage may be predictable. Presence of concrete slope pavement may slow down shear-induced scour while non-sandy soils will perform relatively better under the abutment and approach roadway. Backfills behind abutments and approach slabs should consider soil stabilization.
- Debris impact forces are very significant on the bridge substructure elements, particularly from large debris such as heavy barges, shipping containers, etc. Water-damming effects also introduce drag forces on bridge piers when the debris becomes lodged in between the piers. Bridges can be evaluated for the impact forces under the AASHTO Guide for Vessel Impact

Guide. Only coastal bridges and inlet bridges to be evaluated for the debris (barge) impacts. Span lengths of bridge piers may also be used as criterion to predict likelihood of debris damming, i.e., short spans are more susceptible.

2.3.2. Agency costs of damage

When hazards occur on bridges, the consequences consist of damage for which the agency is directly responsible (i.e., physical damage to the bridge, which will require repairs or replacement efforts) and other types of consequences related to public user costs (user delays, vehicle operations, and accidents)

This section is focused on determining the agency cost of repair or replacement. Data to support this analysis were obtained from Pontis and from a questionnaire survey distributed among FDOT's bridge engineers (DSMEs). In Pontis, the *inspevnt.insptype* field is used to identify inspections done after hazards such as vehicular or vessel accidents (code =”L”) and also after natural hazards such as hurricanes (code = “M”). A special district code (known as “central office bridges”) indicates bridge records that have been removed from the routine inspection process, usually because the bridge was retired or replaced. Table A5.1 in Appendix A5 shows a list of such bridges that have been retired or replaced due to extreme events, as identified in Pontis. The survey questionnaires served as a useful method of eliciting hard-to-find detailed information about the hazard events and their impacts on the structures. A sample completed questionnaire is shown in Figure A5.1 in Appendix A5.

In this study, we reviewed all described damages in the inspectors' comments in Pontis and in the survey results to develop a database of the impact of the hazard events on each structure identified. The pertinent information derived included the following: district location; date of hazard event (or inspection date); structure ID; brief description of hazard; bridge elements damaged; described costs of repair; estimated costs of repair; number of roadway lanes closed; duration of roadway closure; and other comments. Table A5.2 in Appendix A5 shows a summary of the results, but limited to those structures with cost or roadway closure duration estimates available. After an analysis of the damage data, a summary of the count of damages, by specific bridge element or bridge component type, is presented in Table 2.10.

A new field was added to the Pontis and survey results to reflect the level of damage on each structure. As background, a review was done on various existing criteria, especially FEMA's for hurricanes and earthquakes. There are no formal criteria for such classification for Florida bridges, and the only one reported in literature is an adaptation by Padgett et al. (2008) of the FEMA table for bridges damaged during hurricanes. A review of all the described damages due to floods and hurricanes on Florida structures was utilized to develop the scheme shown in Table 2.11, showing the criteria used in classifying damage experienced by Florida structures during the hazard events.

This set of criteria was then used to assign levels of damage observed on the structures in each event, as summarized in Figure 2.29 for bridges and in Figure 2.30 for sign structures. The intensity of the hurricane, as indicated by the category, did not appear to strongly influence the extent of damage. The records showed that except for the two bridges damaged in Florida during Hurricane Ivan, i.e., the two I-10 Escambia bridges, which were completely destroyed, other hurricane damages were mostly at moderate or slight levels. The sign structures showed mostly moderate damage under hurricane categories 1, 2, and 3. Hurricane category 4 caused mostly complete damage to the sign structures (about 50%) while moderate damage was seen on about 35%, and the remaining sign structures were equally divided between extensive and slight damage.

A more detailed analysis of the damage to bridge elements is presented in the next section of this report.

Table 2.10. Summary of structures by element types damaged during hazards in Florida

Element Type	No. of Structures Affected										
	Total for Each Element	Hurricane Katrina (Cat 1)	Hurricane Frances (Cat 2)	Hurricane Rita (Cat 2)	Hurricane Wilma (Cat 3)	Hurricane Dennis (Cat 3)	Hurricane Erin (Cat 3)	Hurricane Jeanne (Cat 3)	Hurricane Charley (Cat 4)	Hurricane/ Tropical Storm Fay	Average per Hurricane
Decks and Slabs	9	1	2		2			3	1		0.9
Columns and Piles	14	1	4		3			5	1		1.4
Pier Walls and Abutments	4	1			3						0.4
Substructures, incl. Caps and Footings	3		1					1	1		0.3
Culvert	3				2					1	0.3
Channel, Scour, incl. Pile Scour	219	2	28	2	131			42	9	4	21.8
Approach Slab	24	1	10	1	5	2	3		1	1	2.4
Railing	12	1	2		7			2			1.2
Fender and Dolphin	19		2		11			5	1		1.9
Abutment Slope Protection	53		7	6	11	4		21		4	5.3
Bulkhead and Seawall	5		3					2			0.5
Wingwall, Retaining Wall, and MSE Walls	13	1	8		2			2			1.3
Sign Horizontal Member (Elem 487)	263	36	7		205			8	7		26.3
Sign Vertical Member (Elem 488)	53	3	4		43			2	2		5.4
Sign Foundation (Elem 489)	135	3	1		116			3	12		13.5
Navigational Lights (Elem 580)	23		1		13			5	4		2.3
Operator Facility (Elem 581)	32		1		28			3			3.2
Warning Gates (Elem 591)	22	1			20			1			2.2
Traffic Signal (Elem 592)	22	1			19			2			2.2
Small signs ("weight limit", "Bridge Ahead", etc.)	45				42			3			4.5
Street Lights and Light Fixtures	17				16				1		1.7
Guardrails, Barrier and Handrails	2							2			0.2
Totals	992	52	81	9	679	6	3	112	40	10	

Note: Not included is the damage by Hurricane Ivan to two I-10 Escambia Bridges.

Table 2.11. Established scheme for classifying levels of damage to structures during hurricane hazards in Florida

	Levels of Damage			
Hazard Type	**Slight**	**Moderate**	**Extensive**	**Complete**
Hurricanes (Bridges)	Debris (tree logs, boats, etc.), insignificant scour, minor damage to channel, and damage to non-structural elements such as street lights, luminaires, lamps, mounted lights, small signs, and railings. Poses no serious structural problem. Structure may need minor repairs.	Washouts at embankments/approach slabs and damage to slope protection system. Overtopping due to flood (deck/slab or culvert damage) and significant scour. Moderate damages, including undermining, to abutments, columns, piles, caps, footings, channel, and bulkhead. Moderate damages to fenders, navigational lights, warning gates, traffic signals, operator facilities, electrical conduit, cables, PLCs, transformers, and equipment. Poses serious structural/functional problems. Structure is repairable.	Extensive damage to culvert, deck, superstructure, substructure, and pertinent bridge elements. Structure is repairable. Poses serious structural/functional problems. May require full replacement of structural component(s).	Severe damage to all or critical structural and non-structural components. Structure need to be completely replaced.
Hurricanes (Sign Structures)	Minor damages such as loss of sign panels, twisting of luminaires, etc. Poses no serious structural problem. Structure may need minor repairs.	Loss of horizontal members, and minor cracks on foundation. Moderate damage to horizontal, vertical members, or foundation. Poses serious structural/functional problems. Structure is repairable.	Extensive damage to panels, chords, trusses, and foundation. Poses serious structural/functional problems. Structure is repairable. May require full replacement of structural component(s).	Severe damage to all or critical structural components. Structure need to be completely replaced.

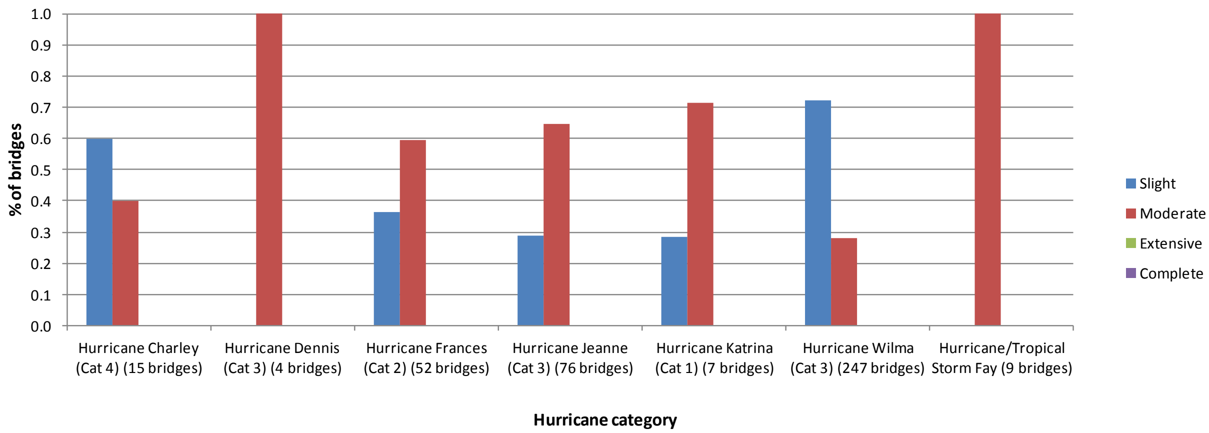


Figure 2.29. Levels of damage to bridges for Hurricane categories
*Note: Not included is the damage by Hurricane Ivan to two I-10 Escambia Bridges costing $243 million to repair.

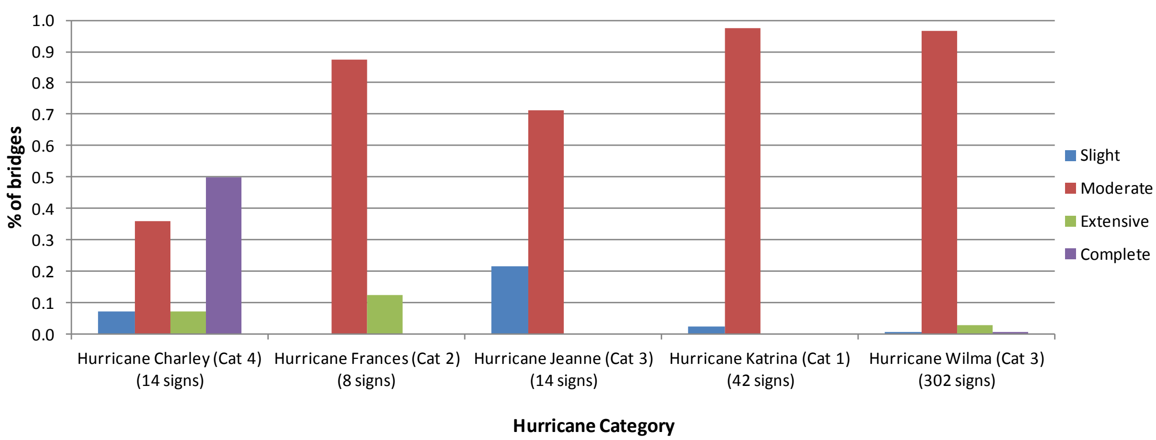


Figure 2.30. Levels of damage on sign structures for hurricane categories

2.3.3. Evaluation of element damage

In quantifying the damage done to the bridge elements by hurricanes and associated floods, the bridge inventory data for identified elements in the hazard event data in Table 2.11 were retrieved from the Pontis database. Specifically, the hazard event data (with event dates, etc.) were merged with *bridge*, *roadway*, and *inspevent* tables' data from Pontis. An estimate was made for each bridge record of the difference between hazard event dates and the inspection dates in the *inspevent* table. These date differences were used to separate inspections into those done before and after the hazard event. By sorting by bridge IDs and dates of inspection, the closest inspections to the hazard event dates were identified and used to create two tables, one for inspections done before the hazard event and the other for inspections after the hazard. The same process was then repeated, but using the Pontis *eleminsp* table instead of the *inspevent* table. This yielded more detailed element-based inspection data.

One of the elements evaluated was Element No. 396 Abutment slope protection. In the NBI condition data records, the closest rating to measuring conditions of this element (the abutment slope) is the NBI Item 61 Channel rating. By definition from FDOT (2011) .. "…this item reflects the overall physical condition associated with the flow of water through the bridge or culvert, including evaluation of stream stability and the condition of the channel, riprap, slope protection, etc. Particular concern is mentioned about signs of excessive water velocity which may affect undermining of slope protection or footings, erosion of banks and realignment of the stream which may result in immediate or potential problems. Thus this is a very good measure of scour damages…" A review was done of the data before and after the hazard events as explained above, using bridges identified as being damaged in their abutment slope protection. As shown in Figure 2.31, the changes in the NBI Item 61 Channel ratings for the affected bridges were observed to indicate that some rapid deterioration had occurred, though the specific change between condition ratings could not be ascertained. It appears the damages were done to bridges in condition ratings 7 and 8.

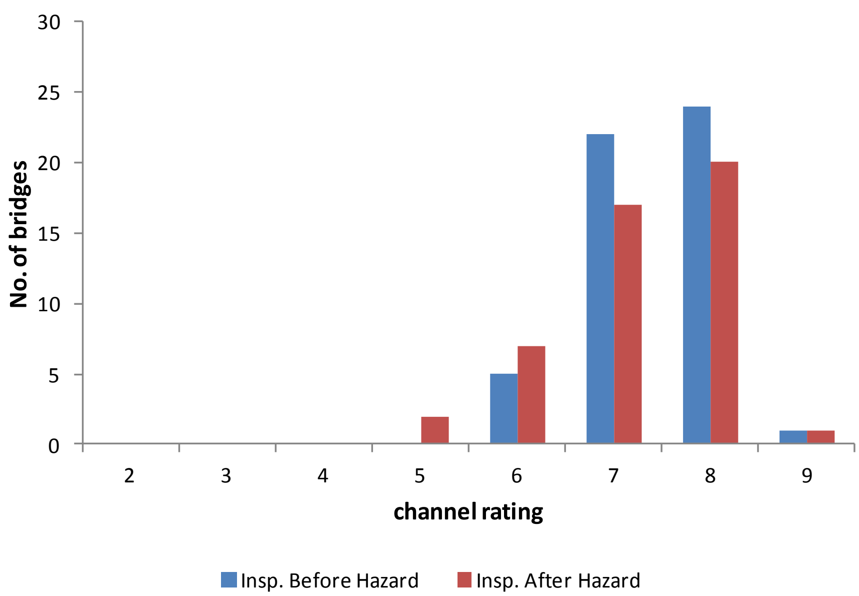


Figure 2.31. Comparison of bridge channel ratings before and after hurricane hazard events

The next effort was to use the element inspection data (*elemisnp)* as explained above, which gave more detailed inspection data. This time the bridges damaged at their abutment slope protection elements are directly identifiable in the records as Element No. 396. Specific hazard events were evaluated and the change in state condition data observed as shown in Table 2.12. Looking at the element quantity in each condition state, it could be seen that during Hurricane Dennis, the damage to the elements appears to be very negligible. Hurricanes Frances and Tropical Storm Fay showed slight damages while Hurricane Wilma showed more significant damages. Hurricanes Jeanne and Rita showed no damage, with data for Hurricane Jeanne actually showing addition of element quantity after the hurricane. Overall for the affected bridges, the damages for the abutment slope protection are portrayed in the last two rows of Table 2.12 in a quantified form of the changes in the condition states of the element. While the exact transitions cannot be ascertained directly, unless by looking at each bridge, it could be seen that some bridge elements lost element portions in the state 1 condition to worse condition states.

Table 2.12. Damage effects of hurricanes on Element No. 396 Abutment Slope Protection

Hurricane Name/	No. of elements	Inspection Before/ After Hazard	Element Quantity (SF)	% in State 1	Quantity in State 1	% in State 2	Quantity in State 2	% in State 3	Quantity in State 3	% in State 4	Quantity in State 4	% in State 5	Quantity in State 5
Hurricane Dennis													
	3	Before	3718.3	1.00	3718.3	0.00	0.0	0.00	0.0	0.00	0.0	0.00	0.0
	3	After	3718.3	1.00	3704.3	0.00	13.9	0.00	0.0	0.00	0.0	0.00	0.0
Hurricane Frances													
	7	Before	3709.9	0.69	2546.9	0.30	1105.0	0.02	58.0	0.00	0.0	0.00	0.0
	7	After	3709.9	0.63	2326.3	0.30	1100.4	0.06	206.6	0.02	76.6	0.00	0.0
Hurricane Jeanne#													
	14	Before	5476.8	0.79	4336.3	0.01	47.8	0.10	540.1	0.10	552.6	0.00	0.0
	14	After	5996.5	0.84	5066.5	0.00	29.9	0.06	353.7	0.09	546.4	0.00	0.0
Hurricane Rita													
	6	Before	4936.2	0.94	4654.1	0.03	160.1	0.02	94.1	0.01	27.9	0.00	0.0
	6	After	4936.2	0.95	4701.2	0.02	74.6	0.03	132.5	0.01	27.9	0.00	0.0
Hurricane Wilma													
	9	Before	3827.2	0.98	3739.5	0.00	17.3	0.02	70.4	0.00	0.0	0.00	0.0
	9	After	3827.2	0.79	3035.7	0.03	111.9	0.07	261.5	0.11	418.1	0.00	0.0
Hurricane/Tropical Storm Fay													
	14	Before	943.1	1.00	943.1	0.00	0.0	0.00	0.0	0.00	0.0	0.00	0.0
	14	After	943.1	0.99	936.7	0.00	2.4	0.00	3.9	0.00	0.0	0.00	0.0
All hazard:	53	Before	22611.4	88.18	19938.0	5.88	1330.2	3.37	762.6	2.57	580.5	0.00	0.0
	53	After	23131.1	85.47	19770.8	5.76	1333.1	4.14	958.2	4.62	1068.9	0.00	0.0

During hurricane Jeanne, three bridges show increase in quantity from before to after, in amounts of about 240, 260, and 16 units.

Another consequence observed on Florida bridges due to hurricanes and flooding was damage to channels in the form of scour. In this case a detailed evaluation of the element inspection data for the channel element (a Florida custom element) was conducted as done above for the abutment slope protection. The results are shown in Table 2.13, this time with additional variations by the environmental classification of the elements. Significant damage can be observed as having occurred to the elements during Hurricane Frances while Hurricanes Jeanne and Wilma suffered slight damages. Hurricanes Charley, Katrina and Rita did not show any damage to the channels. Overall there is a reasonable damage to the elements' condition, where there is an obvious reduction of element quantities from condition state 1 and increase in the quantities for the worse condition states.

Table 2.13. Damage effects of hurricanes on Element No. 290 Channel

Hurricane Name/ Environment	Inspection Before/ After Hazard	Element Quantity (EA)	% in State 1	Quantity in State 1	% in State 2	Quantity in State 2	% in State 3	Quantity in State 3	% in State 4	Quantity in State 4	% in State 5	Quantity in State 5
Hurricane Charley												
3	Before	3	33.3	1	66.7	2	0.0	0	0.0	0	0.0	0
	After	3	33.3	1	66.7	2	0.0	0	0.0	0	0.0	0
4	Before	6	33.3	2	50.0	3	0.0	0	16.7	1	0.0	0
	After	6	33.3	2	50.0	3	0.0	0	16.7	1	0.0	0
All environments	Before	9	33.3	3	55.6	5	0.0	0	11.1	1	0.0	0
	After	9	33.3	3	55.6	5	0.0	0	11.1	1	0.0	0
Hurricane Frances												
3	Before	16	68.8	11	31.3	5	0.0	0	0.0	0	0.0	0
	After	16	37.5	6	50.0	8	6.3	1	6.3	1	0.0	0
4	Before	12	83.3	10	16.7	2	0.0	0	0.0	0	0.0	0
	After	12	50.0	6	25.0	3	25.0	3	0.0	0	0.0	0
All environments	Before	28	75.0	21	25.0	7	0.0	0	0.0	0	0.0	0
	After	28	42.9	12	39.3	11	14.3	4	3.6	1	0.0	0
Hurricane Jeanne												
3	Before	13	15.4	2	76.9	10	7.7	1	0.0	0	0.0	0
	After	13	15.4	2	69.2	9	15.4	2	0.0	0	0.0	0
4	Before	26	61.5	16	23.1	6	7.7	2	7.7	2	0.0	0
	After	26	53.8	14	30.8	8	11.5	3	3.8	1	0.0	0
All environments	Before	39	46.2	18	41.0	16	7.7	3	5.1	2	0.0	0
	After	39	41.0	16	43.6	17	12.8	5	2.6	1	0.0	0
Hurricane Katrina												
4	Before	2	50.0	1	50.0	1	0.0	0	0.0	0	0.0	0
	After	2	50.0	1	50.0	1	0.0	0	0.0	0	0.0	0
Hurricane Rita												
4	Before	2	100.0	2	0.0	0	0.0	0	0.0	0	0.0	0
	After	2	100.0	2	0.0	0	0.0	0	0.0	0	0.0	0
Hurricane Wilma												
3	Before	36	77.8	28	22.2	8	0.0	0	0.0	0	0.0	0
	After	36	72.2	26	25.0	9	2.8	1	0.0	0	0.0	0
4	Before	96	68.8	66	28.1	27	3.1	3	0.0	0	0.0	0
	After	96	64.6	62	32.3	31	3.1	3	0.0	0	0.0	0
All environments	Before	132	71.2	94	26.5	35	2.3	3	0.0	0	0.0	0
	After	132	66.7	88	30.3	40	3.0	4	0.0	0	0.0	0
All hazards	Before	212	65.6	139	30.2	64	2.8	6	1.4	3	0.0	0
All environments	After	212	57.5	122	34.9	74	6.1	13	1.4	3	0.0	0

2.3.4. Costs of repair

The collected data on costs for repairs due hurricane damage are summarized in Table 2.14 for Florida bridges. The data size is somewhat small to make any strong statistical conclusions or inferences. The mean cost per bridge damaged during hurricanes does not correlate directly with the intensity or category of the hurricane. Looking at bridges damaged during Hurricane Wilma, in which there is a reasonable size of the dataset (33), it can be seen that it cost an average of about $21,000 to repair the damage, with a range of between $1,000 and $77,300. It should be noted that the data on the graphs does not include Hurricane Ivan (Cat 3) which destroyed two bridges (Escambia I-10 bridges ID 480213 and 480214) at a replacement cost of $243 million.

Table 2.14. Summary of bridge repair costs due to hurricane damage in Florida

Hurricane Category (No. of bridges affected)	Bridges	No. of Costs Available	Mean Cost ($)	Std. Dev. ($)	Min. Cost ($)	Max. Cost ($)
Hurricane Frances (Cat 2) (52 bridges)	52	2	405,606	557,757	11,212	800,000
Hurricane Rita (Cat 2) (6 bridges)	6	6	206,008	195,883	11,000	455,070
Hurricane Dennis (Cat 3) (4 bridges)	4	4	47,988	16,410	33,000	66,000
Hurricane Wilma (Cat 3) (247 bridges)	247	33	21,085	17,747	1,000	77,300
Tropical Storm Fay (9 bridges)	9	9	229,507	183,806	26,417	524,676

*Note: Not included is the damage by Hurricane Ivan to two I-10 Escambia Bridges costing $243 million to repair.

A further breakdown of the costs indicated that two bridges at moderate levels of damage during Hurricane Frances (Category 2) cost an average of $405,606 to repair, while six bridges with similar levels of damage during Hurricane Rita (Category 2) cost about $206,000 on average. During Hurricane Wilma (Category 3), three bridges with slight damages were observed to cost an average of $10,333 to repair while 30 bridges at moderate level of damage cost $22,160. For Hurricane Dennis, also category 3, the four bridges damaged at moderate levels, cost an average of $47,988 to repair. Tropical Storm Fay also caused moderate levels of damage to nine bridges, costing an average of $229,507 to repair. It should be noted again, that the two I-10 Escambia Bridges were extensively damaged during Hurricane Ivan and cost $243 m to repair. Though the Florida repair cost data does not have many bridges that were extensively or completely damaged, the costs for slight and moderate damages are comparable to the costs reported by Padgett et al. (2008) for damage to bridges during Hurricane Katrina in the states of Alabama, Louisiana, and Mississippi. The cost data from Padgett et al. (2008) was further analyzed and summarized in Table 2.15 and Figure 2.32. More detailed cost data on hurricane damages are provided in Appendix A5.

Table 2.15. Bridge repair costs at damage levels in Hurricane Katrina outside Florida (Padgett et al., 2008)

Damage level and bridge type	Mean Cost ($)	Min Cost ($)	Max Cost ($)	Std Dev ($)	Coeff. of variation	Count
Slight Damage#	2,778	1,000	9,000	894	0.322	9
Moderate Damage#	112,500	25,000	200,000	35,018	0.311	6
Moderate damage movable bridges*	146,429	25,000	350,000	45,023	0.307	7
Moderate damage fixed bridges	2,023,333	10,000	6,000,000	1,988,386	0.983	3
Extensive damage movable bridges	1,839,063	25,000	7,700,000	554,682	0.302	16
Extensive damage fixed bridges	2,110,000	500,000	5,800,000	1,237,646	0.587	4
Complete damage movable bridges	275,500,000	275,000,000	276,000,000	500,000	0.002	2
Complete damage fixed bridges	11,133,333	1,500,000	30,000,000	9,434,040	0.847	3

All are due to wind only and movable bridges in Louisiana; * All cases due to wind only except for one.

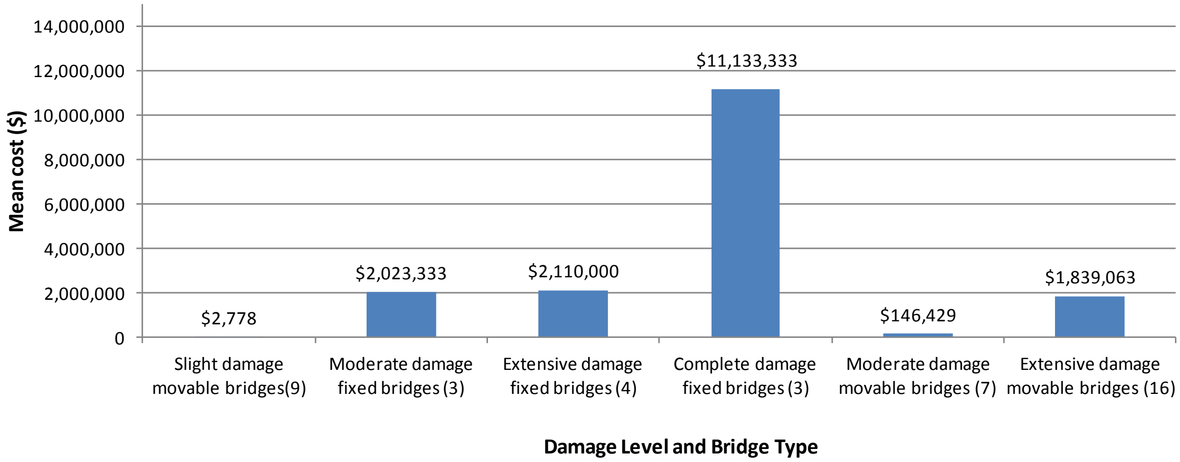


Figure 2.32. Mean costs of bridge repair or replacement due to Hurricane Katrina outside Florida (Not Showing two movable bridges with complete damage at $275.5 m) (Padgett et al., 2008)

Also investigated were the effects of age on the cost of damage suffered by bridge elements. Table 2.16 below shows average ages at which the elements suffered hurricane damage. Given the large standard deviation values in the results, it is difficult to make a distinct separation among the ages at which the bridge elements suffered damage during hurricanes. The approach slabs appear to be damaged in relatively older bridges of an average of 39 years, while the abutment slope protection and channel /pile (scour) appear to suffer damage at the age of about 32 years. Hurricane Wilma appeared to be an exception for the channel/pier scour where bridges were damaged at a relatively lower age of about 30 years. The variation in ages for each bridge element or deficiency is shown in Figure 2.33.

Table 2.16. Variation in age of bridge elements affected by hurricanes

Bridge Element/Deficiency		Mean	Standard Deviation	Median	Count
Channel, Scour, incl. Pile Scour					
	All Hurricane Events	32.1	14.1	31.0	222
	Hurricane Charley	37.7	16.5	34.0	9
	Hurricane Frances	39.8	18.1	41.0	28
	Hurricane Jeanne	33.6	14.7	33.5	42
	Hurricane Wilma	29.4	11.7	27.0	131
Abutment Slope Protection					
	All Hurricane Events	32.3	13.0	31.5	54
Approach Slab					
	All Hurricane Events	39.2	15.0	39.5	24

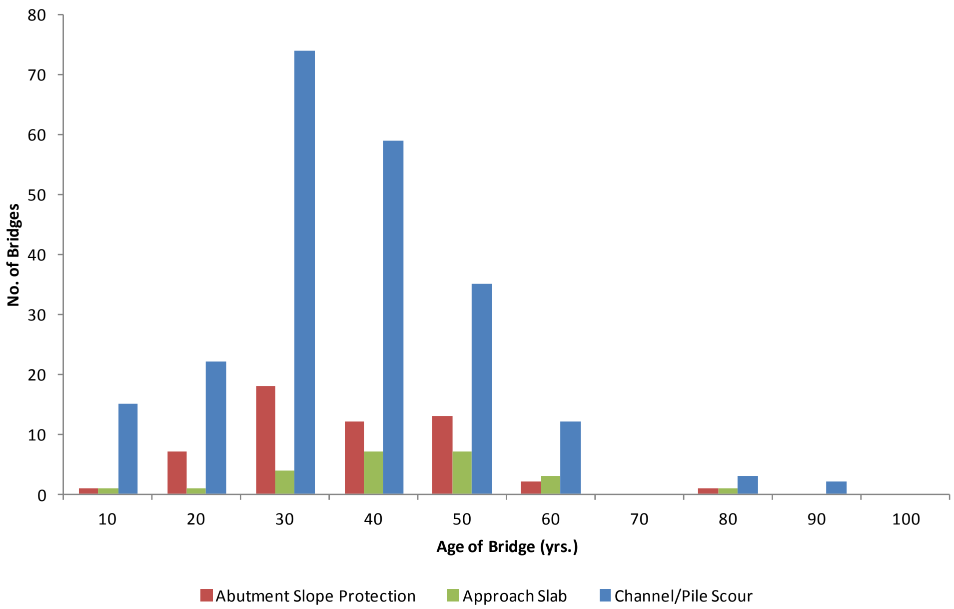


Figure 2.33. Variation in ages of Florida bridge elements affected by hurricanes

In terms of geographic location, most of the damage observed on the channel/Pile scour, abutment slope protection, and approach slab, occurred in Districts 1 and 4. These damages also occurred mostly on state-maintained bridges. Abutment slope protection is damaged more on prestressed concrete, multi-beam or multi-girder bridges than any other types. The approach slab seems to be predominantly damaged on slab types of bridges. These conclusions are based on the results summarized in Tables 2.17 to 2.19. The influence of bridge span length and count on element damage is explored in the histograms in Figures 2.34 and 2.35. The channel/pier scour damage appeared to have occurred on bridges with lower (maximum) span lengths, and low number of major spans. There are limitations to these conclusions given the small data set and also the need to compare the statistical pattern with that of the original bridge inventory not affected by hurricanes.

Table 2.17. No. of bridges by district location and bridge ownership of elements affected by hurricanes

	District Located							Owner Code			
Bridge Element/Deficiency	1	2	3	4	5	6	7	1	2	3	4
Channel, Scour, incl. Pile Scour	140	9	0	47	20	3	3	113	58	1	50
Abutment Slope Protection	13	2	0	21	6	10	2	47	4	2	1
Approach Slab	5	1	0	14	1	3	0	12	5	0	7

Table 2.18. No. of bridges by superstructure materials type for elements affected by hurricanes

	Materialmain Code					
Bridge Element/Deficiency	Concrete	Concrete Continuous	Steel	Steel Continuous	Prestressed Concrete	Prestressed Concrete Continuous
Abutment Slope Protection	7	2	10	0	34	1
Approach Slab	11	1	2	1	9	0

Table 2.19. No. of bridges by superstructure design type for elements affected by hurricanes

	DesignMain Code					
Bridge Element/Deficiency	Slab	Multi-beam or Multi-girder	Tee Beam, or Double Tee Beam	Movable - Bascule	Segmental Box Girder	Channel Beam
Abutment Slope Protection	8	39	2	3	2	0
Approach Slab	12	7	1	1	1	2

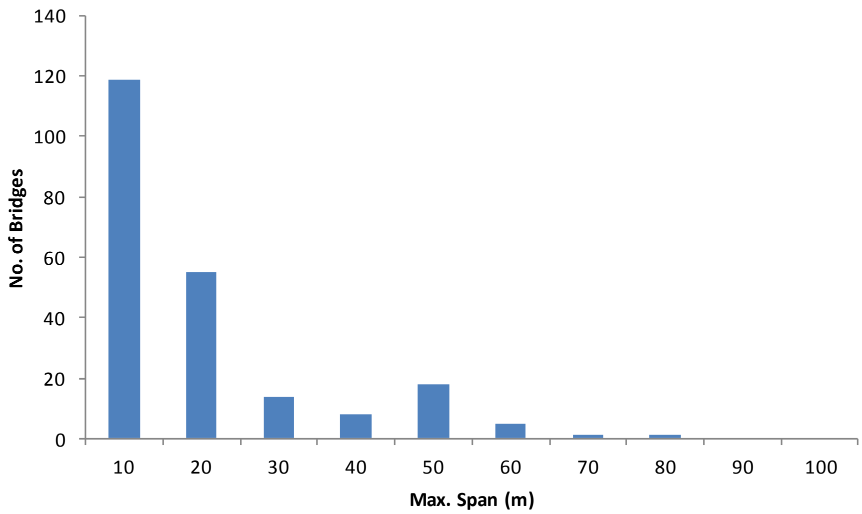


Figure 2.34. Variation in max. spans on Florida bridges with channel scour due to hurricanes

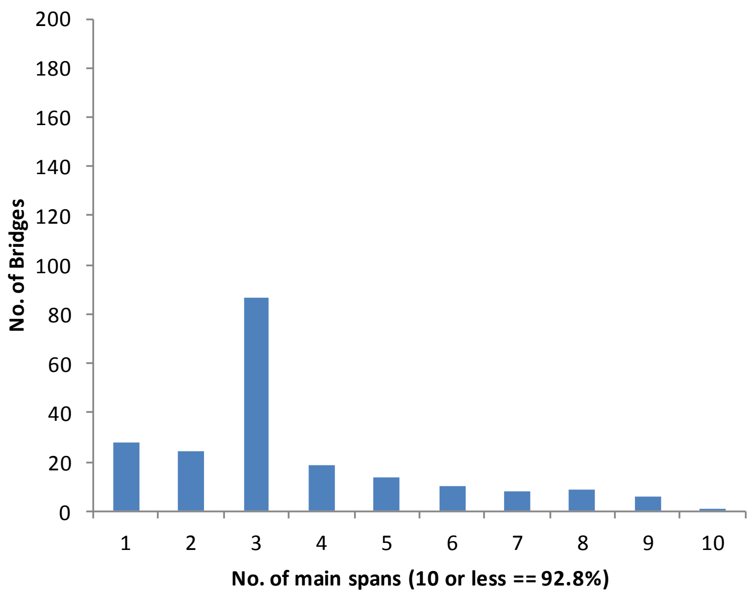


Figure 2.35. Variation in no. of main spans on Florida bridges with channel scour due to hurricanes

2.3.5. User costs associated with hurricanes

Apart from the agency costs to repair the damage to the bridge elements from the hurricane hazard, the public users also incur some costs from the inconvenience or unavailability of the bridge for use. These costs are termed the user costs. One of the variables necessary to estimate such costs is the duration of roadway closures on the bridge when the hazard events occur.

Overall, data on duration of hurricane-related roadway closures were limited; the available data are summarized in Table 2.20.

Table 2.20. Summary of bridge road closure durations due to hurricanes in Florida

			Bridge Closure (Hours)			
Hazard Type	**No. of Bridges**	**No. of Closures**	**Mean**	**Std. Dev.**	**Min.**	**Max.**
Hurricane Charley (Cat 4)	15					
Hurricane Dennis (Cat 3)	4	1	24		24	24
Hurricane Erin (Flood) (Cat 1)	1					
Hurricane Frances (Cat 2)	52					
Hurricane Georges (Cat 4)	1	1	24		24	24
Hurricane Jeanne (Cat 3)	78	1	16		16	16
Hurricane Katrina (Cat 1)	7					
Hurricane Rita (Cat 2)	6					
Hurricane Wilma (Cat 3)	247					
Hurricane/Tropical Storm Fay	9	1	24		24	24
TOTALS	420	4				

Very little information on roadway closures is available from published news. According to SERT (2005), due to Hurricane Dennis, there were road and bridge closures on the following routes: Hwy 98 from SR 65 east to Carrabelle, in Franklin county, closed due to wash out; Stumphole (C.R. 30-E) at Cape San Blas from S.R. 30A to St. Joseph State Park, in Gulf County, open to local traffic only; U.S. 98 from Okaloosa Island to Destin in Okaloosa County was closed. Tolls were suspended on the Mid-Bay and Garcon Point Bridges in Florida's panhandle. Also according to FEMA report on damage during Jeanne/Frances, the Sunshine Skyway Bridge was closed due to high winds. Flooding also led to several road closures and 106 tornadoes were attributed to Hurricane Frances.

It can thus be generally assumed that bridges will be closed both for precautionary measures and also due to physical damage to the bridge elements. Table 2.20 indicates one or two days of road closure duration. Using these assumed durations, existing models that were developed for Florida bridges from previous studies can be used to estimate user costs, i.e., costs associated with user delays and vehicle operations.

When multiple bridges in an area are affected by a single event, it may be necessary to take a network approach, estimating the impact of the bridges' closures on the network of roadways that connect them, rather than isolating each bridge's user costs. This can be primarily done through traffic simulation of the roadway network, using vehicle volume data and other pertinent attributes of the roadways. Scenarios are then created on the network to see the impacts of closures of certain segments of the roadways. The results were evaluated in terms of resulting estimated vehicle delays. Such preliminary network analysis was done using traffic data and roadway network of the Tampa Bay area in Florida.

2.3.5.1. Preliminary network traffic analysis of roadway and bridges in Tampa Bay Area
The network simulations of the Tampa Bay area bridges evaluate the lengthy delays and high user costs that occur from a prolonged closure of the bridges. The simulations were created to understand how bridge closures from extensive hurricane damage would impact are traffic and see if the results justify extensive retrofitting of any of the bridges to prevent closure.

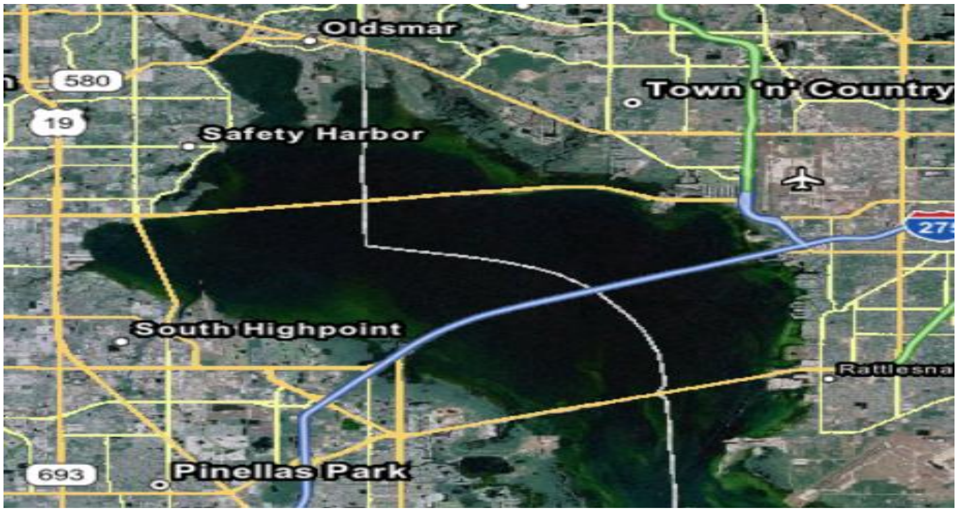


Figure 2.36. Map layout of the Tampa Bay area

The corridor chosen for the network simulation of bay area traffic near Tampa and St. Petersburg Florida is bounded by US-19 to the west, Curlew Rd, Tampa Rd, and W Hillsborough Avenue to the north, Dale Mabry Highway to the East, and Gandy Boulevard North to the south (Figure 2.36). The bridges on the three roadways (US-92, I-275, and SR-60) are being assessed for additional delays occurring if they were to be critically damaged during a hurricane.

The traffic simulations and data were created using CORSIM , a traffic simulation software program. The network was created from connecting links, which function as the roads, using nodes, which function as intersections. The majority of the data necessary to create a functioning and accurate network was taken from GIS layers provided by the FDOT Planning Office. The framework of the network of roadways was created using the GIS measure tool and the CORSIM's x,y coordinate system. The GIS ArcMap provides some information about interchange shape but satellite maps from Google Earth were utilized for the creation of more accurate interchange shapes (Figure 2.37). A similar method was used for link lane data. Some of this data is provided by FDOT GIS data and this information was supplemented by Google Earth satellite maps.

Figure 2.37. Sample aerial photographs used to identify intersections and interchanges

Once the network was created, numerous amounts of data were required to create accurate simulations, the most important being the traffic volumes. Volume data can only be entered at entry nodes in the CORSIM network. This pertinent information such as the directional volumes was derived by appropriately adjusting the AADT provided by FDOT GIS data by the roadway K-factor and D-factor. The interior roadway volumes are simulated by manipulating the relative turn volumes at each node in the network. In order to be as accurate as possible, a Microsoft Excel program was created to estimate the volume of traffic each road experiences as the relative turn volumes are manipulated (Figure 2.38). Given the preliminary nature of this network impact study, the relative turn ratios at intersections was not the most accurate. But the direct link volumes are accurate. Some other situations unaccounted for are the effects of businesses and neighborhoods on the traffic but it is assumed that these are reflected to some extent on the recorded traffic volumes. CORSIM requires information on which turn movements are signalized and the cycle lengths. Once the turn volume data was entered into the nodes, the only information needed was for signalized intersections, the inclusion of left and/or right turn pockets, the lengths of the left/right turn pockets, and freeway acceleration/deceleration lane lengths. Since no data was available, reasonable assumptions were made on the data to provide best traffic flow during the fully operational condition.

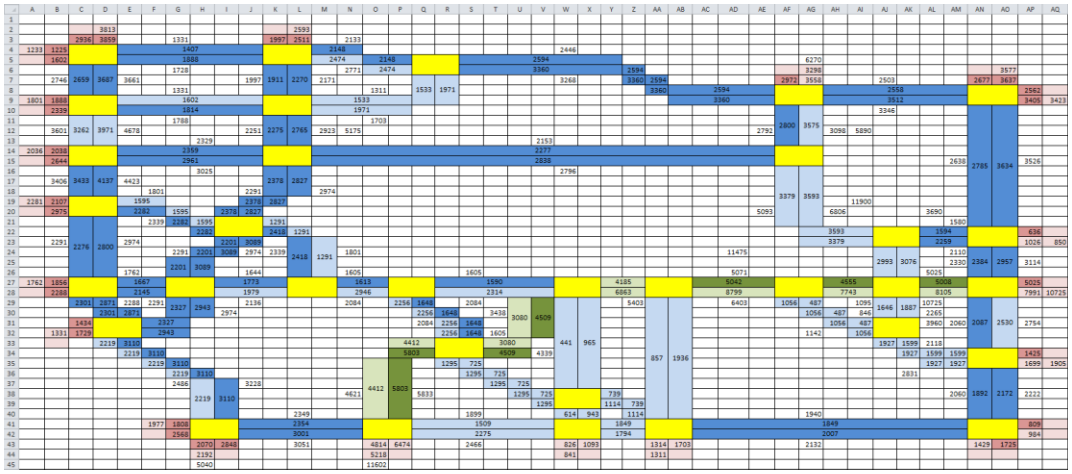


Figure 2.38. Traffic volumes at nodes and links on the Tampa bay area roadway network

Once the fully operation condition network, coded Tampa Bay Bridges, was created, certain turn volumes were changed to effectively "close" a bridge. Traffic was not allowed to enter links that led to the bridge and all traffic was diverted off of roadways that provided through traffic to the bridge. Four different networks were created: The "I-275 Bridge Closed" scenario, the "US-92 Bridge Closed" scenario, the "SR-60 Bridge Closed" scenario, and the "US-92 and SR-60 Bridges Closed" scenario. CORSIM then evaluates each network for delay and provides an animated simulation of each network. One complication is that the I-275 simulation causes so much backup on the freeway links that some of the simulation vehicles "miss" their turns and end up traveling on the "closed" bridge. The percentage of traffic that this occurs with is minor. The simulations as a whole show that the closure of any of the bridges will cause some significant delays but none as severe as when the I-275 bridge is closed.

The results, though with limitations as described above are shown in the following figures, with the captions explaining the specific output.

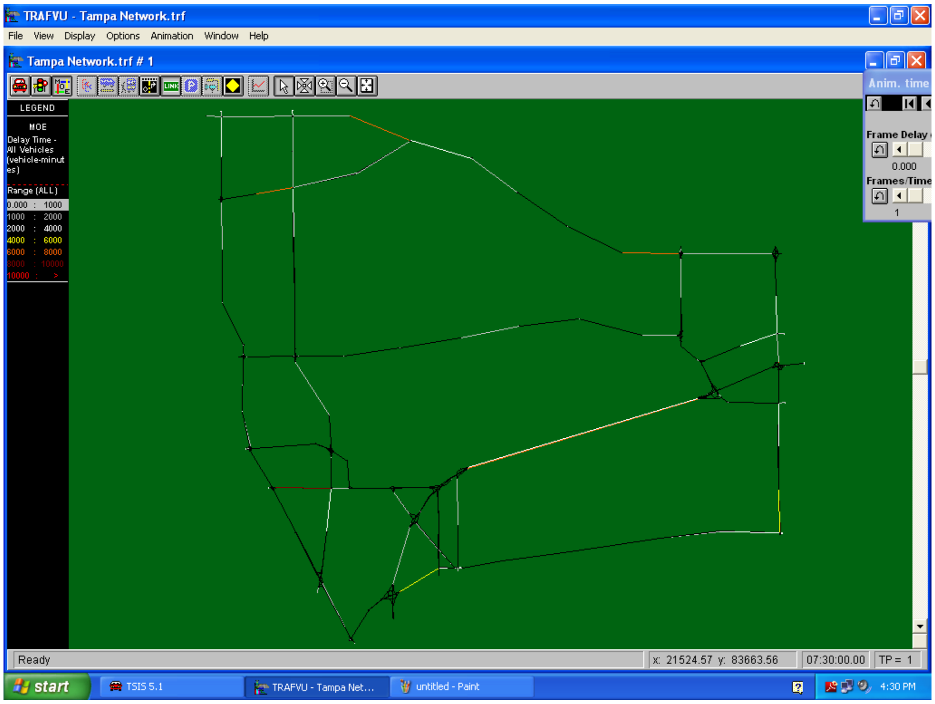


Figure 2.39. Fully operational network (90 minutes of simulation)

Medium delay on eastbound I-275 bridge. Minor delays occurring on links carrying incoming traffic into intersections.

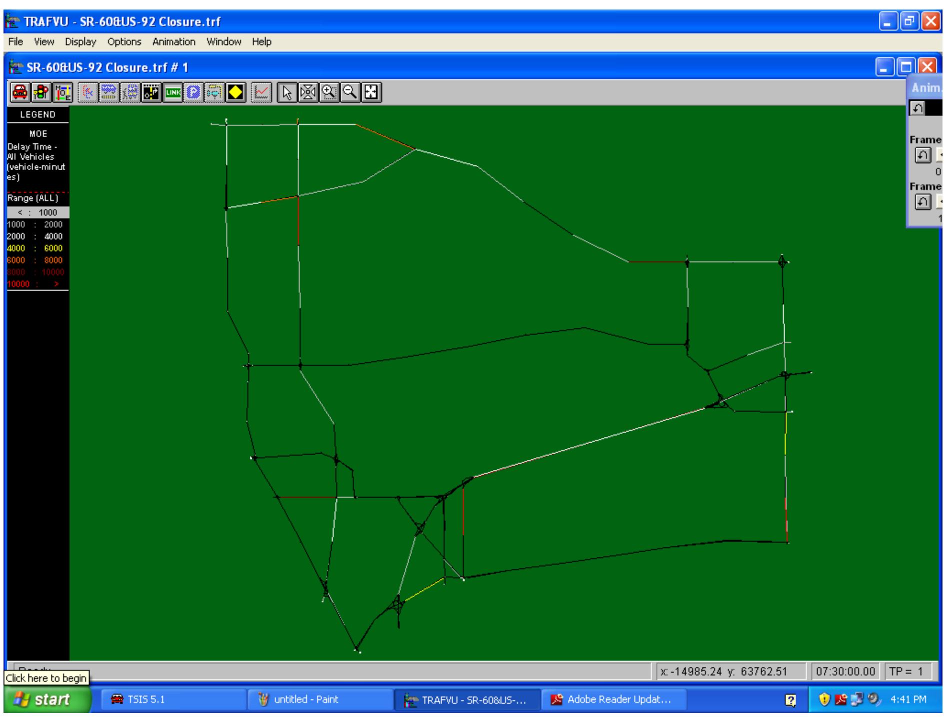


Figure 2.40. SR-60 and US-92 closure on network (90 minutes of simulation)

Significant higher delays occurring in northern surface streets where only medium delays were in fully operational. I-275 Bridge experiencing extreme eastbound delay.

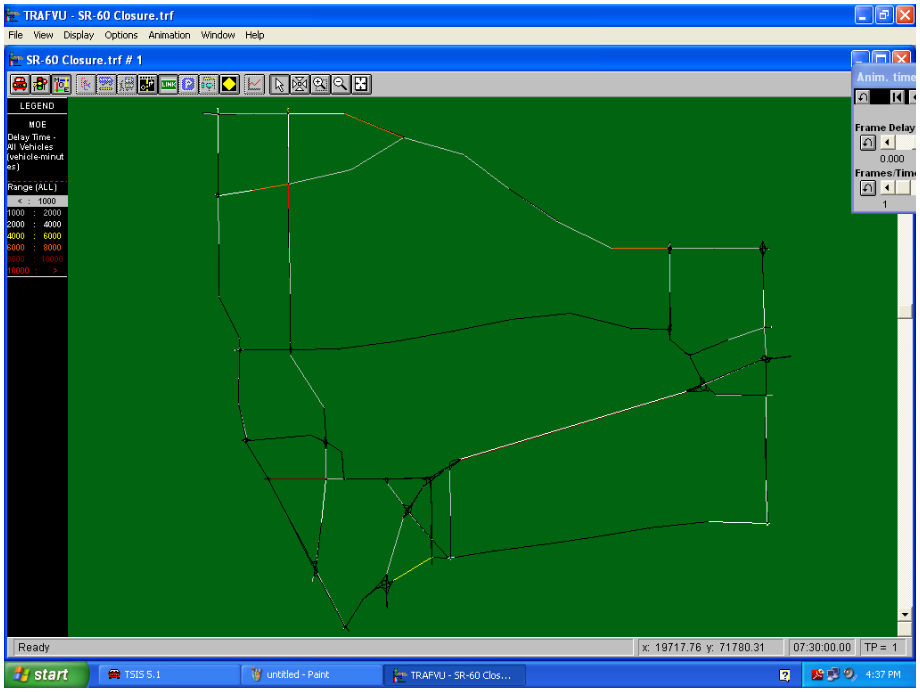


Figure 2.41. SR-60 closure on network (90 minutes of simulation)

Northern routes which serve as closest detour experiencing high to extreme delay. More significant than fully operational.

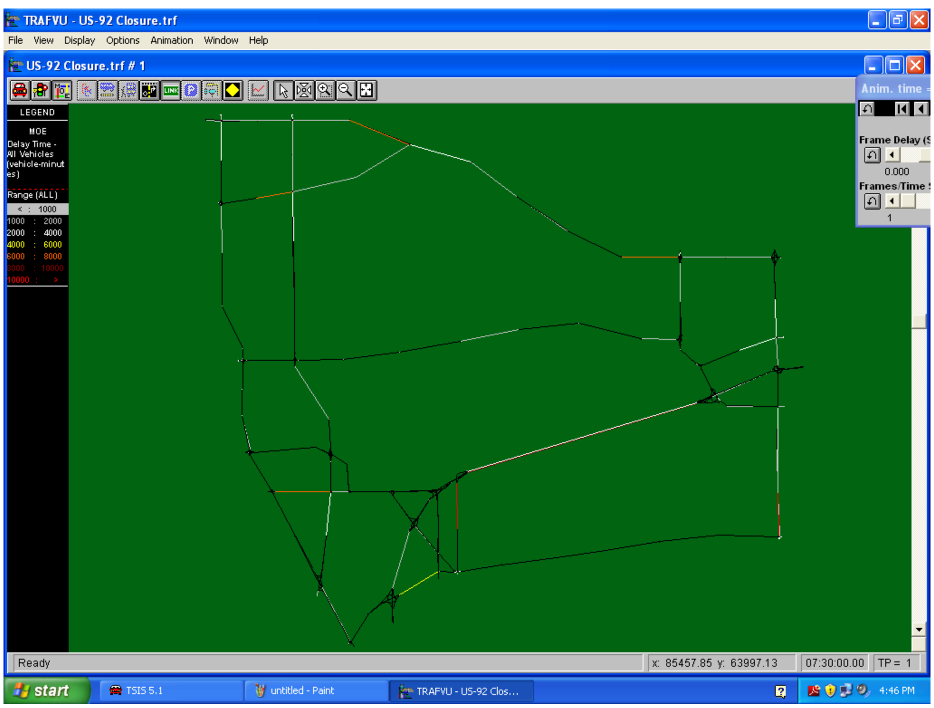


Figure 2.42. US-92 closure on network (90 minutes of simulation)

I-275 bridge eastbound experiencing extreme delay. Intersections close to the closed bridge receiving heavy delay.

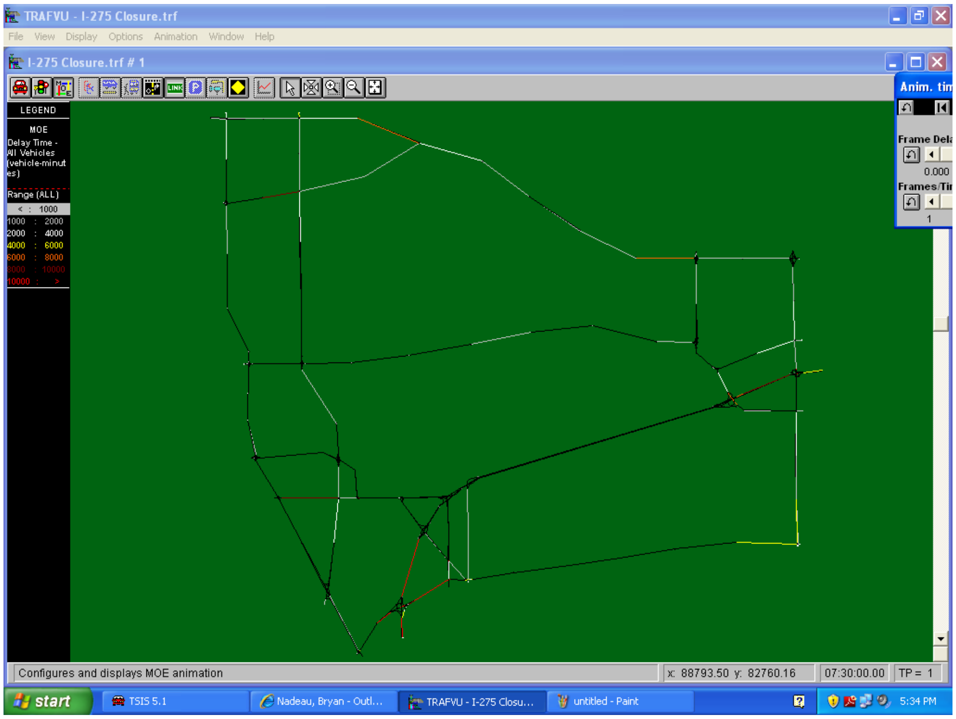


Figure 2.43. I-275 closure on network (90 minutes of simulation)

Extreme delay located on interchanges to I-275. Increased intensity of delay compared with the fully operational network.

2.4. Summary

The vulnerability of bridge elements to hurricane hazards are summarized in Tables 2.22 and 2.23 while the levels of damage to the bridge in general by virtue of design and material type are shown in Table 2.24. During hurricanes, the data collected in this study show that culverts, channel (scour incl. pile scour), sign structures, movable bridge elements, small signs, street lights, guardrails, barriers, and handrails are most vulnerable to damage. But consideration of reported damage to coastal bridges from Hurricane Ivan (I-10 Escambia Bay Bridges) and Hurricane Katrina (Padgett et al., 2008) would make almost all bridge elements significantly vulnerable.

There was a need to separate the coastal bridges from the non-coastal bridges mainly because the extents of damage from hurricanes are very different for the two cases. The storm surge and wave loading effects of the hurricanes affects only the coastal bridges, resulting in specific types of damage, which are more pronounced than damage known to occur on non-coastal bridges. Most of the damages observed from Florida records were for hurricane categories one to three.

For the bridge elements recorded as being damaged on non-coastal bridges, the level of damage at hurricane category 3 was adopted for both categories one and two. For the more severe hurricane categories four and five, the level of damage was estimated as being one unit and two units, respectively, above damage level observed for category three (Table 2.22). For certain elements not specifically identified as damaged in the Florida records, some adjustments were made, to use the level of damage observed for similar elements. For instance, pile jackets is expected to experience the same level of damage as substructure elements like walls, for which there was a record of damage. Traffic arms and signs are expected to be damaged like sign structures. Minimum level (1) of damage was assigned to elements expected to suffer little damage (e.g., joints and bearings on non-coastal bridges), while elements which are not expected to suffer damage at all (e.g., girders on non-coastal bridges), were assigned the value of 0.5, just to avoid elements with zero levels of vulnerability. This is based on observed damage on Florida bridges and publications elsewhere on hurricane damage to bridges. Most movable bridge elements damaged in Florida were at level 5 (extreme level) with one element at level 2; these were mostly coastal bridges. But even for non-coastal bridges, damage by inundation is very common and many movable bridge elements are very vulnerable to damage at the extreme level. In particular, the operator facility of movable bridges can easily be damaged during hurricanes.

For coastal bridges, the damage data on categories four to five is very limited for Florida bridges as mentioned earlier, but the data from outside Florida, as reported in various publications, were used to establish the vulnerability levels. Other than the case of the Escambia Bridges in Florida, many reported cases from Texas, Louisiana, and Alabama were used, including Hurricanes Ivan and Katrina. These cases were all category three hurricanes, but the damages were very pronounced. The levels of damage observed in the category three was adopted for category two but reduced by one unit for category one; for categories four and five, the level of vulnerability was increased by one unit, if the level at category three was not 5 already (Table 2.23). In terms of bridge superstructure design type and material type, the vulnerability of coastal bridges to damage from hurricanes is shown in Table 2.24. The empty cells in the table indicate where the design and material type of bridge does not exist. One example from the information in this table is that non-continuous, slab and girder/beam design types for timber and concrete coastal bridges are the most vulnerable to damage from hurricanes.

Costs associated with the repair and replacement of bridge elements or entire bridges were presented in this report, including costs from other reports for hazards outside Florida. Where the data were available, repair costs at the various levels of damage were estimated in terms of the average total costs or unit costs.

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
12	Bare Concrete Deck	1	1	1	2	2
13	Unp Conc Deck/AC Ovl	1	1	1	2	2
28	Steel Deck/Open Grid	1	1	1	2	2
29	Steel Deck/Conc Grid	1	1	1	2	2
30	Corrug/Orthotpc Deck	1	1	1	2	2
31	Timber Deck	1	1	1	2	2
32	Timber Deck/AC Ovly	1	1	1	2	2
38	Bare Concrete Slab	1	1	1	2	2
39	Unp Conc Slab/AC Ovl	1	1	1	2	2
54	Timber Slab	1	1	1	2	2
55	Timber Slab/AC Ovly	1	1	1	2	2
98	Conc Deck on PC Pane	1	1	1	2	2
99	PS Conc Slab	1	1	1	2	2
101	Unpnt Stl Box Girder	0.5	0.5	0.5	2	2
102	Paint Stl Box Girder	0.5	0.5	0.5	2	2
104	P/S Conc Box Girder	0.5	0.5	0.5	2	2
105	R/Conc Box Girder	0.5	0.5	0.5	2	2
106	Unpnt Stl Opn Girder	0.5	0.5	0.5	2	2
107	Paint Stl Opn Girder	0.5	0.5	0.5	2	2
109	P/S Conc Open Girder	0.5	0.5	0.5	2	2
110	R/Conc Open Girder	0.5	0.5	0.5	2	2
111	Timber Open Girder	0.5	0.5	0.5	2	2
112	Unpnt Stl Stringer	0.5	0.5	0.5	2	2
113	Paint Stl Stringer	0.5	0.5	0.5	2	2
115	P/S Conc Stringer	0.5	0.5	0.5	2	2
116	R/Conc Stringer	0.5	0.5	0.5	2	2
117	Timber Stringer	0.5	0.5	0.5	2	2
120	U/Stl Thru Truss/Bot	0.5	0.5	0.5	2	2
121	P/Stl Thru Truss/Bot	0.5	0.5	0.5	2	2
125	U/Stl Thru Truss/Top	0.5	0.5	0.5	2	2
126	P/Stl Thru Truss/Top	0.5	0.5	0.5	2	2
130	Unpnt Stl Deck Truss	0.5	0.5	0.5	2	2
131	Paint Stl Deck Truss	0.5	0.5	0.5	2	2
135	Timber Truss/Arch	0.5	0.5	0.5	2	2
140	Unpnt Stl Arch	0.5	0.5	0.5	2	2
141	Paint Stl Arch	0.5	0.5	0.5	2	2
143	P/S Conc Arch	0.5	0.5	0.5	2	2
144	R/Conc Arch	0.5	0.5	0.5	2	2
145	Other Arch	0.5	0.5	0.5	2	2
146	Misc Cable Uncoated	0.5	0.5	0.5	2	2
147	Misc Cable Coated	0.5	0.5	0.5	2	2
151	Unpnt Stl Floor Beam	0.5	0.5	0.5	2	2
152	Paint Stl Floor Beam	0.5	0.5	0.5	2	2
154	P/S Conc Floor Beam	0.5	0.5	0.5	2	2
155	R/Conc Floor Beam	0.5	0.5	0.5	2	2
156	Timber Floor Beam	0.5	0.5	0.5	2	2
160	Unpnt Stl Pin/Hanger	0.5	0.5	0.5	2	2
161	Paint Stl Pin/Hanger	0.5	0.5	0.5	2	2
201	Unpnt Stl Column	1	1	1	2	2
202	Paint Stl Column	1	1	1	2	2
204	P/S Conc Column	1	1	1	2	2
205	R/Conc Column	1	1	1	2	2
206	Timber Column	1	1	1	2	2
207	P/S Conc Holl Pile	1	1	1	2	2
210	R/Conc Pier Wall	2	2	2	3	3

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
211	Other Mtl Pier Wall	2	2	2	3	3
215	R/Conc Abutment	2	2	2	3	3
216	Timber Abutment	2	2	2	3	3
217	Other Mtl Abutment	2	2	2	3	3
220	R/C Sub Pile Cap/Ftg	1	1	1	2	2
230	Unpnt Stl Cap	1	1	1	2	2
231	Paint Stl Cap	1	1	1	2	2
233	P/S Conc Cap	1	1	1	2	2
234	R/Conc Cap	1	1	1	2	2
235	Timber Cap	1	1	1	2	2
240	Metal Culvert	3	3	3	4	4
241	Concrete Culvert	3	3	3	4	4
242	Timber Culvert	3	3	3	4	4
243	Misc Culvert	3	3	3	4	4
290	Channel	3	3	3	4	4
298	Pile Jacket Bare	2	2	2	3	3
299	Pile Jacket/Cath Pro	2	2	2	3	3
300	Strip Seal Exp Joint	1	1	1	2	2
301	Pourable Joint Seal	1	1	1	2	2
302	Compressn Joint Seal	1	1	1	2	2
303	Assembly Joint/Seal	1	1	1	2	2
304	Open Expansion Joint	1	1	1	2	2
310	Elastomeric Bearing	1	1	1	2	2
311	Moveable Bearing	1	1	1	2	2
312	Enclosed Bearing	1	1	1	2	2
313	Fixed Bearing	1	1	1	2	2
314	Pot Bearing	1	1	1	2	2
315	Disk Bearing	1	1	1	2	2
320	P/S Conc Appr Slab	2	2	2	3	3
321	R/Conc Approach Slab	2	2	2	3	3
330	Metal Rail Uncoated	2	2	2	3	3
331	Conc Bridge Railing	2	2	2	3	3
332	Timb Bridge Railing	2	2	2	3	3
333	Other Bridge Railing	2	2	2	3	3
334	Metal Rail Coated	2	2	2	3	3
356	Steel Fatigue SmFlag	0	0	0	0	0
357	Pack Rust Smart Flag	0	0	0	0	0
358	Deck Cracking SmFlag	0	0	0	0	0
359	Soffit Smart Flag	0	0	0	0	0
360	Settlement SmFlag	0	0	0	0	0
361	Scour Smart Flag	0	0	0	0	0
362	Traf Impact SmFlag	0	0	0	0	0
363	Section Loss SmFlag	0	0	0	0	0
369	Sub.Sect Loss SmFlag	0	0	0	0	0
370	Alert Smart Flag	0	0	0	0	0
386	Fender/Dolphin Uncoa	2	2	2	3	3
387	P/S Fender/Dolphin	2	2	2	3	3
388	R/Conc Fender/Dolphi	2	2	2	3	3
389	Timber Fender/Dolphi	2	2	2	3	3
390	Other Fender/Dolphin	2	2	2	3	3
393	Blkhd Sewl Metal Unc	2	2	2	3	3
394	R/Conc Abut Slope Pr	2	2	2	3	3
395	Timber Abut Slope Pr	2	2	2	3	3
396	Other Abut Slope Pro	2	2	2	3	3
397	Drain. Syst Metal	1	1	1	2	2
398	Drain. Syst Other	1	1	1	2	2
399	Other Xpansion Joint	1	1	1	2	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
474	Walls Uncoated	3	3	3	4	4
475	R/Conc Walls	3	3	3	4	4
476	Timber Walls	3	3	3	4	4
477	Other Walls	3	3	3	4	4
478	MSE Walls	3	3	3	4	4
480	Mast Arm Found	4	4	4	5	5
481	Paint Mast Arm Vert	4	4	4	5	5
482	Galvan Mast Arm Vert	4	4	4	5	5
483	Other Mast Arm Vert	4	4	4	5	5
484	Paint Mast Arm Horzn	4	4	4	5	5
485	Galvan Mast Arm Horz	4	4	4	5	5
486	Other Mast Arm Horzn	4	4	4	5	5
487	Sign Member Horiz	4	4	4	5	5
488	Sign Member Vertical	4	4	4	5	5
489	Sign Foundation	5	5	5	5	5
495	Uncoat High Mast L.	4	4	4	5	5
496	Painted High Mast L.	4	4	4	5	5
497	Galvan. High Mast L.	4	4	4	5	5
498	Other High Mast L.P.	4	4	4	5	5
499	H. M. L. P. Found.	4	4	4	5	5
540	Open Gearing	2	2	2	3	3
541	Speed Reducers	2	2	2	3	3
542	Shafts	2	2	2	3	3
543	Shaft Brgs and Coupl	2	2	2	3	3
544	Brakes	2	2	2	3	3
545	Emergency Drive	2	2	2	3	3
546	Span Drive Motors	2	2	2	3	3
547	Hydraulic Power Unit	2	2	2	3	3
548	Hydraulic Piping Sys	2	2	2	3	3
549	Hydraulic Cylinders	2	2	2	3	3
550	Hopkins Frame	2	2	2	3	3
560	Locks	2	2	2	3	3
561	Live Load Shoes	2	2	2	3	3
562	Counterweight Suppor	2	2	2	3	3
563	Acc Ladd & Plat	2	2	2	3	3
564	Counterweight	2	2	2	3	3
565	Trun/Str and Cur Trk	2	2	2	3	3
570	Transformers	2	2	2	3	3
571	Submarine Cable	2	2	2	3	3
572	Conduit & Junc. Box	2	2	2	3	3
573	PLCs	2	2	2	3	3
574	Control Console	2	2	2	3	3
580	Navigational Lights	2	2	2	3	3
581	Operator Facilities	5	5	5	5	5
582	Lift Bridge Spec. Eq	2	2	2	3	3
583	Swing Bridge Spec. E	2	2	2	3	3
590	Resistance Barriers	2	2	2	3	3
591	Warning Gates	5	5	5	5	5
592	Traffic Signals	5	5	5	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage.

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
12	Bare Concrete Deck	4	5	5	5	5
13	Unp Conc Deck/AC Ovl	4	5	5	5	5
28	Steel Deck/Open Grid	4	5	5	5	5
29	Steel Deck/Conc Grid	4	5	5	5	5
30	Corrug/Orthotpc Deck	4	5	5	5	5
31	Timber Deck	4	5	5	5	5
32	Timber Deck/AC Ovly	4	5	5	5	5
38	Bare Concrete Slab	4	5	5	5	5
39	Unp Conc Slab/AC Ovl	4	5	5	5	5
54	Timber Slab	4	5	5	5	5
55	Timber Slab/AC Ovly	4	5	5	5	5
98	Conc Deck on PC Pane	4	5	5	5	5
99	PS Conc Slab	4	5	5	5	5
101	Unpnt Stl Box Girder	4	5	5	5	5
102	Paint Stl Box Girder	4	5	5	5	5
104	P/S Conc Box Girder	4	5	5	5	5
105	R/Conc Box Girder	4	5	5	5	5
106	Unpnt Stl Opn Girder	4	5	5	5	5
107	Paint Stl Opn Girder	4	5	5	5	5
109	P/S Conc Open Girder	4	5	5	5	5
110	R/Conc Open Girder	4	5	5	5	5
111	Timber Open Girder	4	5	5	5	5
112	Unpnt Stl Stringer	4	5	5	5	5
113	Paint Stl Stringer	4	5	5	5	5
115	P/S Conc Stringer	4	5	5	5	5
116	R/Conc Stringer	4	5	5	5	5
117	Timber Stringer	4	5	5	5	5
120	U/Stl Thru Truss/Bot	3	4	4	5	5
121	P/Stl Thru Truss/Bot	3	4	4	5	5
125	U/Stl Thru Truss/Top	3	4	4	5	5
126	P/Stl Thru Truss/Top	3	4	4	5	5
130	Unpnt Stl Deck Truss	3	4	4	5	5
131	Paint Stl Deck Truss	3	4	4	5	5
135	Timber Truss/Arch	3	4	4	5	5
140	Unpnt Stl Arch	3	4	4	5	5
141	Paint Stl Arch	3	4	4	5	5
143	P/S Conc Arch	3	4	4	5	5
144	R/Conc Arch	3	4	4	5	5
145	Other Arch	3	4	4	5	5
146	Misc Cable Uncoated	3	4	4	5	5
147	Misc Cable Coated	3	4	4	5	5
151	Unpnt Stl Floor Beam	4	5	5	5	5
152	Paint Stl Floor Beam	4	5	5	5	5
154	P/S Conc Floor Beam	4	5	5	5	5
155	R/Conc Floor Beam	4	5	5	5	5
156	Timber Floor Beam	4	5	5	5	5
160	Unpnt Stl Pin/Hanger	4	5	5	5	5
161	Paint Stl Pin/Hanger	4	5	5	5	5
201	Unpnt Stl Column	2	3	3	4	4
202	Paint Stl Column	2	3	3	4	4
204	P/S Conc Column	2	3	3	4	4
205	R/Conc Column	2	3	3	4	4
206	Timber Column	2	3	3	4	4
207	P/S Conc Holl Pile	2	3	3	4	4
210	R/Conc Pier Wall	2	3	3	4	4

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
211	Other Mtl Pier Wall	2	3	3	4	4
215	R/Conc Abutment	2	3	3	4	4
216	Timber Abutment	2	3	3	4	4
217	Other Mtl Abutment	2	3	3	4	4
220	R/C Sub Pile Cap/Ftg	2	3	3	4	4
230	Unpnt Stl Cap	2	3	3	4	4
231	Paint Stl Cap	2	3	3	4	4
233	P/S Conc Cap	2	3	3	4	4
234	R/Conc Cap	2	3	3	4	4
235	Timber Cap	2	3	3	4	4
240	Metal Culvert	3	4	4	5	5
241	Concrete Culvert	3	4	4	5	5
242	Timber Culvert	3	4	4	5	5
243	Misc Culvert	3	4	4	5	5
290	Channel	4	5	5	5	5
298	Pile Jacket Bare	3	4	4	5	5
299	Pile Jacket/Cath Pro	3	4	4	5	5
300	Strip Seal Exp Joint	3	4	4	5	5
301	Pourable Joint Seal	3	4	4	5	5
302	Compressn Joint Seal	3	4	4	5	5
303	Assembly Joint/Seal	3	4	4	5	5
304	Open Expansion Joint	3	4	4	5	5
310	Elastomeric Bearing	3	4	4	5	5
311	Moveable Bearing	3	4	4	5	5
312	Enclosed Bearing	3	4	4	5	5
313	Fixed Bearing	3	4	4	5	5
314	Pot Bearing	3	4	4	5	5
315	Disk Bearing	3	4	4	5	5
320	P/S Conc Appr Slab	4	5	5	5	5
321	R/Conc Approach Slab	4	5	5	5	5
330	Metal Rail Uncoated	4	5	5	5	5
331	Conc Bridge Railing	4	5	5	5	5
332	Timb Bridge Railing	4	5	5	5	5
333	Other Bridge Railing	4	5	5	5	5
334	Metal Rail Coated	4	5	5	5	5
356	Steel Fatigue SmFlag	0	0	0	0	0
357	Pack Rust Smart Flag	0	0	0	0	0
358	Deck Cracking SmFlag	0	0	0	0	0
359	Soffit Smart Flag	0	0	0	0	0
360	Settlement SmFlag	0	0	0	0	0
361	Scour Smart Flag	0	0	0	0	0
362	Traf Impact SmFlag	0	0	0	0	0
363	Section Loss SmFlag	0	0	0	0	0
369	Sub.Sect Loss SmFlag	0	0	0	0	0
370	Alert Smart Flag	0	0	0	0	0
386	Fender/Dolphin Uncoa	4	5	5	5	5
387	P/S Fender/Dolphin	4	5	5	5	5
388	R/Conc Fender/Dolphi	4	5	5	5	5
389	Timber Fender/Dolphi	4	5	5	5	5
390	Other Fender/Dolphin	4	5	5	5	5
393	Blkhd Sewl Metal Unc	4	5	5	5	5
394	R/Conc Abut Slope Pr	4	5	5	5	5
395	Timber Abut Slope Pr	4	5	5	5	5
396	Other Abut Slope Pro	4	5	5	5	5
397	Drain. Syst Metal	3	4	4	5	5
398	Drain. Syst Other	3	4	4	5	5
399	Other Xpansion Joint	3	4	4	5	5
474	Walls Uncoated	3	4	4	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
475	R/Conc Walls	3	4	4	5	5
476	Timber Walls	3	4	4	5	5
477	Other Walls	3	4	4	5	5
478	MSE Walls	3	4	4	5	5
480	Mast Arm Found	3	4	4	5	5
481	Paint Mast Arm Vert	3	4	4	5	5
482	Galvan Mast Arm Vert	3	4	4	5	5
483	Other Mast Arm Vert	3	4	4	5	5
484	Paint Mast Arm Horzn	3	4	4	5	5
485	Galvan Mast Arm Horz	3	4	4	5	5
486	Other Mast Arm Horzn	3	4	4	5	5
487	Sign Member Horiz	3	4	4	5	5
488	Sign Member Vertical	3	4	4	5	5
489	Sign Foundation	3	4	4	5	5
495	Uncoat High Mast L.	3	4	4	5	5
496	Painted High Mast L.	3	4	4	5	5
497	Galvan. High Mast L.	3	4	4	5	5
498	Other High Mast L.P.	3	4	4	5	5
499	H. M. L. P. Found.	3	4	4	5	5
540	Open Gearing	3	4	4	5	5
541	Speed Reducers	3	4	4	5	5
542	Shafts	3	4	4	5	5
543	Shaft Brgs and Coupl	3	4	4	5	5
544	Brakes	3	4	4	5	5
545	Emergency Drive	3	4	4	5	5
546	Span Drive Motors	3	4	4	5	5
547	Hydraulic Power Unit	3	4	4	5	5
548	Hydraulic Piping Sys	3	4	4	5	5
549	Hydraulic Cylinders	3	4	4	5	5
550	Hopkins Frame	3	4	4	5	5
560	Locks	3	4	4	5	5
561	Live Load Shoes	3	4	4	5	5
562	Counterweight Suppor	3	4	4	5	5
563	Acc Ladd & Plat	3	4	4	5	5
564	Counterweight	3	4	4	5	5
565	Trun/Str and Cur Trk	3	4	4	5	5
570	Transformers	3	4	4	5	5
571	Submarine Cable	3	4	4	5	5
572	Conduit & Junc. Box	3	4	4	5	5
573	PLCs	3	4	4	5	5
574	Control Console	3	4	4	5	5
580	Navigational Lights	3	4	4	5	5
581	Operator Facilities	3	4	4	5	5
582	Lift Bridge Spec. Eq	3	4	4	5	5
583	Swing Bridge Spec. E	3	4	4	5	5
590	Resistance Barriers	3	4	4	5	5
591	Warning Gates	3	4	4	5	5
592	Traffic Signals	3	4	4	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

Table 2.24. General vulnerability to hurricane damage for coastal bridges by design and material types

	Superstructure material type								
Superstructure design type	Concrete	Concrete Continuous	Steel	Steel Continuous	Prestressed concrete	Prestressed concrete continuous	Wood or Timber	Masonry	Aluminum, Wrought Iron, or Cast Iron
Slab	5	3			5	3	5		
Stringer/MultiBeam/Girder	5	3	3	2	5	3	5		
Girder & Floorbeam	5	3	3	2	5	3	5		
Tee Beam	5	3	3	2	5	3	5		
Box Beam or Girders - Multiple	5	3	3	2	5	3	5		
Box Beam/Girders - Single or Spread	5	3	3	2	5	3	5		
Frame (except frame culverts)									
Orthotropic									
Truss - Deck									
Truss - Thru									
Arch - Deck								2	
Arch - Thru								2	
Suspension				2					
Stayed Girder				2					
Movable - Lift			4	3					
Movable - Bascule			4	3					
Movable - Swing			4	3					
Tunnel									
Culvert	4		4		4		4	4	4
Mixed Types									
Segmental Box Girder	4								
Channel Beam	4								

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible)

2.5. References

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3. Tornadoes

In Florida, tornadoes can occur at any time of the year. Tornadoes can form on their own, or they can accompany hurricanes and tropical storms. Generally, weather patterns produce the strongest tornadoes between February and May. The extent or strength of a tornado is typically indicated using the Fujita scale as shown in Table 3.1.

Table 3.1. Tornado Classification: The Fujita Scale (NOAA 2011)

Category (F-Number)	Wind Speed mph (km/h)	Intensity Description	Type of Damage
0	40 - 72	Gale Tornado	Minor structural damage. Tree branches break off, and small trees are uprooted. Damages large signs boards.
1	73 - 112	Moderate Tornado	Hurricane wind like speed. Roofs ripped off houses, and mobile homes are overturned. Cars are pushed off the road and attached garages might be destroyed.
2	113-157	Significant Tornado	Extensive damage. Roofs torn off frame houses. Mobile homes destroyed. Trains' box cars turned over. Large trees torn from the ground and snapped. Light objects become deadly missiles.
3	158 - 206	Severe Tornado	Roofs of well built houses are ripped off. Numerous trees ripped out of the earth. Trains are overturned.
4	207 - 260	Devastating Tornado	Well build houses are totally destroyed. Newly build houses are thrown considerable distances. Cars thrown and other large objects become missiles.
5	261 - 318	Incredible Tornado	Well built houses are torn out of the ground and thrown great distances. Trees become debarked, and car size missiles fly up to one hundred meters. Steel reinforced concrete structures are badly damaged.
6	319 - 379	Inconceivable Tornado	Winds at these speeds are highly unlikely, but if a tornado this big did occur, the wreckage would be so great it would be unidentifiable. Everything would be destroyed.

3.1. Risk assessment: likelihood estimates for tornadoes

Tornado data are available from the National Weather Service GIS Data Portal. These data include tornados in the years 1950 to 2010, and hail and wind from 1955 to 2010, in shapefile and Personal Database File formats. The data were prepared in a USA Contiguous Lambert Conformal Conic projection with North American Datum 1983. Figures 3.1 to 3.3 show the national display of Tornado touchdowns, lift points, and tracks respectively.

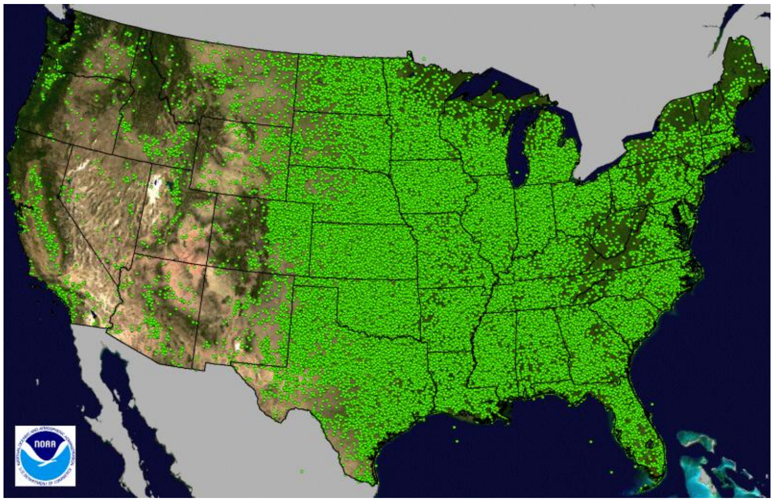

Figure 3.1. Tornado Touchdown Points in the United States (1950 – 2010) (NOAA 2011).

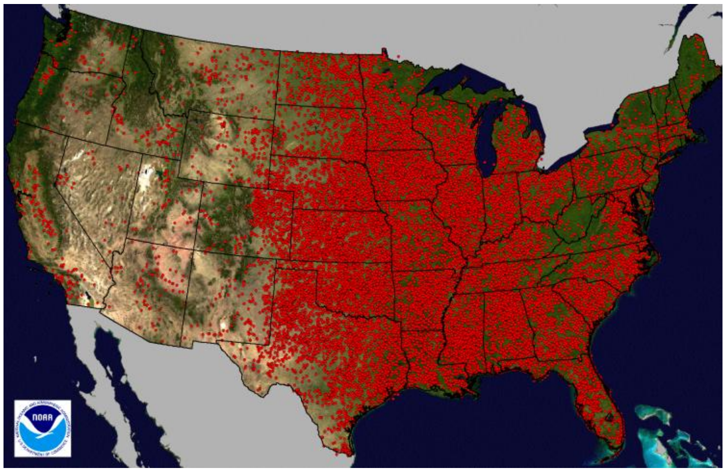

Figure 3.2. Tornado Lift Points in the United States (1950 – 2010) (NOAA 2011).

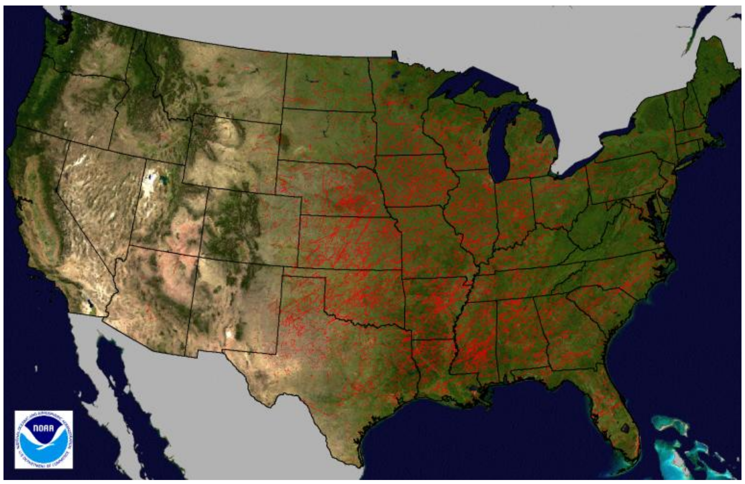

Figure 3.3. Tornado Tracks in the United States (1950 – 2010) (NOAA 2011).

The tornado data in these three formats were downloaded for the state of Florida. Each tornado record indicates important information including the date, time, the Fujita Scale (measure of strength), injuries, fatalities, width, and length. Using ESRI ArcGIS 9.3, simple spatial join commands were used in the ArcToolBox to query the overlay between the Tornado GIS layer and the Florida DOT bridge GIS layer. Using a buffer of 1 mile, the recorded locations of tornadoes near each bridge were identified and the bridge assigned the tornado and its attributes. The 1 mile value as buffer was chosen arbitrarily because the widths of tornados can vary significantly, up to 2.5 miles.

Shown below in figure 3.4 is the identified set of tornadoes historically recorded within one mile of a bridge in a specific area of Leon County. Another bridge is shown in Figure 3.5 with two tornado points in Pasco County.

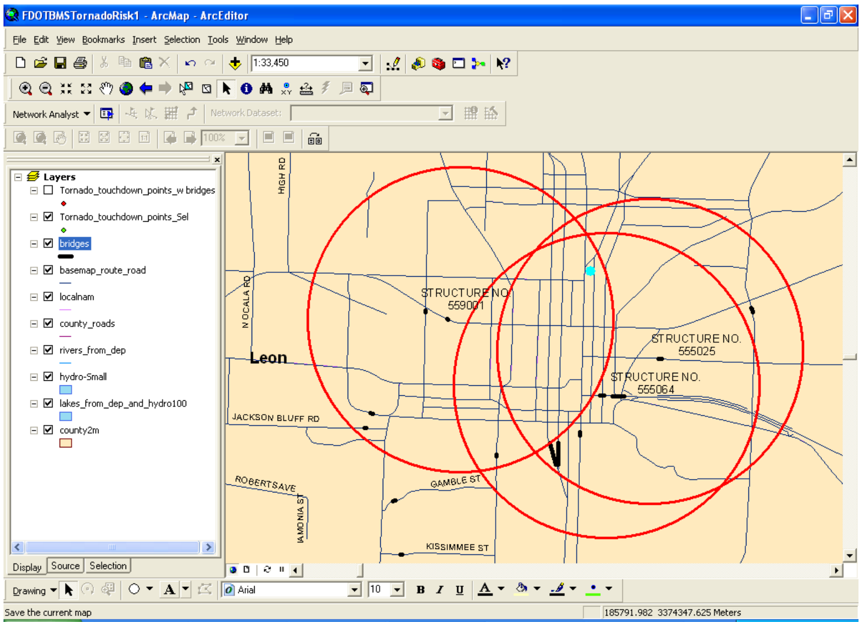

Figure 3.4. Bridges in leon county within one mile of a recorded tornado touch down point.

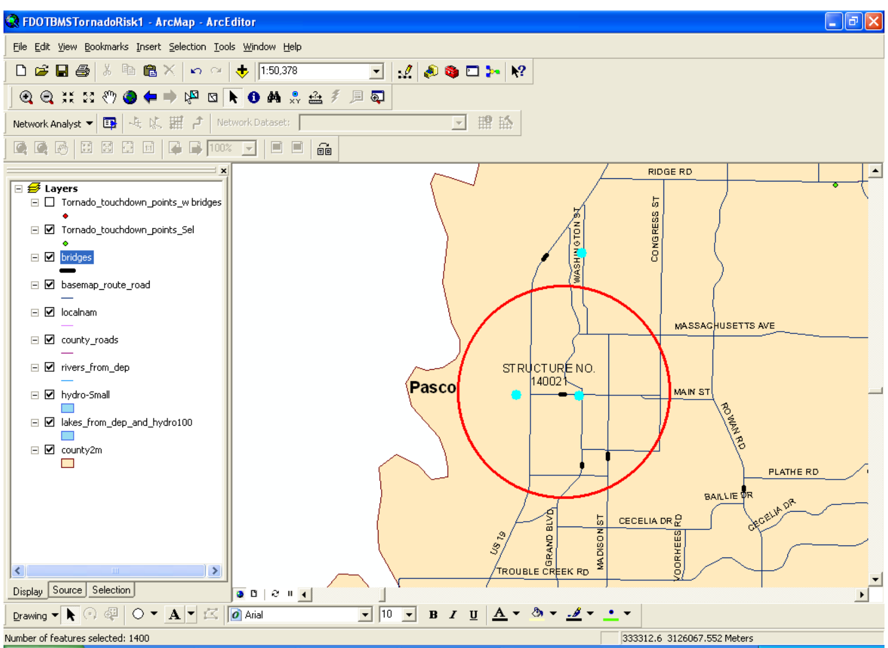

Figure 3.5. Bridges in Pasco county within one mile of recorded tornado touch down points.

3.1.1. Analysis of Tornado data

As described in detail earlier for hurricanes, it will also be assumed that tornadoes touch down at specific locations according to the Poisson distribution. Thus what is needed is the average annual occurrence rate at each bridge location. From the spatially merged data of the tornado and bridge GIS layers, the mean rate is obtained by dividing the frequency or total number of occurrences of tornadoes near the bridge, by the total duration of the data observation, i.e. 60 years, from 1950 to 2010. It was observed that 1301 of the 2231 bridges affected by tornados historically, had experienced only one such incident in the 60 years of observation. There were 788 bridges with between 2 and 5 tornado touch downs during the 60 years while only 64 has 10 or more tornado experiences. In considering only bridges experiencing tornadoes with Fujita scale greater than F1, the frequency was reduced, with 51 of 443 bridges having had more than one of such tornadoes within 60 years. Only two bridges have experienced more than two of those tornado categories in 60 years.

Using the same methodology applied above to hurricane wind occurrence, the probability of occurrence can be predicted for tornadoes also. Let us consider that for example, the Structure ID 154403 used for illustration under the hurricane wind analysis. Within the period of 60 years, 5 tornadoes are recorded to have occurred in the vicinity of this particular bridge location. This implies the mean occurrence rate, or λ = 5/60 or 0.083.

According to equation 2.4 presented in the section of this report on hurricanes, the probability that there will be a tornado at this location within a year is given as

$F_T(1) = P(T \leq 1) = 1 - \exp\left[-0.083 * 1\right]$ or 0.0796.

Also, the probability of having a tornado within the next 10 years at this bridge location is estimated as
$F_T(1) = P(T \leq 10) = 1 - \exp\left[-0.083 * 10\right]$ or 0.564.

Another approach taken in estimating the likelihood of tornado occurrence was using recorded history of tornado events in each county in Florida (NOAA 2011). It was observed that the data had a range of 60 years, dated from March 16, 1950 to August 30, 2010. For category of Tornado, the count of occurrence in each county was determined and divided by the 60-year time span to estimate the mean occurrence rate, or λ as described above. Using the same equations demonstrated above, the probabilities of occurrence of tornadoes within one and 10 years can be estimated.

Estimates of the likelihoods of tornado occurrence near Florida bridges are presented in more detail in Appendix A2. In comparison with other common natural hazards such as hurricanes and wildfires, estimates of the likelihood of tornado occurrence are summarized here for various categories in Figure 3.6. The mean annual probabilities of hazard events appear to decrease with increase in the tornado intensity (category).

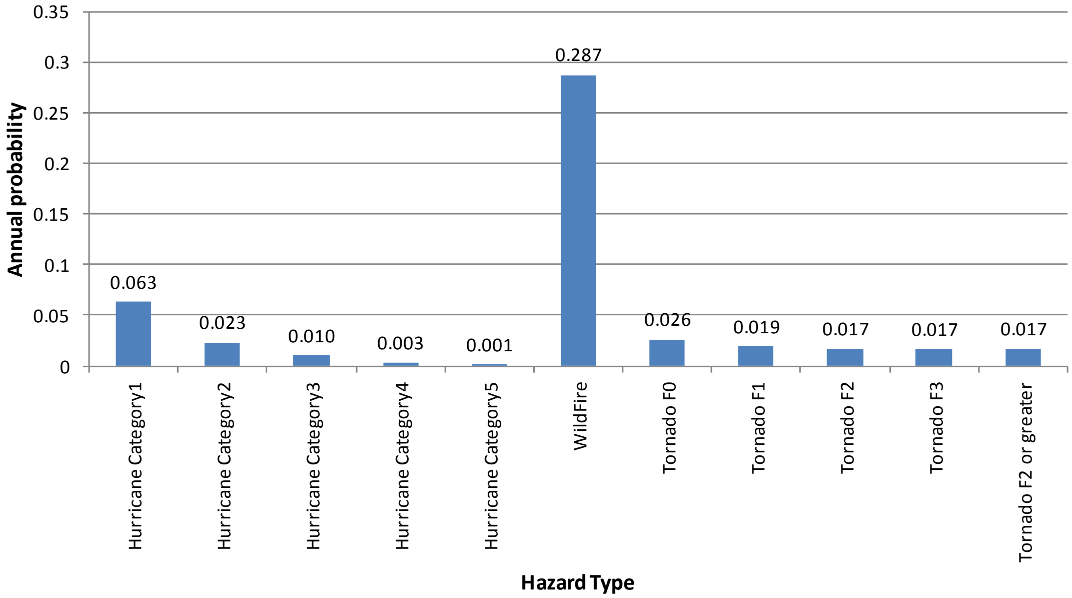

Figure 3.6. Mean annual probabilities of hazard events at Florida bridge locations

3.2. Risk assessment: consequences of tornadoes

Tornado is one of the most dangerous and damaging types of natural hazard in the United States, as they cause loss of lives and considerable property damage. Tornados often create and transport large amounts of debris that can be ejected at high velocities.

3.2.1. Lessons learned from previous tornadoes

Tornado is one of the most dangerous and damaging types of natural hazard in the United States, as they cause loss of lives and considerable property damage. Tornados often create and transport large amounts of debris that can be ejected at high velocities. Pierce et al (2009) states that "According to eyewitness reports and documented damage assessments, some of the largest objects that have been lifted and transported by tornados include refrigerators and pianos, fertilizer tanks and propane tanks, cars, vans, farm wagons, school buses, and tractor-trailers. For instance, an empty fertilizer tank weighing 26,000 pounds was moved about 3,900 feet during a tornado that passed through Lubbock, Texas in May 1970 (Figure 3.7).

Figure 3.7. Examples of tornado projectiles (Pierce et al., 2009)

Due to the development of modern building methods, materials and construction techniques, the span lengths of flexible structures such as suspension and cable-stayed bridges, have been increased to record setting levels. The susceptibility to wind actions of these large bridges is increasing accordingly. The original Tacoma Narrows Bridge in Washington State, opened to traffic in July 1940, was the third largest suspension bridge in the U.S at the time of construction. Failure occurred on the bridge only four months after its opening to the public due to strong winds. The collapse of the bridge shocked and intrigued bridge engineers to conduct various scientific investigations on bridge aerodynamics.

Long-span bridges are more susceptible to wind actions. It has been known that tornado-induced strong winds have much higher turbulence intensity than that of moderate winds, and a high intensity tornado is mainly composed of turbulent winds, and therefore during its occurrence, it is more likely to cause major damage to long span bridges as opposed to short spanned bridges.

The effect of strong winds on different types of bridges can be reviewed by classifying bridges into five categories; beam, truss, arch, suspension, cable stayed girder, and movable bridges. It has been observed that out of the five aforementioned bridge types, long truss beams located high above the ground level are most vulnerable to strong winds. Some bridge elements on movable bridges are very susceptible to damage from strong winds. The first element is the operator facility because it is more or less a building, and buildings are known to be very vulnerable to tornadoes and hurricane strong winds. Other vulnerable elements on the movable bridges include the navigational lights, signs, etc.

Many literatures state that generally, long spanned bridges are more susceptible to the effects of strong winds than shorter spanned ones. For tornado hazards which also involve strong winds, another bridge attribute that should be considered in assigning influence factors is the vertical underclearance. Many bridges which were reported as being completely destroyed during tornadoes were truss bridges that were highly elevated along the spans with very high underclearances. An example is the tornado observed in McKean County, in Pennsylvania, on July 21, 2003, which completely damaged the Kinzua Bridge (Figure 3.8) (McKean 2011). The winds associated with the tornado were estimated to be over 94 mph. It was also concluded that tall trestle bent viaducts was vulnerable to catastrophic collapse in extreme wind events because the anchor bolt systems have been either designed improperly or weakened through corrosion or fatigue.

A tornado also tore through Monticello, Indiana on April 31, 1974, with strong winds pushing four spans of a Penn Central Transportation Company railroad bridge off their piers (Figure 3.9) (Monticello 2011). The bridge deck width was small, being a one-track railroad.

a. before

b. after

Figure 3.8. Damage to the Kinzua Railroad Bridge from tornado in McKean County, Pennsylvania in 2003 (McKean 2011)

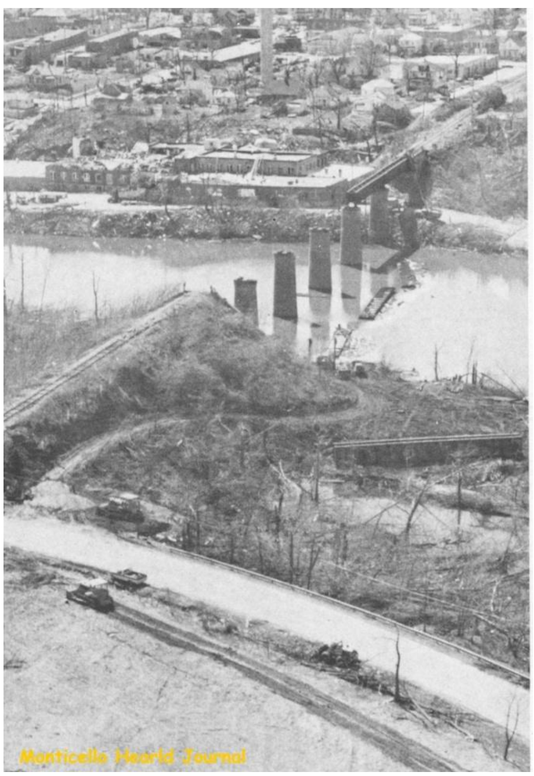

Figure 3.9. Damage to Railroad Bridge from tornado in Monticello, Indiana in 1974 (Monticello 2011)

The main lessons learned on bridges susceptible to damage from strong winds due to hurricanes and tornadoes, include the following:

- Bridges with spans greater than 150 ft should be considered very susceptible particularly those bridges of the truss types with high underclearances.
- Elements on the movable bridges are also susceptible to damages

Using the current Pontis database again, a review was done on Florida's state-maintained bridges which are classified by design as truss, suspension, or stayed girder (using NBI Item No. 42B *designmain*). This filter resulted in 144 bridges. The maximum span length of each bridge was considered as the main attribute for wind-susceptibility. Of the 144 bridges, about 72% have maximum spans of 40 m or less while about 24% have maximum spans between 40m and 160 m (Figure 3.10). The remaining 4% have maximum span lengths between 160 m and 400 m.

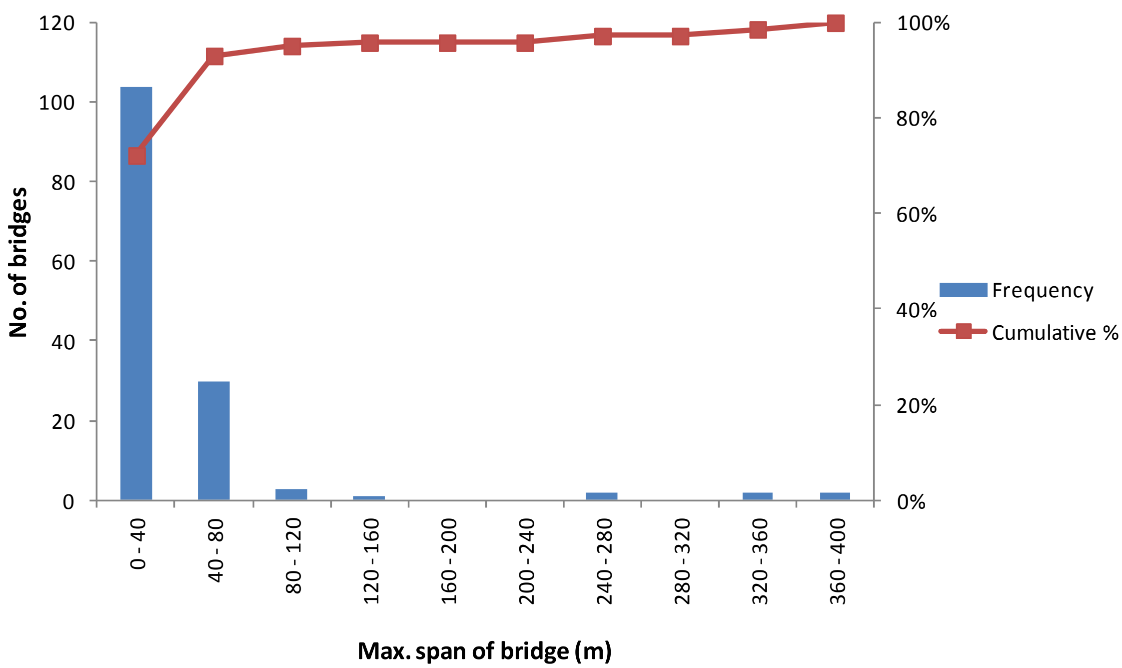

Figure 3.10. Variation in maximum span lengths of state-maintained bridges (truss, suspension or stayed girder)

Among these design type of bridges, those with maximum spans greater than 40 m (40 bridges) were further reviewed for their vertical underclearances. The results are illustrated in Figure 3.11. Of these 40 bridges, 75% or 30 bridges have vertical underclearance greater than 4 m, with one bridge, about 40 m maximum span, having a clearance of 11.5 m (Bridge ID 469017, GRAND PANAMA RESORT, Pedestrian overpass, on US98A SR30A). Nine of these 40 bridges have vertical underclearance over 6 m, and seven of them are pedestrian overpasses while the other two carry the facility “SR-228 (HART BDG.)” crossing over St. Johns River. It appears that the issue of extreme combinations of long span and vertical underclearance on truss, suspension or stayed girder bridges may not be a major concern, as the slightly serious cases identified are for non-vehicular carrying bridges. In other words, based on the consideration of the pertinent bridge attributes, the common tornadoes (up to F4) and strong winds may not really threaten the existing bridge inventory, if the bridges are in good physical conditions.

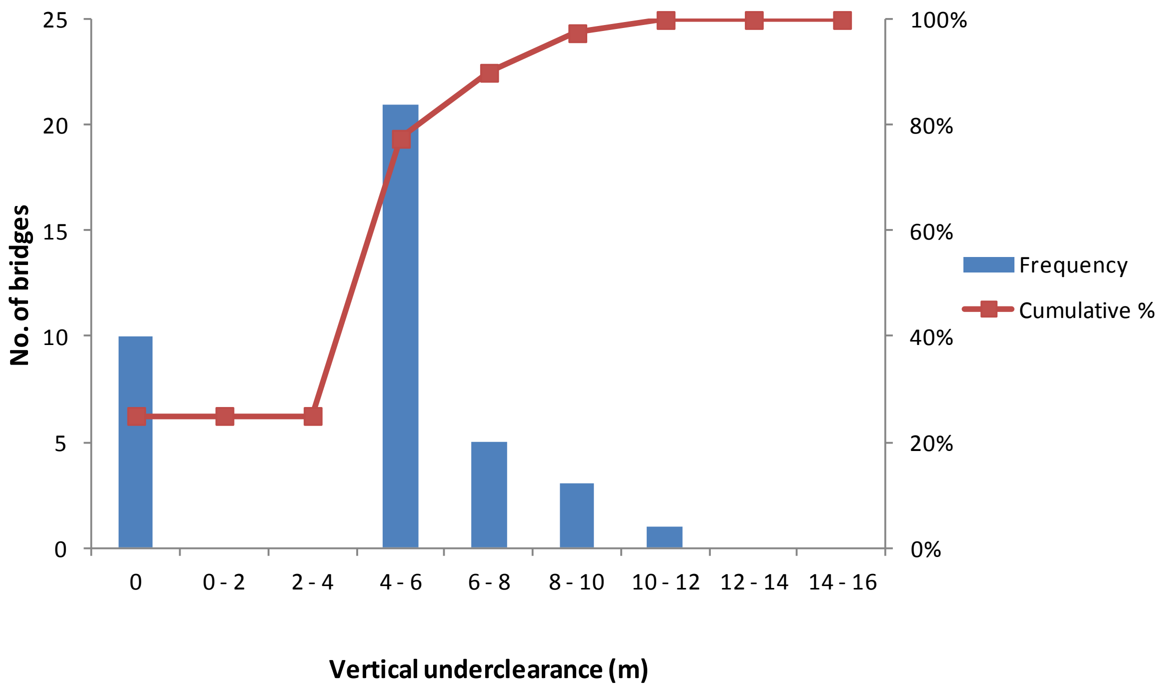

Figure 3.11. Variation in vertical underclearances for state-maintained bridges (truss, suspension or stayed girder) with maximum span lengths > 40 m.

3.3. Summary

No data were available on Florida bridges regarding damage during tornadoes but the literature review and some damages observed under hurricanes suggests that, as shown in Tables 3.2 and 3.3, some elements and bridge types are more vulnerable than others. For instance, signs, railings, and movable bridge elements are very vulnerable. Also truss bridges and/or narrow deck bridges with high underclearances are very vulnerable to severe damage under tornadoes and strong winds.

Table 3.2. Levels of vulnerability of bridge elements to damage from tornadoes and strong winds.

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	2	476	Timber Walls	3
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	1	477	Other Walls	2
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	1	480	Mast Arm Found	4
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	4
31	Timber Deck	2	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	4
32	Timber Deck/AC Ovly	2	234	R/Conc Cap	1	483	Other Mast Arm Vert	4
38	Bare Concrete Slab	1	235	Timber Cap	2	484	Paint Mast Arm Horzn	4
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	1	485	Galvan Mast Arm Horz	4
54	Timber Slab	2	241	Concrete Culvert	1	486	Other Mast Arm Horzn	4
55	Timber Slab/AC Ovly	2	242	Timber Culvert	3	487	Sign Member Horiz	4
98	Conc Deck on PC Pane	1	243	Misc Culvert	2	488	Sign Member Vertical	4
99	PS Conc Slab	1	290	Channel	3	489	Sign Foundation	4
101	Unpnt Stl Box Girder	1	298	Pile Jacket Bare	3	495	Uncoat High Mast L.	4
102	Paint Stl Box Girder	1	299	Pile Jacket/Cath Pro	3	496	Painted High Mast L.	4
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	2	497	Galvan. High Mast L.	4
105	R/Conc Box Girder	1	301	Pourable Joint Seal	2	498	Other High Mast L.P.	4
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	2	499	H. M. L. P. Found.	4
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	2	540	Open Gearing	3
109	P/S Conc Open Girder	1	304	Open Expansion Joint	2	541	Speed Reducers	3
110	R/Conc Open Girder	1	310	Elastomeric Bearing	2	542	Shafts	3
111	Timber Open Girder	2	311	Moveable Bearing	2	543	Shaft Brgs and Coupl	3
112	Unpnt Stl Stringer	3	312	Enclosed Bearing	2	544	Brakes	3
113	Paint Stl Stringer	2	313	Fixed Bearing	2	545	Emergency Drive	3
115	P/S Conc Stringer	2	314	Pot Bearing	2	546	Span Drive Motors	3
116	R/Conc Stringer	2	315	Disk Bearing	2	547	Hydraulic Power Unit	3
117	Timber Stringer	3	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	3
120	U/Stl Thru Truss/Bot	4	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	3
121	P/Stl Thru Truss/Bot	4	330	Metal Rail Uncoated	4	550	Hopkins Frame	3
125	U/Stl Thru Truss/Top	4	331	Conc Bridge Railing	4	560	Locks	3
126	P/Stl Thru Truss/Top	4	332	Timb Bridge Railing	4	561	Live Load Shoes	3
130	Unpnt Stl Deck Truss	4	333	Other Bridge Railing	4	562	Counterweight Suppor	3
131	Paint Stl Deck Truss	4	334	Metal Rail Coated	4	563	Acc Ladd & Plat	3
135	Timber Truss/Arch	2	356	Steel Fatigue SmFlag		564	Counterweight	3
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	3
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	3
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	3
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	3
145	Other Arch	1	361	Scour Smart Flag		573	PLCs	3
146	Misc Cable Uncoated	4	362	Traf Impact SmFlag		574	Control Console	3
147	Misc Cable Coated	4	363	Section Loss SmFlag		580	Navigational Lights	4
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag		581	Operator Facilities	4
152	Paint Stl Floor Beam	3	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	3
154	P/S Conc Floor Beam	2	386	Fender/Dolphin Uncoa	2	583	Swing Bridge Spec. E	3
155	R/Conc Floor Beam	2	387	P/S Fender/Dolphin	2	590	Resistance Barriers	3
156	Timber Floor Beam	3	388	R/Conc Fender/Dolphi	2	591	Warning Gates	4
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	2	592	Traffic Signals	4
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	2			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	2			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	2			
204	P/S Conc Column	2	395	Timber Abut Slope Pr	3			
205	R/Conc Column	2	396	Other Abut Slope Pro	2			
206	Timber Column	3	397	Drain. Syst Metal	2			
207	P/S Conc Holl Pile	2	398	Drain. Syst Other	2			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	2			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	2			
215	R/Conc Abutment	1	475	R/Conc Walls	2			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Table 3.3. Levels of vulnerability of bridge types to damage from tornadoes and strong winds.

Superstructure design - main unit	Low underclearance	High underclearance
Slab	1	2
Stringer/MultiBeam/Girder	1	2
Girder & Floorbeam	1	2
Tee Beam	1	2
Box Beam or Girders - Multiple	1	2
Box Beam/Girders - Single or Spread	1	2
Frame (except frame culverts)	1	2
Orthotropic	1	2
Truss - Deck	4	5
Truss - Thru	4	5
Arch - Deck	1	2
Arch - Thru	1	2
Suspension	4	4
Stayed Girder	4	4
Movable - Lift	3	4
Movable - Bascule	3	4
Movable - Swing	3	4
Tunnel	2	4
Culvert	2	3
Mixed Types	2	2
Segmental Box Girder	1	2
Channel Beam	1	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

3.4. References

National Oceanic and Atmospheric Administration (NOAA), (2011). National Climatic Data Center, Website: http://www.ncdc.noaa.gov Accessed 2011.

Pierce, C., Jenkins, J., and Joseph, L. (2009) Ground Damage Assessment from Tornado-Borne Projectiles. Forensic Engineering 2009: pp. 709-718.

McKean, (2001). Collapse of the Kinzua Bridge, Accessed May 2011, from http://www.dcnr.state.pa.us/info/kinzuabridgereport/kinzua.html and http://www.dcnr.state.pa.us/stateparks/parks/kinzuabridge.aspx

Monticello, (2011). Collapse of a Penn Central Transportation Company railroad bridge, Accessed May, from (http://www.april31974.com/indiana_tornado_data.htm)

4. Wildfires

Florida is regularly subjected to wildfires. The fires often occur during the intense heat of summer, but can occur at any time during the year. During a drought, even a small spark - such as from a discarded cigarette - can quickly become a massive wildfire, and Florida's frequent, intense lightning storms make conditions even more dangerous.

Fires can easily lead to road closures in Florida. This is due, in part, to the fact that fire is a very important part of Florida's ecosystem; many native plants will burn and regenerate quite easily. In rural areas, huge walls of quickly moving flames and smoke can make roads impassable. Even a distant fire can lead to road closures, as blowing smoke can quickly reduce visibility to just a few feet and make driving extremely dangerous. On January 9, 2008, a thick mix of fog and smoke led to a catastrophic 70-vehicle pileup on Interstate 4 in Polk County.

Muck Fires are common in Florida. In much of Florida, the ground beneath the top layer of soil is made up of loose, organic material, which is known as "muck." When a fire on the surface burns down into the muck, the organic material can ignite, producing a stubborn, smelly blaze. This blaze, a muck fire, can leave embers smoldering underground long after the surface fire has been extinguished. The muck fire can spread underground, burning tree roots, and making trees unstable. The trees can then fall on firefighters or other people in the area. Containing a muck fire can be very difficult for firefighters. Falling trees and destabilized ground can make it nearly impossible to bring firefighting equipment into a forested area where a muck fire is burning. Also, in order to extinguish a muck fire, the ground must be thoroughly soaked. This can require the earth to be turned over so that water can better reach burning areas of muck.

Smoke is likely to be the major source of disruption to a road and bridge network, as it will cover a much larger area than the actual fire itself. Even if a fire lasts for a duration of weeks, it is unlikely a bridge network will be affected for the total length of this time as the fire front will constantly be moving and the wind direction will change throughout the course of the fire. This means that, while multiple roads could be closed in the course of a fire, it is unlikely that these closures will coincide with each other, and hence the impact of wildfires will, in most areas, be minimized by the availability of alternative routes. Fire may also melt and burn the surface of the road and damage roadside furniture such as delineators, signs, barriers and more. The level of service provided will be reduced accordingly until repairs and replacement are complete, but the duration of total closure is likely to be small relative to other hazards.

4.1. Risk assessment: likelihood estimates for wildfires

A detailed set of historical wildfires with date range of 1980 to 2010 was obtained from the Florida Department of Forestry. The data were available in both GIS shapefile formats and databases. Each wildfire record indicates important information, including the date, fuel type, and fire cause. Using ESRI ArcGIS 9.3, simple spatial join commands were used in the ArcToolBox to query the overlay between the Wildfire GIS layer and the Florida DOT bridge GIS layer. Using a buffer of 1 mile, the recorded locations of wildfire near each bridge were identified, and the bridge was assigned the wildfire event and its attributes. The 1-mile value as buffer was chosen arbitrarily. Sample data for the wildfire records are shown in Table 4.1.

As shown in Figure 4.1, three bridges (Structure Nos. 370016, 370013, and 370027) located in Suwannee county have the enclosed wildfire locations as indicated. The selected fire spots are highlighted. It should be noted that some of these fire locations have repeated information, i.e., multiple incidents for one point. A detailed list has been generated indicating for each bridge the assigned risk of flood has described above.

Table 4.1. Sample data for wildfire occurrence near bridge locations.

ROADWAY	ROAD_SIDE	STRUCTURE_	YEAR	START_DATE	FUEL_TYPE	FIRE_CAUSE
01010000	C	010045	1981	19810326	7	5
01010101	C	010050	1981	19810603	1	1
01075000	L	010059	1981	19810111	1	5
01075000	R	010060	1981	19810111	1	5
01060000	C	010061	1981	19810307	1	5
01060000	C	010061	1981	19811221	5	5
01060000	C	010062	1981	19810321	1	5
01060000	C	010063	1981	19810321	1	5
01511000	C	010066	1981	19810207	5	5
01511000	C	010066	1981	19810603	1	1
01040000	L	010067	1981	19810118	1	5
01040000	L	010067	1981	19810203	5	5
01040000	L	010067	1981	19810203	5	5
01040000	R	010068	1981	19810118	1	5
01040000	R	010068	1981	19810203	5	5

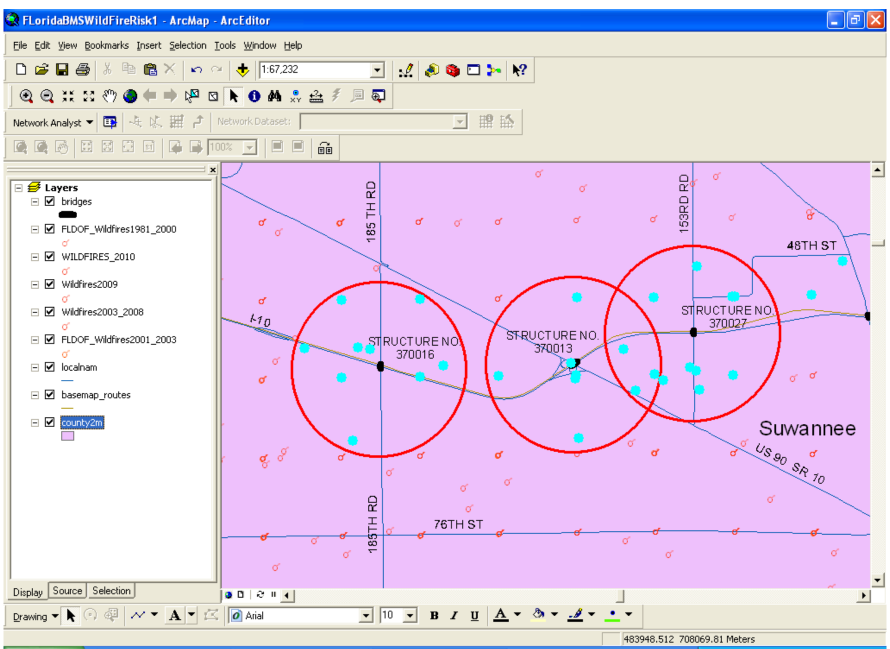

Figure 4.1. Bridges in Suwannee county within one mile of recorded wildfire locations

As described in detail above for the other natural hazards, it will also be assumed that wildfires occur at specific locations according to the Poisson distribution. Thus what is needed is the average annual occurrence rate at each bridge location. From the spatially merged data of the wildfire and bridge GIS layers, the mean rate is obtained by dividing the frequency or total number of occurrences of wildfires near the bridge, by the total duration of the data observation, i.e. 30 years, from 1980 to 2010. As shown in Figures 4.2 and 4.3, it was observed that 538 of the 6637 bridges affected by wildfires historically had experienced more than one such incident per year in the 30 years of observation. There were 16 bridges with more than three wildfires per year during the 30 years while only 3 had more than four per year.

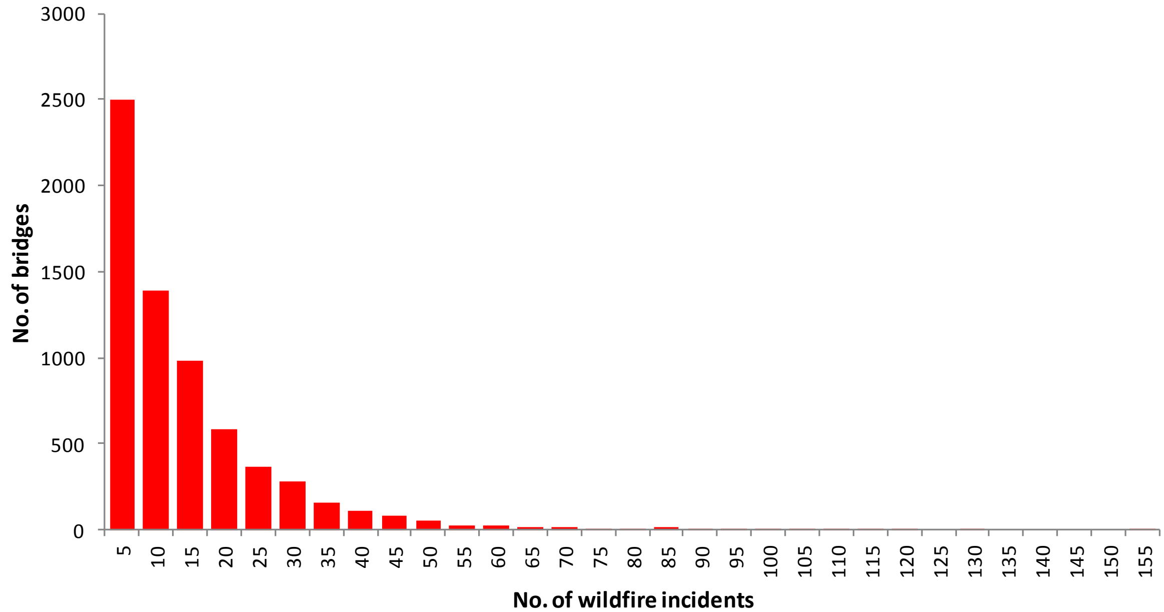

Figure 4.2. Frequency of wildfire incidents near bridge locations (1980 to 2010)

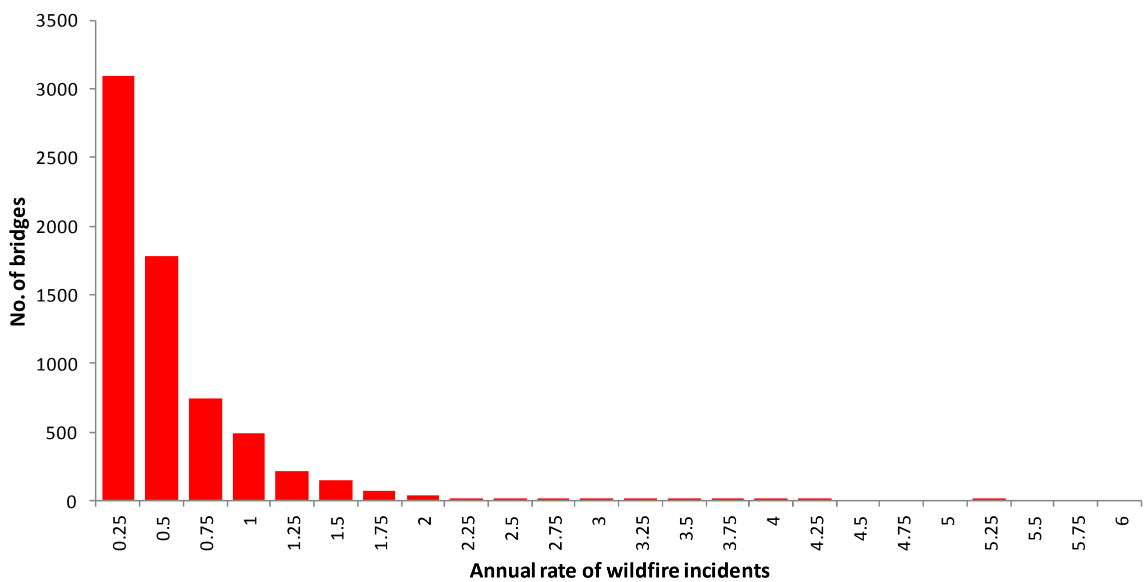

Figure 4.3. Annual rates of wildfire incidents near bridge locations(1980 to 2010)

Using the same methodology applied above to hurricane wind occurrence, the probability of occurrence can be predicted for tornadoes also. Let us consider that for example, the Structure ID 154403 used for illustration under the hurricane wind analysis. Within the period of 30 years, only one wildfire was recorded to have occurred in the vicinity of this particular bridge location. This implies the mean occurrence rate, or λ = 1/30 or 0.033. According to equation 3.4 the probability that there will be a wildfire at this location within a year is given as

$F_T(1) = P(T \leq 1) = 1 - \exp[-0.033*1]$ or 0.03246.

Also, the long-term probability of having a wildfire, within the next 10 years, at this bridge location is estimated as

$F_T(1) = P(T \leq 10) = 1 - \exp[-0.033*10]$ or 0.2811.

The estimates of the likelihood of wildfire occurrence are summarized here for various categories in Figure 4.4. Looking the skewed distribution of the annual probabilities (Figure 4.4), it is obvious that Florida experiences a lot of wildfire events, with majority of the bridge locations experiencing up to 60% of occurrence every year. The mean annual probabilities of hazard events appear to decrease with increase in the tornado intensity (category).

Another approach taken in estimating the likelihood of wildfire occurrence was using recorded history of wildfire events in each county in Florida (NOAA 2011). It was observed that the data ranged in date from March 4, 1996 to December 21, 2010, implying a range of 14 years. The count of occurrence in each county was determined and divided by the 14 years time span to estimate the mean occurrence rate, or λ as described above. Using the same equations demonstrated above, the probabilities of occurrence of tornadoes within one and 10 years can be estimated.

Estimates of the likelihoods of wildfire occurrence near Florida bridges are presented in more details in Appendix A3. It should be noted that the intensity of the wildfire cannot be readily ascertained form these historical records, so the model indicates just the occurrence of the wildfire event.

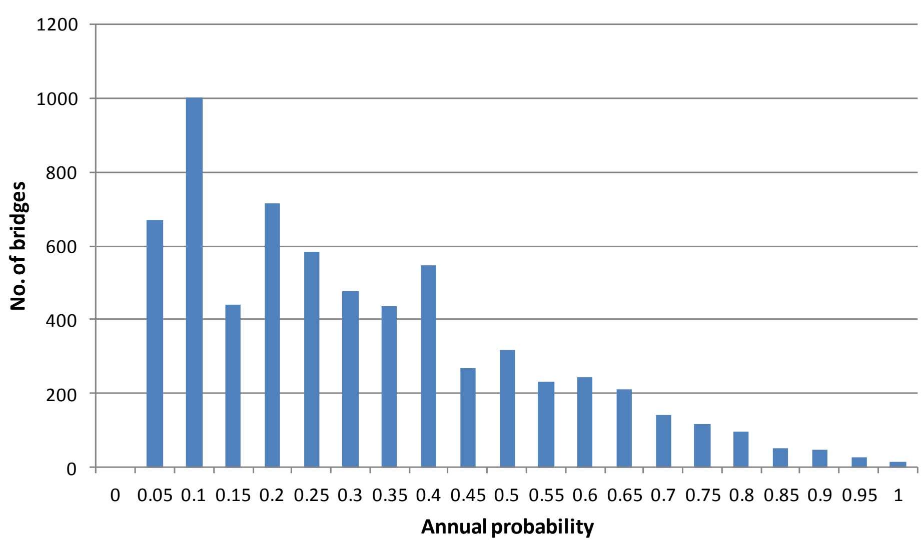

Figure 4.4. Mean annual probabilities of wildfire events at Florida bridge locations

4.2. Risk assessment: consequences of wildfires

When hazards occur on bridges, the consequences consist of damage for which the agency is directly responsible (i.e., physical damage to the bridge, which will require repairs or replacement efforts) and other types of consequences related to public user costs (user delays, vehicle operations, and accidents). But first, it is important to note that the extent of the consequences or impact of hazards on bridges is dependent on attributes related to the bridge and its location. This can be referred to as the vulnerability or susceptibility of the bridge to damage from the particular hazard. If a wildfire engulfs a bridge, the damage that occurs is dependent on the bridge material. Timber is most vulnerable, followed by steel and then concrete.

Since the wildfire intensity is much less than that of fire, say from a fuel tanker explosion, serious damage may be restricted to certain specific bridge elements, such as timber structural and non-structural elements, railings, signs, lightings, and other non-structural elements. The more pronounced impact of wildfire will be in terms of user costs.

According to Unified (2007), during a case of wildfire in Florida in 2007, it was stated that the driving conditions became dangerous in Baker and Columbia counties which had large areas of active wildfires (Figure 4.4). There was deployment of "Smoke on the Road" signs, with several road closures and detours. There were road closures on various roads in impacted areas, including US-441 from I-10 to SR-6 in Columbia County. Other roads that were open but subject to closure included I-75 from Georgia/Florida state border to Lake City, and I-10 from Live Oak to Sanderson. The same report stated that other approach roadways and bridges may be closed, including neighboring detours, especially in areas where visibility-deteriorates rapidly due to smoke.

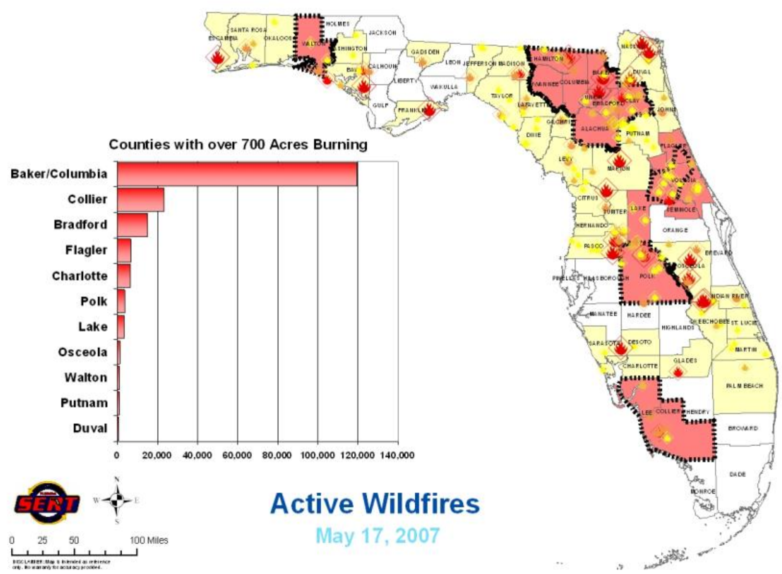

Figure 4.5. Active wildfires in Florida in 2007 (Unified 2007)

Also Morton et al. (2003) presented 10 case studies done on national scope to show the impacts of wildfires. The locations of the case study fire event are shown in Figure 4.5. It was stated in this report that wildfires are very active in Florida. The case discussed for Florida was the Calton fire in Sarasota County in 2001, covering 6000 acres. This event led to the closure of 3.5 mile stretch of I-75 for 12 hours (from late afternoon to early morning), with traffic backups.

The “Double Trouble” fire in New Jersey also led to closure of a 24-mile stretch of the middle of the Garden State Parkway (Toll-road) for 12 hours in 2002, with an estimated direct cost of $15,000, not including lost toll revenue. The Garden State Parkway parallels the New Jersey coastline and carries a large portion of the north-south non-commercial traffic in New Jersey. In June 2002, The Parkway generated toll revenues totaled more than $17.3 million, or nearly 50 million car-uses. It was stated that the timing of the closure, a Sunday afternoon during the summer months, when many vacationers visit the New Jersey Shore, was a major inconvenience.

Another wildfire incident, the Battle Creek fire burned 12,420 acres of the Black Hills National Forest and private lands in South Dakota in 2002. The fire burned within one mile of Mount Rushmore National Memorial, and the possibility of damage to Mount Rushmore elevated Battle Creek to a Class 1 fire and to the number one priority fire in America in August 2002. The fire incident led to the closure of Hwy 16 for 3 days, at a cost to $18,000 to the South Dakota DOT.

This report showed that the structures that are seriously damaged in wildfires are buildings.

Figure 4.6. The ten case study fire locations (Morton et al., 2003)

Though the case studies are limited in terms of the number, it can be reasonably assumed that road closures constitute the most threat to roadways and bridges, rather than physical member damage. It also seems that major roadways (Interstates and Principal arterials) are closed for about 12 hrs to one day due to the typical urgent responses. On the other hand, local roads may take longer to open after closure, say three days or slightly more.

4.3. Summary

Methods were used to estimate the probability of the occurrence of wildfire events near a bridge and also in the county where the bridge is located. Though the model does not indicate the intensity of the wildfire, these events are very common in Florida with many bridges likely to experience it in its vicinity. The consequences of wildfire event will be most severe on local roads in terms of agency and user costs, particularly for timber material bridges.

As shown in Table 4.2, the vulnerability of bridge elements to damage during wildfire vary, with timber elements being most vulnerable, followed by joints, bearings, railings, and movable bridge elements.

Depending on the wildfire intensity, the different bridge design and material types may experience different levels of vulnerability to damage. Table 4.3 shows that timber superstructures are most vulnerable followed by steel structural members. Continuous superstructures may also transmit the effects of the wildfire faster than non-continuous designs.

Table 4.2. Levels of vulnerability of bridge elements to damage from wildfires

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	5	476	Timber Walls	5
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	4	477	Other Walls	4
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	2	480	Mast Arm Found	2
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	2	481	Paint Mast Arm Vert	2
31	Timber Deck	5	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	2
32	Timber Deck/AC Ovly	5	234	R/Conc Cap	1	483	Other Mast Arm Vert	2
38	Bare Concrete Slab	1	235	Timber Cap	5	484	Paint Mast Arm Horzn	2
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	2	485	Galvan Mast Arm Horz	2
54	Timber Slab	5	241	Concrete Culvert	1	486	Other Mast Arm Horzn	2
55	Timber Slab/AC Ovly	5	242	Timber Culvert	5	487	Sign Member Horiz	2
98	Conc Deck on PC Pane	1	243	Misc Culvert	4	488	Sign Member Vertical	2
99	PS Conc Slab	1	290	Channel	2	489	Sign Foundation	2
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	2	495	Uncoat High Mast L.	2
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	2	496	Painted High Mast L.	2
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	4	497	Galvan. High Mast L.	2
105	R/Conc Box Girder	1	301	Pourable Joint Seal	4	498	Other High Mast L.P.	2
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	4	499	H. M. L. P. Found.	2
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	4	540	Open Gearing	3
109	P/S Conc Open Girder	1	304	Open Expansion Joint	4	541	Speed Reducers	3
110	R/Conc Open Girder	1	310	Elastomeric Bearing	4	542	Shafts	3
111	Timber Open Girder	5	311	Moveable Bearing	4	543	Shaft Brgs and Coupl	3
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	4	544	Brakes	3
113	Paint Stl Stringer	2	313	Fixed Bearing	4	545	Emergency Drive	3
115	P/S Conc Stringer	1	314	Pot Bearing	4	546	Span Drive Motors	3
116	R/Conc Stringer	1	315	Disk Bearing	4	547	Hydraulic Power Unit	3
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	3
120	U/Stl Thru Truss/Bot	2	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	3
121	P/Stl Thru Truss/Bot	2	330	Metal Rail Uncoated	3	550	Hopkins Frame	3
125	U/Stl Thru Truss/Top	2	331	Conc Bridge Railing	2	560	Locks	3
126	P/Stl Thru Truss/Top	2	332	Timb Bridge Railing	5	561	Live Load Shoes	3
130	Unpnt Stl Deck Truss	2	333	Other Bridge Railing	4	562	Counterweight Suppor	3
131	Paint Stl Deck Truss	2	334	Metal Rail Coated	3	563	Acc Ladd & Plat	3
135	Timber Truss/Arch	5	356	Steel Fatigue SmFlag		564	Counterweight	3
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	3
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	3
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	3
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	3
145	Other Arch	4	361	Scour Smart Flag		573	PLCs	3
146	Misc Cable Uncoated	2	362	Traf Impact SmFlag		574	Control Console	3
147	Misc Cable Coated	2	363	Section Loss SmFlag		580	Navigational Lights	3
151	Unpnt Stl Floor Beam	2	369	Sub.Sect Loss SmFlag		581	Operator Facilities	3
152	Paint Stl Floor Beam	2	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	3
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	2	583	Swing Bridge Spec. E	3
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	2	590	Resistance Barriers	3
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	1	591	Warning Gates	3
160	Unpnt Stl Pin/Hanger	2	389	Timber Fender/Dolphi	5	592	Traffic Signals	3
161	Paint Stl Pin/Hanger	2	390	Other Fender/Dolphin	4			
201	Unpnt Stl Column	2	393	Blkhd Sewl Metal Unc	1			
202	Paint Stl Column	2	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	5			
205	R/Conc Column	1	396	Other Abut Slope Pro	4			
206	Timber Column	5	397	Drain. Syst Metal	3			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	3			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	4			
211	Other Mtl Pier Wall	4	474	Walls Uncoated	4			
215	R/Conc Abutment	1	475	R/Conc Walls	1			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Table 4.3. Levels of vulnerability of bridge types to damage from wildfires

Superstructure design type	Concrete	Concrete Continuous	Steel	Steel Continuous	Prestressed concrete	Prestressed concrete continuous	Wood or Timber	Masonry	Aluminum, Wrought Iron, or Cast Iron
Slab	3	3			3	3	5		
Stringer/MultiBeam/Girder	3	3	4	4	3	3	5		
Girder & Floorbeam	3	3	4	4	3	3	5		
Tee Beam	3	3	4	4	3	3			
Box Beam or Girders - Multiple	3	3			3	3			
Box Beam/Girders - Single or Spread	3	3			3	3			
Frame (except frame culverts)									
Orthotropic									
Truss - Deck			4	4					
Truss - Thru			4	4					
Arch - Deck								3	
Arch - Thru								3	
Suspension			4	4					
Stayed Girder			4	4					
Movable - Lift	3	3	4	4					
Movable - Bascule	3	3	4	4					
Movable - Swing	3	3	4	4					
Tunnel									
Culvert									
Mixed Types									
Segmental Box Girder	3	3			3	3			
Channel Beam			4	4					

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); min of 1 assigned to bridge types with estimated zero level of vulnerability. Also assumes hurricane categories 3 and above.

* some cells are blank because the bridge type will not exist, e.g., there is no Masonry suspension bridge.

4.4. References

Douglas C. Morton, Megan E. Roessing, Ann E. Camp, and Mary L. Tyrrell, (2003). Assessing the Environmental, Social, and Economic Impacts of Wildfire, GISF Research Paper 001, Global Institute of Sustainable Forestry, Forest Health Initiative, School of Forestry & Environmental Studies, Yale University.

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5. Floods

Because so much of Florida is at or near sea level, flooding is a common problem. Even a minor flood can be a disaster for the people who are forced to cope with it. Quickly-rising water can cause millions of dollars of damage to homes and businesses. Flooding is caused by factors as discussed in the following paragraphs.

Hurricanes and tropical storms can bring significant storm surges, which can flood large areas. Even a Category 2 hurricane brings a storm surge of at least six feet above normal. A major hurricane can bring a storm surge of twice that, or even more. For Floridians who live only a few feet above sea level, this can be a major problem. If a storm surge occurs at high tide, the effects can be devastating.

Florida regularly experiences strong thunderstorms, especially in summer, leading to heavy rains. These storms can produce great amounts of rain very quickly. If drainage systems are unable to keep up with the water levels, flooding occurs. Even when there is adequate drainage in one area, the water may flow into a river, and the sudden rise in the river level may flood a different area.

Florida's low-lying, populated areas are often divided into flood zones. These zones detail which places are likely to be affected by different categories of hurricanes and tropical storms and when residents should be ready to evacuate. Emergency-management officials use these zones to simplify the incredibly difficult process of hurricane evacuations.

Bridges on the Florida State Highway network are prone to several different modes of failure during a flooding event. These include collapse caused by scouring of the piers, hydraulic loading on the piers, hydraulic loading on the bridge deck, erosion of the abutments, as well as debris such as large trees or logs carried by the flood waters striking the piers. Although bridge designs incorporate features to reduce the chances of failure, designing bridges to withstand the most extreme flooding events is not generally a viable option. The uncertainty that surrounds return periods for different flooding events means that it is possible that the designer of a bridge ensured that the bridge could sustain floods that were believed to have a return period of 100 years where in reality the return period could be much shorter.

5.1. Risk assessment: likelihood estimates for floods

Two sources were utilized for the acquisition of Flood GIS data: the FEMA Map office and the Florida Geographic Data Library (FGDL). The FGDL is a collection of Geospatial Data compiled by the University of Florida GeoPlan Center with support from the Florida Department of Transportation (FGDL 2011). GIS data available in FGDL are collected from various state, federal, and other agencies (data sources) who are data stewards, producers, or publishers. According to metadata described in FGDL (2011), the flood GIS dataset contains information about the flood hazards within zones used by the Federal Emergency Management Agency (FEMA) to designate the Special Flood Hazard Area (SFHA) and for insurance rating purposes. These data represent the flood hazard areas that are or will be depicted on the Flood Insurance Rate Map (FIRM). There is one polygon for each contiguous flood zone designated. This information is required for all draft Digital Flood Insurance Rate Maps. The Digital Flood Insurance Rate Map (DFIRM) Database depicts flood risk information and supporting data used to develop the risk data. The primary risk classifications used are the 1-percent-annual-chance flood event (100 year), the 0.2-percent-annual-chance flood event (500 year), and areas of minimal flood risk. The DFIRM Database is derived from the following: Flood Insurance Studies (FISs), previously published

Flood Insurance Rate Maps (FIRMs) – both published by FEMA, flood hazard analyses performed in support of the FISs and FIRMs, and new mapping data, where available.

Also, the FIRM is the basis for floodplain management, mitigation, and insurance activities for the National Flood Insurance Program (NFIP). Insurance applications include enforcement of the mandatory purchase requirement of the Flood Disaster Protection Act, which "... requires the purchase of flood insurance by property owners who are being assisted by Federal programs or by Federally supervised, regulated or insured agencies or institutions in the acquisition or improvement of land facilities located or to be located in identified areas having special flood hazards," Section 2 (b) (4) of the Flood Disaster Protection Act of 1973. In addition to the identification of Special Flood Hazard Areas (SFHAs), the risk zones shown on the FIRMs are the basis for the establishment of premium rates for flood coverage offered through the NFIP. The DFIRM Database presents the flood risk information depicted on the FIRM in a digital format suitable for use in electronic mapping applications. The DFIRM database is a subset of the Digital FIS database that serves to archive the information collected during the FIS.

Limited by availability from FEMA, the data is restricted to the following counties: Alachua, Baker, Bay, Charlotte, Columbia, Dixie, Escambia, Flagler, Gadsden, Gilchrist, Gulf, Hamilton, Hillsborough, Holmes, Jackson, Lafayette, Lake, Lee, Leon, Madison, Marion, Miami-Dade, Monroe, Nassau, Okaloosa, Orange, Osceola, Pinellas, Santa Rosa, Seminole, St. Johns, Suwannee, Taylor, Union, Volusia, and Walton.

The most important information in the GIS attribute database table is the designated field “FLD_ZONE” which is coded by FEMA to imply the risk level of the particular zone, in terms of the percentage probability of flooding, as shown in Table 5.1.

Based on these Flood zone designations, the FGDL dataset added a new field to indicate the flood risks as follows: High risk - coastal areas (Zones V, VE, and V1 – 30; High risk areas (Zones A, AE, A1-A30, AH, AO, AR, and A99); Moderate to low risk areas (Zones B, C, and X, with insurance purchase not required in these zones); and Undetermined (Zone D).

The flood in the high risk zones are 100-year floods, i.e., their annual rate of occurrence is 1/100 or 1%. The moderate risk floods are 500-year floods, or their annual rate is 0.2% while the moderate-to-low-risk zones are considered to be outside the flood plains, or in other words, have zero annual rate of occurrence.

Using ESRI ArcGIS 9.3, simple spatial join commands were used in the ArcToolBox to query the overlay between the flood GIS layer and the Florida DOT bridge GIS layer. The intersection of the layers yielded the bridges lying in each of the listed risk categories. The result was produced in terms of Structure ID, roadway ID, Roadside (C, L, or R), and county of location. A detailed list has been generated indicating for each bridge the assigned risk of flood as described above.

Table 5.1. Assignment of flood zone descriptions

FLD_ZONE	Description
A	Zone A is the flood insurance rate zone that corresponds to the 1-percent annual chance floodplains that are determined in the Flood Insurance Study by approximate methods of analysis. Because detailed hydraulic analyses are not performed for such areas, no Base Flood Elevations or depths are shown within this zone. Mandatory flood insurance purchase requirements apply.
AE and A1-A30	Zones AE and A1-A30 are the flood insurance rate zones that correspond to the 1-percent annual chance floodplains that are determined in the Flood Insurance Study by detailed methods of analysis. In most instances, Base Flood Elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone. Mandatory flood insurance purchase requirements apply.
AH	Zone AH is the flood insurance rate zone that corresponds to the areas of 1-percent annual chance shallow flooding with a constant water-surface elevation (usually areas of ponding) where average depths are between 1 and 3 feet. The Base Flood Elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone. Mandatory flood insurance purchase requirements apply.
AO	Zone AO is the flood insurance rate zone that corresponds to the areas of 1-percent shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average flood depths derived from the detailed hydraulic analyses are shown within this zone. In addition, alluvial fan flood hazards are shown as Zone AO on the Flood Insurance Rate Map. Mandatory flood insurance purchase
AR	Zone AR is the flood insurance rate zone used to depict areas protected from flood hazards by flood control structures, such as a levee, that are being restored. FEMA will consider using the Zone AR designation for a community if the flood protection system has been deemed restorable by a Federal agency in consultation with a local project sponsor; a minimum level of flood protection is still provided to the community by the system; and restoration of the flood protection system is scheduled to begin within a designated time period and in accordance with a progress plan negotiated between the community and FEMA. Mandatory purchase requirements for flood insurance will apply in Zone AR, but the rate will not exceed the rate for an unnumbered Zone A if the structure is built in compliance with Zone AR floodplain management regulations.
A99	Zone A99 is the flood insurance rate zone that corresponds to areas within the 1-percent annual chance floodplain that will be protected by a Federal flood protection system where construction has reached specified statutory milestones. No Base Flood Elevations or depths are shown within this zone. Mandatory flood insurance purchase requirements apply.
D	The Zone D designation is used for areas where there are possible but undetermined flood hazards. In areas designated as Zone D, no analysis of flood hazards has been conducted. Mandatory flood insurance purchase requirements do not apply, but coverage is available. The flood insurance rates for properties in Zone D are commensurate with the uncertainty of the flood risk.
V	Zone V is the flood insurance rate zone that corresponds to areas within the 1-percent annual chance coastal floodplains that have additional hazards associated with storm waves. Because approximate hydraulic analyses are performed for such areas, no Base Flood Elevations are shown within this zone. Mandatory flood insurance purchase requirements apply.
VE	Zone VE is the flood insurance rate zone that corresponds to areas within the 1-percent annual chance coastal floodplain that have additional hazards associated with storm waves. Base Flood Elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone. Mandatory flood insurance purchase requirements apply.
B, C, and X	Zones B, C, and X are the flood insurance rate zones that correspond to areas outside the 1-percent annual chance floodplain, areas of 1-percent annual chance sheet flow flooding where average depths are less than 1 foot, areas of 1-percent annual chance stream flooding where the contributing drainage area is less than 1 square mile, or areas protected from the 1-percent annual chance flood by levees. No Base Flood Elevations or depths are shown within this zone. Insurance purchase is not required in these zones.
0.2 pct annual chance flood hazard	Areas with a 0.2% annual chance of flooding (500 year).

The flood data has already being classified into risk categories by virtue of the flood zones that the bridge falls. Sample data are shown in Tables 5.2 to 5.4.

Table 5.2. Sample data for flood high risk level for bridge locations

ROADWAY	ROAD_SIDE	STRUCTURE_	COUNTY	FLOODPLAIN	RISK_LEVEL	DESCRIPT
26040000	C	260006	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26530500	C	264138	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26000013	C	262501	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26250000	C	260940	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26506001	C	264875	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26260000	R	260081	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26080000	R	260103	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26508000	C	260016	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26110000	C	260025	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26000001	C	264146	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26060000	C	260045	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA
26510000	C	260051	ALACHUA	100-YEAR FLOODPLAIN	HIGH RISK AREAS	INSIDE SPECIAL FLOOD HAZARD AREA

Table 5.3. Sample data for flood moderate risk level for bridge locations

ROADWAY	ROAD_SIDE	STRUCTURE_	FLOODPLAIN	COUNTY	RISK_LEVEL	DESCRIPT
01000078	C	014118	500-YEAR FLOODPLAIN	CHARLOTTE	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10130000	L	100068	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270004	C	100087	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10140000	C	100094	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270000	C	100095	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270000	C	100294	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10190009	C	100295	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270000	C	100297	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10130001	R	100300	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10150000	C	100335	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10130001	L	100585	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270113	C	100674	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10270114	C	100675	500-YEAR FLOODPLAIN	HILLSBOROUGH	MODERATE RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA

Table 5.4. Sample data for flood moderate-to-low risk level for bridge locations

ROADWAY	ROAD_SIDE	STRUCTURE_	FLOODPLAIN	COUNTY	RISK_LEVEL	DESCRIPT
01075000	L	010069	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	R	010070	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	L	010071	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	R	010072	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	L	010073	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	R	010074	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	L	010082	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01075000	R	010083	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01000089	C	014103	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
01530000	C	014115	OUTSIDE FLOODPLAIN	CHARLOTTE	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
02010000	C	020026	OUTSIDE FLOODPLAIN	MARION	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA
10040000	C	100001	OUTSIDE FLOODPLAIN	HILLSBOROUGH	MODERATE TO LOW RISK AREAS	OUTSIDE SPECIAL FLOOD HAZARD AREA

Below in Table 5.5 is the summary of all the bridges by county, with indication of the risk levels. The annual occurrence rate λ is also shown. Appendix A4 shows more details on the estimates of flooding likelihood on Florida bridges. As demonstrated earlier for hurricanes and tornadoes, the events can be assumed to be a Poisson process and the probability of future occurrence can be predicted using the equations presented.

Table 5.5. Summary of bridges location by county in flood risk categories

COUNTY	NO. OF BRIDGES IN RISK CATEGORY High-Risk ($\lambda = 0.01$)	High-Risk Coastal ($\lambda = 0.01$)	Moderate-Risk ($\lambda = 0.02$)	Moderate-To-Low-Risk ($\lambda = 0.0$)
ALACHUA	32	0	2	34
BAKER	34	0	1	35
BAY	72	8	3	20
CHARLOTTE	64	4	1	16
COLUMBIA	37	0	0	37
DIXIE	35	6	0	4
ESCAMBIA	85	6	4	108
FLAGLER	28	0	5	25
GADSDEN	57	0	1	37
GILCHRIST	4	0	1	0
GULF	31	1	0	4
HAMILTON	24	0	1	17
HILLSBOROUGH	259	2	18	419
HOLMES	126	0	7	59
JACKSON	80	0	5	17
LAFAYETTE	13	0	0	0
LAKE	36	0	0	59
LEE	92	13	17	93
LEON	55	0	3	57
MADISON	31	0	3	18
MARION	23	0	3	46
MIAMI-DADE	620	10	27	534
MONROE	61	40	5	2
NASSAU	45	1	12	38
OKALOOSA	85	3	2	48
ORANGE	105	0	3	508
OSCEOLA	85	0	8	81
PINELLAS	151	32	31	115
SANTA ROSA	103	11	7	55
SEMINOLE	22	0	9	80
ST. JOHNS	47	1	4	32
SUWANNEE	21	0	0	18
TAYLOR	46	4	1	10
UNION	19	0	0	5
VOLUSIA	88	0	16	69
WALTON	101	1	0	42
TOTALS	2817	143	200	2742

5.2. Risk assessment: consequences of floods

When hazards occur on bridges, the consequences consist of damage for which the agency is directly responsible (i.e., physical damage to the bridge, which will require repairs or replacement efforts) and other types of consequences related to public user costs (user delays, vehicle operations, and accidents). This section will address only the agency costs-related consequences. But first, it is important to note that the extent of the consequences or impact of hazards on bridges is dependent on attributes related to the bridge and its location. This can be referred to as the vulnerability or susceptibility of the bridge to damage from the particular hazard. For instance, bridges located on the coast are more vulnerable to damage from flooding, hurricane storm surge and wave loading than those bridges located inland, away from the coasts. This measure of vulnerability can be used to enhance the overall estimate of risk of the bridge to a particular hazard. In the examples given above, a bridge that is already very vulnerable will have a higher overall risk than another (less vulnerable) bridge that is exposed to the same likelihood estimate of occurrence of the particular hazard.

5.2.1. Lessons learned from previous floods

According to the U.S. Geological Survey (USGS), flooding causes more death and property damage than any other natural occurring hazard, with three-quarters of all federal disaster declarations due at least in part to a flooding event. Bridges are prone to several different failure modes during a flood. They include collapse caused by scouring of the piers, hydraulic loading on the piers, hydraulic loading on the bridge deck, erosion of the abutments, as well as debris such as large trees or logs carried by the flood waters striking the piers (Seville and Metcalfe ,2005) (Figure 5.1). Due to the effects of flooding, it is expected that coastal bridges will experience higher consequences of flooding than mainland bridges.

Figure 5.1. The Ashhurst Bridge, Manawatu, New Zealand (Seville and Metcalfe, 2005)

According to Stearns and Padgett (2011), Hurricane Ike landed on September 13, 2008 in Texas, causing severe damage to the infrastructure in Houston/Galveston area. Data were collected for timber and major bridges from post-assessment surveys and reconnaissance reports, and a study of the failure modes of structures. Generally, the damage is described as being due to the following processes: inundation of bridge decks and superstructures; debris impact; erosion of abutment support; and erosion of approaches. Most severely damaged were timber bridges with low clearance over water. Of the 53 damaged bridges evaluated, 26 were observed to be damaged due to storm surge and wave loading, 25 bridges experienced scour around abutments and wing walls, while impact damage from debris affected four bridges. Peak storm surge level was found to be greater than 14 feet and wave heights more than 5 feet at most damaged locations. It was also observed that 17 rural bridges located inland, away from the surge zones, also experienced damage, primarily in the form of scour and impact damage. This is due to increased water flow rates and flooding.

From a similar report on Hurricane Katrina by Padgett et al. (2008), it was stated that, among other noted damages, there was inundation of electrical and mechanical equipment, primarily affecting movable bridges. There was damage on submerged electrical and mechanical equipment because they were not designed for extended wetting or immersion in rushing flood water. Gilberto et al. (2007) reported that during Katrina, fast-moving water undermined bridge piers and caused flooding of mechanical and control rooms of moveable bridges, and that flooding on movable bridges may lead to highly conductive and corrosive salt water damaging electrical and mechanical bridge controls, leaving them stuck either in the open or closed position.

In a flooding event, waterborne debris including trees, bushes, and occasionally manmade materials often accumulate on bridges or make direct impact (force) on the bridge elements. Shapes and sizes of debris accumulation can vary from a small cluster of debris on a bridge pier to a near complete blockage of the bridge waterway opening (Lagasse et al., 2010). These accumulations cause obstruction, constrictions, redirection of flow though the bridge openings or excessive scour at bridge foundations.

There are too many variables that have to be considered to be able to get an accurate probability of a bridge being impacted by debris in the event of a flood. They include the magnitude of the flood, the location of the bridge, the amount of grounded objects around the bridge, the weight of the objects around the bridge, and lots more. This makes it almost impossible to be able to predict the possibility of debris impact on a bridge. Due to the excessive number of variables, debris influence factors will solely be based on the bridge features.

All debris impact predictions take into account the width of a pier as a factor that contributes to likelihood of a pier being hit by debris. The factor is because a larger width is more prone than a smaller one. Another factor that can contribute to failure of a pier by debris impact is spacing between the piers. A smaller space is more likely to cause large debris to be trapped between the piers; this in turn increases the resistance of water flow, ultimately resulting in higher damage to the pier. Based on this assumption it can be concluded that bridges with longer spans will have a smaller probability of having debris strike the piers that bridges with smaller span lengths. For example, a 400 meters long bridge with five spans and four columns will have a higher likelihood of debris impact than a 400 meters long, four-span bridge with three columns. Using the total length of the bridge divided by the total number of spans, the average length of the spans can be determined. It can be arbitrarily assumed that bridges with average clear spans less than 20 m will considered more susceptible than those with spans more than 20 m. Thus bridges with average spans of 20 meters and less may be considered as having influence factor of 1, while those with lengths greater than 20 meters are assigned influence factors of 0.8.

Analysis of the current Pontis database for Florida's state-maintained bridges (owner codes 1, 31, and 33) was done to evaluate the variation in the span lengths. These are bridges coded as being over any form of waterway, including highway-waterway, railroad-waterway, etc. (NBI Item No. 42 – service under). The number of approach spans and main spans were added together to get the total number of spans. Then the bridge length was divided by the number of spans to estimate an average span length for the bridge. The variation in the overall data for average span lengths is shown in Figure 5.2. It could be seen that about 83% of the bridges have an average span less than or equal to 20 m, and about 55% have an average span length of 10 m or less. This indicates that debris impact potential may be a major concern on Florida state-maintained bridges, especially for those in the coastal areas or riverine areas with high likelihoods of hurricane and flooding.

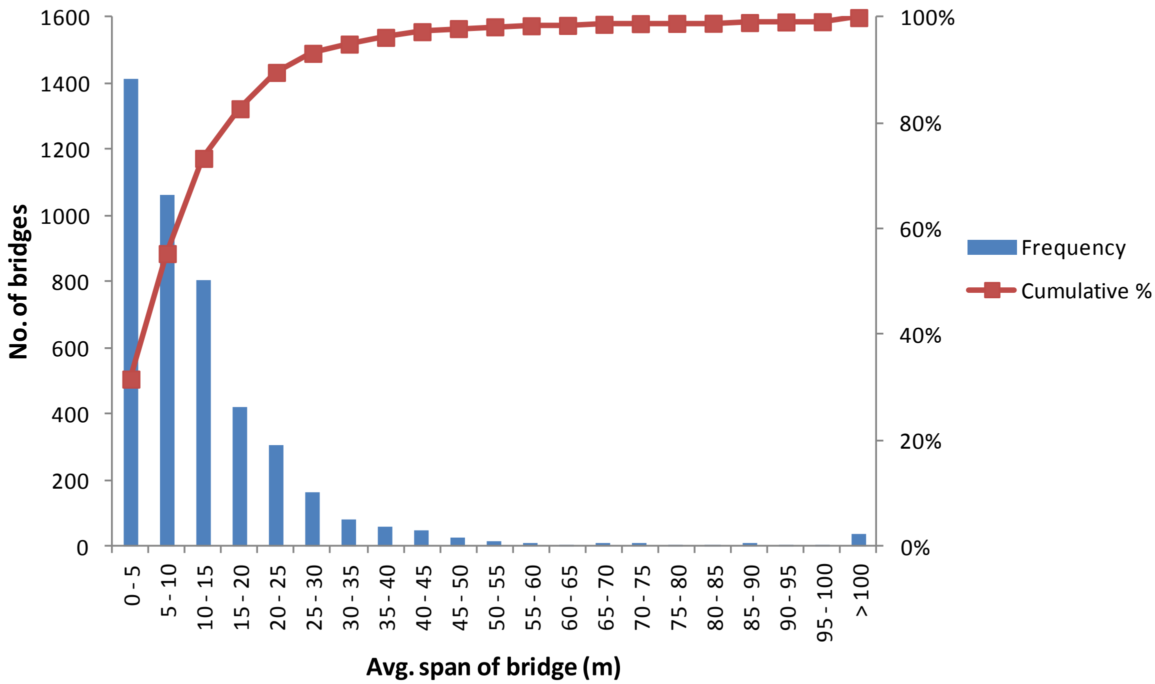

Figure 5.2. Variation in average span lengths of Florida's state-maintained bridges

5.3. Summary

Estimating the likelihood of flooding is most accurately done on a bridge-by-bridge basis, where the detailed hydrology and hydraulics data for the specific location will be critically analyzed. For the objectives of this research, use of the FEMA flood data may be adequate, with the results presented above. The consequences of flooding could not be well quantified in this study due to lack of historical data on flooding effects on bridges. But the observed damage on Florida bridges due to hurricanes can be partially used to infer the damage expected from flooding.

As shown in Table 5.6, the vulnerability of bridge elements to damage during flooding vary, with channel elements being most vulnerable, followed by culverts, approach slabs, slope protection, walls, footings, and movable bridge elements. Based on the average span of the bridge, the vulnerability to damage from flooding may differ. Short spans are more vulnerable than longer spans due to the constriction and faster speed of water flow in the former, with the potential of debris impact also higher in short span bridges. Table 5.7 shows this vulnerability for the various bridge types.

Table 5.6. Levels of vulnerability of bridge elements to damage from flooding

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	3	476	Timber Walls	4
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	3	477	Other Walls	4
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	4	478	MSE Walls	4
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	3	480	Mast Arm Found	2
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	3	481	Paint Mast Arm Vert	1
31	Timber Deck	2	233	P/S Conc Cap	3	482	Galvan Mast Arm Vert	1
32	Timber Deck/AC Ovly	2	234	R/Conc Cap	3	483	Other Mast Arm Vert	1
38	Bare Concrete Slab	1	235	Timber Cap	3	484	Paint Mast Arm Horzn	1
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	4	485	Galvan Mast Arm Horz	1
54	Timber Slab	2	241	Concrete Culvert	4	486	Other Mast Arm Horzn	1
55	Timber Slab/AC Ovly	2	242	Timber Culvert	4	487	Sign Member Horiz	1
98	Conc Deck on PC Pane	1	243	Misc Culvert	4	488	Sign Member Vertical	1
99	PS Conc Slab	1	290	Channel	5	489	Sign Foundation	2
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	4	495	Uncoat High Mast L.	1
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	4	496	Painted High Mast L.	1
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	1
105	R/Conc Box Girder	1	301	Pourable Joint Seal	3	498	Other High Mast L.P.	1
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	2
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	3	540	Open Gearing	4
109	P/S Conc Open Girder	1	304	Open Expansion Joint	3	541	Speed Reducers	4
110	R/Conc Open Girder	1	310	Elastomeric Bearing	3	542	Shafts	4
111	Timber Open Girder	2	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	4
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	3	544	Brakes	4
113	Paint Stl Stringer	2	313	Fixed Bearing	3	545	Emergency Drive	4
115	P/S Conc Stringer	1	314	Pot Bearing	3	546	Span Drive Motors	4
116	R/Conc Stringer	1	315	Disk Bearing	3	547	Hydraulic Power Unit	4
117	Timber Stringer	2	320	P/S Conc Appr Slab	4	548	Hydraulic Piping Sys	4
120	U/Stl Thru Truss/Bot	2	321	R/Conc Approach Slab	4	549	Hydraulic Cylinders	4
121	P/Stl Thru Truss/Bot	2	330	Metal Rail Uncoated	1	550	Hopkins Frame	4
125	U/Stl Thru Truss/Top	2	331	Conc Bridge Railing	1	560	Locks	4
126	P/Stl Thru Truss/Top	2	332	Timb Bridge Railing	1	561	Live Load Shoes	4
130	Unpnt Stl Deck Truss	2	333	Other Bridge Railing	1	562	Counterweight Suppor	4
131	Paint Stl Deck Truss	2	334	Metal Rail Coated	1	563	Acc Ladd & Plat	4
135	Timber Truss/Arch	2	356	Steel Fatigue SmFlag		564	Counterweight	4
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	4
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	4
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	4
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	4
145	Other Arch	2	361	Scour Smart Flag		573	PLCs	4
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag		574	Control Console	4
147	Misc Cable Coated	3	363	Section Loss SmFlag		580	Navigational Lights	4
151	Unpnt Stl Floor Beam	2	369	Sub.Sect Loss SmFlag		581	Operator Facilities	4
152	Paint Stl Floor Beam	2	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	4
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	3	583	Swing Bridge Spec. E	4
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	3	590	Resistance Barriers	4
156	Timber Floor Beam	2	388	R/Conc Fender/Dolphi	3	591	Warning Gates	4
160	Unpnt Stl Pin/Hanger	2	389	Timber Fender/Dolphi	3	592	Traffic Signals	4
161	Paint Stl Pin/Hanger	2	390	Other Fender/Dolphin	3			
201	Unpnt Stl Column	2	393	Blkhd Sewl Metal Unc	3			
202	Paint Stl Column	2	394	R/Conc Abut Slope Pr	4			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	4			
205	R/Conc Column	1	396	Other Abut Slope Pro	4			
206	Timber Column	2	397	Drain. Syst Metal	3			
207	P/S Conc Holl Pile	3	398	Drain. Syst Other	3			
210	R/Conc Pier Wall	3	399	Other Xpansion Joint	3			
211	Other Mtl Pier Wall	3	474	Walls Uncoated	4			
215	R/Conc Abutment	3	475	R/Conc Walls	4			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Table 5.7. Levels of vulnerability of bridge types to damage from flooding

Superstructure design - main unit	Short average span	Long average span
Slab	4	2
Stringer/MultiBeam/Girder	4	2
Girder & Floorbeam	4	2
Tee Beam	4	2
Box Beam or Girders - Multiple	4	2
Box Beam/Girders - Single or Spread	4	2
Frame (except frame culverts)	4	2
Orthotropic	4	2
Truss - Deck	4	2
Truss - Thru	4	2
Arch - Deck	4	2
Arch - Thru	1	1
Suspension	4	2
Stayed Girder	4	2
Movable - Lift	4	2
Movable - Bascule	4	2
Movable - Swing	4	2
Tunnel	4	2
Culvert	4	2
Mixed Types	4	2
Segmental Box Girder	4	2
Channel Beam	4	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

5.4. References

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6. Scour

The occurrence of scour is associated with hurricanes and floods and there are two approaches to predicting scour at bridge locations. First, an elaborate mechanistic approach is available and well described in the Florida Scour Manual and other publications where the soil properties, hydraulic data, bridge geometric attributes, and other pertinent data are utilized, through various equations to estimate the scour depth. These equations are derived based on both theoretical and laboratory considerations. The other approach is primarily empirical, where bridge inventory data (NBI) are used, with a bit of theoretical considerations, to establish likelihood of scour and the risks. The latter has been made popular by the FHWA, as evidenced in the HYRISK Software and also the Unknown Foundation Procedure manual for Florida bridges. Stein et al. (1999) appeared to be the main author of the HYRISK approach which uses mainly NBI data and some consideration of hydraulic principles, to derive the methodology. As will be discussed in the following sections, a link can be reasonably assumed to exist between the two approaches of using detailed theoretical basis, and the empirical method.

6.1. Risk assessment: likelihood estimates for scour

The approach by Stein et al. (1999) seems to be acceptable by FHWA in estimating the probability of failure due to scour. Though based on some subjective estimates, it is simple and direct approach which goes beyond the estimates of likelihood as described above. It computes the probability of failure based on the flow depth and scour vulnerability, as well as the overtopping frequency and scour vulnerability. By combining the probability of failure with the associated replacement and user costs, an estimate is obtained for the overall risk at the bridge due to scour.

6.1.1. Use of hydraulics data to estimate scour likelihood

An excellent method of estimating the probability of scour is the use of detailed hydrology and hydraulics data. Some historical data on river elevations, basically in the form of hydrographs (gauge heights and discharge) are available for some Florida locations at NOAA’s National Weather Service website and linked USGS websites (e.g., NOAA’s Advanced Hydrologic Prediction Service http://water.weather.gov/ahps/). For example, a recent hydrograph at a gauge location near Lake City and Live Oak in Florida is shown in Figure 6.1 for Suwannee River at White Springs near the I-10 and I-75 highways.

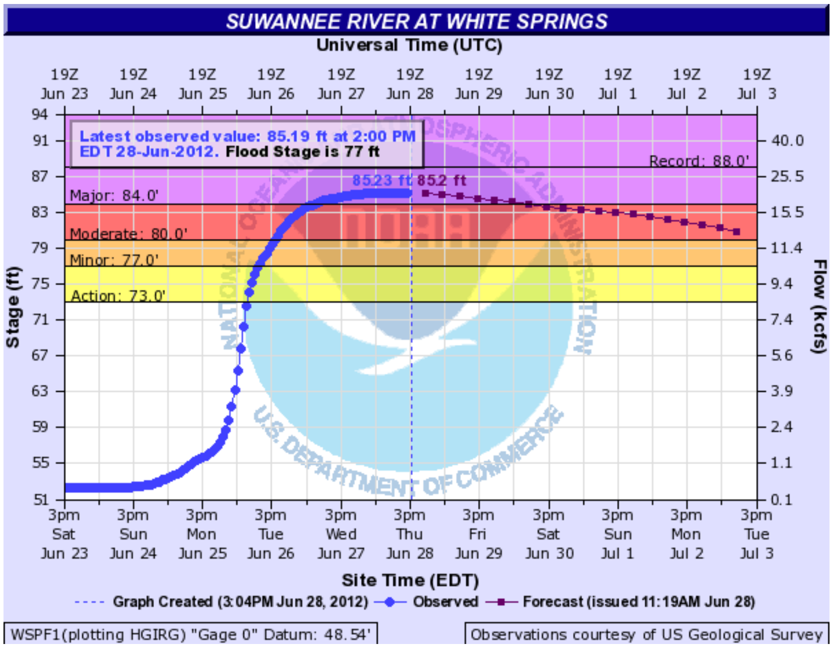


Figure 6.1. Recent hydrograph for Suwannee River at White Springs (NOAA 2012)

The flood impacts of the gauge readings are listed in Table 6.1 with the historical crests in Table 6.2. The gauge datum is 48.54 ft. A comparison of the flood elevation data with the existing bridge deck elevation will indicate the overtopping events at the bridge. In this case, I-75 and the railroad bridge must have been closed in 1973 due to flooding (Table 6.2).

Unfortunately many of the data sites have incomplete data or data that are only provisional and not validated yet. Thus the study cannot go into a detailed approach of estimating overtopping frequency for each bridge using the hydrographs. This type of information can be and are assumed to have been used by the bridge inspectors to assess the overtopping frequency at each bridge location for entry into the NBI Item 71 Waterway adequacy. Moreover, the FDOT Districts and State drainage office will have access to more accurate and complete flood data for assessing the overtopping frequency.

Table 6.1. Flood categories at Suwannee River at White Springs

Flood Category	Flood Stage (ft.)	Flood impacts
Major Flood	88.00	I-75 will be closed.
Major Flood	86.50	The railroad bridge at the gage site floods.
Moderate	81.00	Area known as Suwannee Valley is inundated and secondary roads are closed. Flooding begins at Stephen F. Foster State Park.
Moderate	80.00	Water begins to enter homes and secondary roads become flooded.
Minor	77.00	
Action	73.00	

Table 6.2. Historical crests (gauge heights) at Suwannee River at White Springs

Date	Gauge Height (ft.)	Date	Gauge Height (ft.)
04/30/1928	79.13	04/10/1973	88.02
10/01/1928	82.44	04/10/1984	85.40
01/05/1942	78.78	04/18/1984	81.48
09/25/1945	81.74	02/21/1986	80.67
04/05/1948	85.19	03/20/1991	79.79
03/22/1959	83.15	01/31/1998	77.3
09/17/1964	84.36	02/27/1998	84.86
04/20/1966	79.33	10/03/2004	84.00
09/30/1970	79.44		

Another example is also shown for a recent hydrograph at a gauge location near Tallahassee, Florida in Figure 6.2 for Aucilla River at Lamont near the US 27 roadway. The flood impacts of the gauge readings are listed in Table 6.3 with the historical crests in Table 6.4. In this case, historically, it appears that no major flooding that affected US 27 has occurred but lowland flooding affected houses in 1973, 1985 and 1986 (Table 6.4).

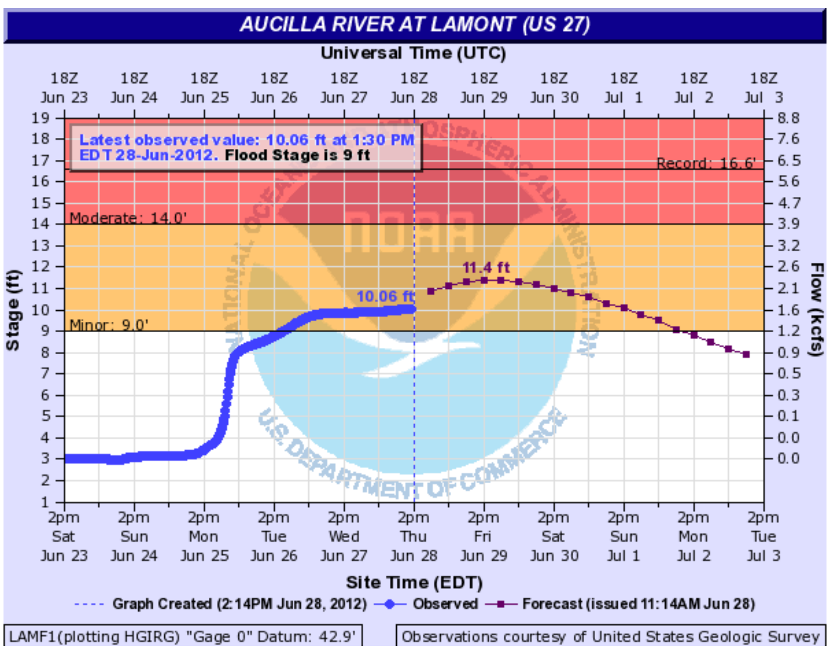


Figure 6.2. Recent hydrograph for Aucilla River at Lamont (NOAA 2012)

Table 6.3. Flood categories at Aucilla River at Lamont

Flood Category	Flood Stage (ft.)	Description
Major Flood	20	Major flood damage will occur. Water will reach U S Highway 27.
Moderate Flood	14	A few houses near U S Highway 27 will flood. Widespread lowland flooding will occur. Water will approach a few houses on U S Highway 27.
Flood	9	Minor lowland flooding begins.

Table 6.4. Historical crests (gauge heights) at Aucilla River at Lamont

Date	Gauge Height (ft.)	Date	Gauge Height (ft.)	Date	Gauge Height (ft.)
09/18/1957	14.90	04/04/1987	12.19	04/16/2003	9.54
03/18/1959	13.00	03/11/1988	11.18	06/09/2003	9.72
04/08/1973	16.57	09/12/1988	9.80	06/23/2003	9.38
12/07/1976	11.10	02/23/1990	10.69	07/05/2003	8.86
03/16/1978	10.60	02/03/1991	13.90	08/18/2003	9.81
03/04/1979	10.00	03/08/1991	14.86	04/08/2005	13.17
04/09/1984	14.53	08/08/1991	11.10	08/26/2008	12.10
09/08/1985	9.48	02/27/1992	9.86		
02/16/1986	13.99	12/20/1995	12.00		

6.1.2. Use of NBI data to estimate scour likelihood

The objective here is to estimate the likelihood of scour at the bridge locations, so the approach to be followed in the study will involve using the NBI Item 71 (Waterway Adequacy), as similarly done in the Stein et al (1999)'s methodology. The NBI data fields are subjective in nature but some of the entries related to scour are based on the bridge engineer/inspectors assessment and/or analyses of the situation, some of which will involve detailed scour analyses. According to the FDOT BMS Coding Guide, the entries for Item 71 are based on historical data and input from the District Drainage Engineer.

NBI Item 71 appraises the waterway opening under the bridge with respect to passage of flow, with regards to chances of overtopping (frequency), relative to the functional classification of the roadways being carried by the bridge, as well as a consideration of possible traffic delays resulting from occurrence of the flooding. The frequencies of bridge overtopping were categorized as follows: Remote (greater than 100 years); Slight (11 to 100 years); Occasional (3 to 10 years); and Frequent (less than 3 years). Stein et al (1999) simplified these return periods to indicate the frequencies, respectively as 100 years, 50 years, 10 years, and 2 years. Table 6.5 shows the definition and explanation of NBI Item 71, including the implied annual probability of each rating.

Using Florida's functional class definition on the roadways, Table 6.5 further illustrates the same information. Except for functional classes 8,9,17, and 19 (Minor collectors and locals), the estimates are the same for all functional classes. It can be argued that without information on the geometrical and hydraulic attributes of the bridge, the functional classes should influence only the vulnerability or consequences of the scour rather than the probability of its occurrence. In other words, it is reasonable to apply one set of probability distribution relative to the waterway adequacy rating of the bridge, i.e.,

P(Overtopping) = {0.50 0.10 0.10 0.10 0.02 0.02 0.02 0.01}

These probabilities can be used to predict the likelihood of bridge scour, though it should be compared with the estimates separately done for flooding and hurricane events at the bridge sites, as these events also cause scour. The main concern is not to double-count these situations.

While the NBI Item 113 (Scour Critical) is very important, it is used to evaluate the vulnerability of the bridge to scour as it expresses the current conditions, but not to estimate the likelihood of scour occurrence. Stein et al (1999) utilized this information to estimate the probability of failure and the overall risk due to scour. Another scour data is available is the *scrrating* field (userbrg table in the Pontis database) which rates the scour at each bridge location according to definition shown in Table 6.7. But again, it should be noted that these ratings are risks, based on a combination of the likelihoods and consequences of the scour occurrence. With focus on just the estimate of likelihood of occurrence of scour, the *scrrating* data can be used as the desired probabilities. The annual probability of bridge overtopping is shown in Table 6.6.

Table 6.5. NBI Item 71 Waterway adequacy and implied annual probability of bridge overtopping

Principal Arterials - Interstates, Freeways, or Expressways	Principal and Minor Arterials and Major Collectors	Minor Collectors and Locals	Description Code	Chance of Overtopping	Annual Probability
N	N	N	Bridge not over a waterway.	N/A	N/A
9	9	9	Bridge deck and roadway approaches above flood water elevations (highwater). Chance of overtopping is remote.	Remote	0.01
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway	Slight	0.02
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.	Slight	0.02
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.	Occasional	0.10
3	4	5	Bridge deck above roadway approaches. Occasional overtopping of roadway approacheswith significant traffic delays.	Occasional	0.10
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic	Occasional	0.10
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic	Frequent	0.50
2	2	2	Occasional or frequent overtopping of bridge deck and roadway approaches with	Frequent	0.50
0	0	0	Bridge closed.		

Table 6.6. Annual probability of flood overtopping at bridge locations

			Waterway Adequacy (NBI Item 71)									
			0	2	3	4	5	6	7	8	9	N
Functional Class (NBI Item 26)	Rural		Bridge Closed									
	Principal Arterial – Interstate	1		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Principal Arterial – Other	2		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Arterial	6		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Major Collector	7		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Collector	8		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Local	9		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Urban											
	Principal Arterial – Interstate	11		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Principal Arterial - Other Freeways or Expressways	12		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Other Principal Arterial	14		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Arterial	16		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Collector	17		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Local	19		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N

Table 6.7. Pontis table userbrg definition for scour rating evaluation (*scrrating* field)

Code	Description
@	Unknown
!	Not applicable
1	Low Risk - Low
2	Low Risk - Medium
3	Low Risk - High
4	Scour Susceptible - Low
5	Scour Susceptible - Medium
6	Scour Susceptible - High
7	Scour Critical
8	Minimal Risk - Notes: Bridge with NBI item 113 coded “U” (Unknown Foundation) and a Lifetime Risk Cost < $15,000 based on Unknown Foundation Procedural Manual.
9	Low Risk Unknown - Notes: Bridge with NBI item 113 coded “U” (Unknown Foundation), and a Lifetime Risk Cost > $15,000, and foundation determined to be Low Risk by calculations performed based on Unknown Foundation Procedural Manual.

6.1.3. Overview of bridge scour likelihood estimates

Applying the methodology described above (Table 6.6) to the Florida bridge inventory, with focus on state-maintained bridges, the estimates of scour likelihoods are summarized in Table 6.8. Further review of the results also shows the list of four bridges with very high estimates of likelihood (50% annual probability) in Table 6.9 while bridges with high estimates (10% annual probability) are shown in Table 6.10. The latter table is illustrated with bridges carrying greater than or equal to 15000 vehicles per day, to emphasize their functional importance. From these summaries it could be observed that most of the bridges (about 70%) have 2% or less chances of overtopping every year or once every 50 years while about 6% have may experience the overtopping once every 10 years. A very few (four) bridges have a serious exposure to the likelihood of bridge overtopping; only one of the bridges have a significant amount of daily traffic going over it.

Table 6.8. Summary of scour (overtopping) likelihood estimates on Florida bridges

Annual Probability of Overtopping	No. of Bridges	% of Bridges
0.01	216	6.5%
0.02	2112	63.3%
0.10	210	6.3%
0.50	4	0.1%
N (Bridge not over waterway)	792	23.7%
Closed	4	0.1%
Totals	3338	

Table 6.9. State-maintained bridges with scour (overtopping) likelihood estimate of 0.5

BRKEY	Feature Intersected	Facility Carried	Functional class	Average Daily Traffic (ADT)	Year of ADT	%ADT Truck	Waterway Adequacy Rating	Scour Critical Rating*	Substructure Rating	Channel Rating	Culvert Rating	Year built	No. of Main Spans	Bridge length
264144	HERRION BAYOU	US98 SR30	09	51	2001	0	2	U	3	3	N	1965	8	60.96
157840	ROCKY CREEK	SR-121	16	18420	2003	5	2	3	6	7	N	1963	2	7.00
045001	SHARP BEND CANAL	US-27 (SR-25)	19	100	1984	0	3	6	3	3	N	1961	2	6.70
726601	B23 CANAL	IRR SR-93 (I-75)SB	19	151	2010	5	3	U	5	6	N	1967	3	36.61

* U -- Unknown foundation

Table 6.10. State-maintained bridges with scour (overtopping) likelihood estimate of 0.1 (ADT ≥ 15000)

BRKEY	Feature Intersected	Facility Carried	Functional class	Average Daily Traffic (ADT)	Year of ADT	%ADT Truck	Waterway Adequacy Rating	Scour Critical Rating*	Substructure Rating#	Channel Rating	Culvert Rating#	Year built	No. of Main Spans	Bridge length
570012	BROWARD RIVER	SR-105 (HECKSCHER)	02	45000	2008	4	5	8	N	8	7	1948	1	180.40
780022	DRAINAGE DITCH	SR91 TPK 134.1	02	17900	2010	2	5	8	N	7	6	1957	3	9.00
010037	AINGER CREEK	SR-776	06	23500	1993	15	5	6	6	6	N	1954	5	22.90
120108	Clay Gully N. Branch	SR56	06	27000	2010	8	5	8	N	7	7	2009	8	16.76
150235	BIG SLOUGH CANAL	I-75 SB	14	59500	2010	3	4	8	7	7	N	1981	9	164.59
160084	MOCCASIN CREEK	SR-2	14	18600	2010	7	5	8	N	6	5	1951	5	38.10
120067	CYPRESS CREEK	I-75 SB	16	23500	1996	5	4	6	N	5	5	1963	3	47.76
157820	YELLOW WATER BRANCH	US-301 (SR-200)	16	18420	2003	5	3	6	7	7	N	1965	2	6.40
480137	East Dog Creek	I10 SR8	16	19750	2010	6	5	8	8	8	N	1970	2	6.31
754014	CANAL C-111	SR 5 / US-1	16	16777	1992	5	5	U	6	8	N	2010	3	71.02
754066	MIA Canal/Okeechobee RD	SR 934 WB (870989)	16	18118	2002	5	5	8	7	7	N	2005	3	138.00
780003	DRAINAGE CANAL	US441-SR15	16	16000	2010	3	4	8	7	6	N	1950	3	13.70
105624	TERRA CEIA BAY	US-19/SR-55SB	17	22000	2009	10	5	8	7	7	N	1954	7	103.00
724073	Spruce Creek N. Relief	US-1 NB	17	21601	2010	50	6	U	7	6	N	2001	7	65.20
724351	Nova Canal	Big Tree Road	17	31531	2010	10	5	8	N	5	6	1988	3	9.45
724379	Nova Road Canal	Sixth St.	19	33121	2004	2	5	8	N	6	6	1959	1	7.32

* U -- Unknown foundation
# N -- Not applicable

6.2. The risk assessment model: consequences of scour

The consequences of scour on the bridge have been reported under the section for hurricanes in this report.

6.3. Summary

Some methods have been presented on estimating the likelihood of scour. Actually, scour is not an event but the consequence of other hazard events such as hurricanes and flooding. So most of the discussions presented under hurricanes and flooding can be applied to the occurrence of scour on the bridges.

6.4. References

Stein, S. M., Young, G. K., Trent, R. E., and Pearson, D. R. (1999). "Prioritizing Scour Vulnerable Bridges Using Risk. Journal of Infrastructure Systems, American Society of Civil Engineers (ASCE, New York, NY.

National Oceanic and Atmospheric Administration (NOAA), (2012). National Weather Service and Advanced Hydrologic Prediction Service. Website: http://water.weather.gov/ahps/
Accessed 2012.

7. Accidents (vehicle or vessel collision) and bridge overloads

As mentioned earlier in this report, there is another set of hazards including those due to impact to the bridge superstructure by vehicles (e.g., fuel tankers) or to bridge substructures by vessels on waterways. These impacts are known to sometimes result in fire hazards. In addition, there is a likelihood of hazard due to overload vehicles traversing the bridges. These types of hazard will be classified Man-made (Unintentional) hazards.

7.1. Risk assessment: likelihood estimates for collisions/overloads

Roadway accidents are the main sources of vehicle-bridge collisions while some studies have identified various reasons for vessels colliding with bridge members in the waterways. To estimate the likelihood of such occurrences, various models have to reviewed and developed, as described in the following sections.

7.1.1. Roadway accidents involving trucks

The occurrence of accidents or crashes on the highway has been modeled by various research studies, and two particular studies have been conducted in the past on Florida bridges (Thompson et al. 1999; and Sobanjo and Thompson, 2011). Roadway crashes can result in vehicle damage, injuries to vehicle occupants, damage to bridge elements, and involvement of fire. With focus on the crashes particularly involving fuel tankers, which may result in fire hazards, it is important to be able to predict the probability of having such crashes at a bridge location. The first step will be to be able to predict the annual count of crashes at each bridge. Both models in Thompson et al. (1999) and Sobanjo and Thompson (2011) are capable of doing this.

But first, it worth it to mention a study that was conducted in Texas very recently on the truck impacts on bridge piers (Buth et al. 2010). In this study, a risk analysis methodology was developed for vehicle-bridge column/abutment collisions. Example accidents (19) involving collision of trucks with bridge piers are presented from various locations nationwide (10 sites from Texas). Collapse of the bridge occurred in four out of the 19 cases reviewed, while the remaining 15 had pier damage ranging from minor to extensive levels. Of the 15 bridges with pier damage (no bridge collapse), there were two cases with fire incidents.

The Texas study can be broken down into two sections. First a detailed Finite Element Analysis of vehicle impact on rigid piers was done, with the impact forces quantified through simulation. Secondly, a crash risk model was developed to estimate the probability of a truck hitting a bridge pair. In the crash model, the probability of crash was predicted using approximated Poisson distribution models. The crash data for four years was utilized, including information on Run-Off-Road (ROR) crashes for trucks.

The following parameters were defined in order to estimate the crash probability:

1. The probability of a truck ROR crash (P_{T_ROR}) =

   No. of truck RORs on 1 mile segment of roadway/No. of opportunities in AADT on 1 mile segment roadway.

   The denominator is equal to the total no. of trucks using the mile segment in four years, i.e., 4* Truck AADT*365.
2. The probability of truck hitting bridge pier (THBP), given that it ROR ($P_{THBP}|_{T\_ROR}$) =

   No. of trucks hitting bridge pier/No. of truck ROR crashes.

3. The probability of truck hitting bridge pier (P_{THBP}) = ($P_{THBP|T\_ROR}$)* (P_{T_ROR})

A similar but not exact approach will be followed in estimating probability of crashes at bridge locations in Florida. An accident risk model was developed by Thompson et al. (1999) using historical crash data, in which crashes at bridge sites are mentioned to be strongly affected by narrowness of the bridge (defined as the ratio of the number of lanes (lanes) to the roadway width (*roadwidth*) on the bridge), approach alignment (*appralign*), deck condition (*dkrating*), functional classification (*funclass*), bridge length (length), and traffic volume (Average daily Traffic or ADT). Based on the Thompson et al.'s (1999) original model the annual count of crashes *E(y)* at each bridge location can be predicted. Tables 7.1 and 7.2 describe the intermediate variables and the required coefficients for the model.

Table 7.1. Intermediate variables for accident model

Name	Formula	Range in data set
UrbanArterial	funcclass=14 or 16	true or false
AlignLE6	appralign<=6	true or false
Narrowness	lanes/roadwidth	0.06-0.36
ADT		1 to 324,806
BadDeck	dkrating<=6	true or false

Table 7.2. Parameters and statistics of Thompson et al. (1999) model

For bridge where	Variable	Coefficient	Std.Error	t value
UrbanArterial=false	Constant	-377.3701	66.0689	-5.7118
UrbanArterial=true	Constant	886.0098	109.9613	8.2835
All bridges	lanes*length	0.7323	0.0455	16.1039
AlignLE6=false and BadDeck=false	Narrowness*ADT	0.3904	0.0087	44.9273
AlignLE6=true and BadDeck=false	Narrowness*ADT	0.5031	0.0194	25.8690
AlignLE6=false and BadDeck=true	Narrowness*ADT	0.4531	0.0257	17.6592
AlignLE6=true and BadDeck=true	Narrowness*ADT	0.7899	0.0556	14.2052

Using the Average Daily Traffic (ADT), and assuming 365 days per year, the probability of crash at a bridge location, P_{Crash} is estimated as a ratio of the annual count of crashes, E(y), to the opportunities of such crashes in one year, i.e.,

$$P_{Crash} = \frac{E(y)}{ADT*365} \quad (1)$$

This will generate estimates of roadway crashes generally, but for the case of crashes involving fuel tankers, another approach was tried where the vehicle classification scheme was utilized. The idea here is that the fuel tankers can be identified as some specific vehicle classes and those more specific counts of trucks using the roadway can be used to estimate the crashes involving trucks that could cause such fire incidents. The FHWA vehicle classification scheme is shown in Tables 7.3 and 7.4 and Figure 7.1. A brief literature review and Internet search for types of fuel tankers showed that most are in vehicle class 10 (Figure 7.2), while very few may occasionally be in classes 8 and 9.

Table 7.3. The FHWA vehicle classification scheme F

Class	Vehicle Type	Description
1	Motorcycles	All two- or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handle bars rather than wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheeled motorcycles
2	Passenger cars	All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers
3	Four tire, single unit	All two-axle, four-tire, vehicles other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, carryalls, and minibuses. Other two-axle, four-tire single unit vehicles pulling recreational or other light trailers are included in this classification
4	Buses	All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles.
5	Two axle, six tire single unit	All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., having two axles and dual rear wheels
6	Three axle, single unit	All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., having three axles
7	Four or more axle, single unit	All trucks on a single frame with four or more axles
8	Four or less axle single trailer	All vehicles with four or less axles consisting of twounits, one of which is a tractor or straight truck power unit
9	Five axle, single trailer	All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit
10	Six or more axle, single trailer	All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit
11	Five or less axle, multi trailer	All vehicles with five or less axles consisting of three or more units, one of which is a tractor or straight truck power unit
12	Six axle, multi trailer	All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit
13	Seven or more axle, multi trailer	All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

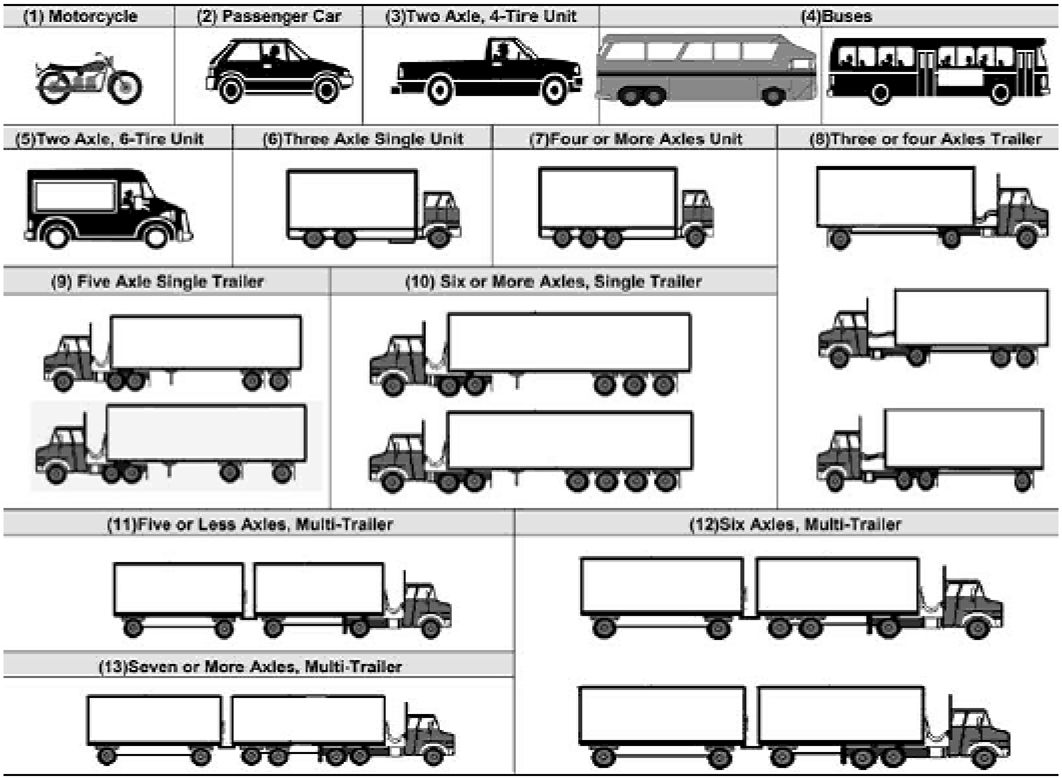


Figure 7.1. FHWA vehicle classification scheme

Table 7.4. FDOT's vehicle classification scheme

CLASS	DESCRIPTION
01	MOTORCYCLES
02	CARS
03	PICK-UPS AND VANS
04	BUSES
05	2-AXLE, SINGLE UNIT TRUCKS
06	3-AXLE, SINGLE UNIT TRUCKS
07	4-AXLE, SINGLE UNIT TRUCKS
08	2-AXL TRCTR W/ 1 OR 2-AXL TRLR, 3-AXL TRCTR W/ 1-AXL TRLR
09	3-AXLE TRACTOR W/ 2-AXLE TRLR
10	3-AXLE TRACTOR W/ 3-AXLE TRLR
11	5-AXLE MULTI-TRLR
12	6-AXLE MULTI-TRLR
13	ANY 7 OR MORE AXLE
14	NOT USED
15	OTHER

Figure 7.2. Typical fuel tankers using the highways

The probability of a fuel tanker (FT) being involved in a roadway crash, $P_{CrashFT}$ can be interpreted as a probability of a roadway crash, P_{Crash}, and a fuel tanker being on the roadway, or estimated as

$$P_{CrashTF} = P_{Crash} * P_{FT} \tag{2}$$

Where,

P_{FT} =Probability of a fuel tanker being on the roadway, which is basically the proportion of the traffic volume that are fuel tankers.

The FDOT collects traffic data at many locations on the Florida highways, including the Portable Traffic Monitoring Sites (PTMS). The data from the PTMS are unique in that they contain information such as traffic volume, speed, vehicle class, etc. Our research is interested in the vehicle classification data.

The Shapefile GIS data for the PTMS is available on the Florida Department of Transportation's website, showing the different locations of PTMS in Florida. Information in the shapefile included AADT, year, *cosite* (a data field for location ID of PTMS), but not the information for the vehicle classes. Data on vehicle classes was obtained from a different FDOT source, the Traffic Data DVD from the FDOT's Planning Division. This data also contained *cosite* information, which was used to merge the data with the PTMS GIS shapefile. Using spatial join tools in the GIS, the nearest PTMS location to the bridge was identified and the revised PTMS data merged with information from the bridge data. All the selected PTMS locations were directly on the same roadway as the bridges and very close to the bridge starting or ending points. Sample data are shown in Table 7.5. It should be noted that this information is not available for all the bridge sites; only 564 bridges have the vehicle classification data.

Table 7.5. Sample bridge and vehicle classification data

Structure ID	Data Year	Cosite No.	Vehicle Class 01	Vehicle Class 02	Vehicle Class 03	Vehicle Class 04	Vehicle Class 05	Vehicle Class 06	Vehicle Class 07	Vehicle Class 08	Vehicle Class 09	Vehicle Class 10	Vehicle Class 11	Vehicle Class 12	Vehicle Class 13	Vehicle Class 14	Vehicle Class 15
020002	2010	025008	1.01	68.19	24.55	0.27	2.63	0.43	0.05	1.98	0.81	0.06	0	0	0.01	0	0
040055	2010	040004	0.46	33.78	29.27	0.33	7.76	3.31	1.72	7.39	13.23	1.13	0.43	0.63	0.56	0	0
100012	2010	100010	0.91	64.53	24.72	0.23	4.15	0.94	0.03	1.81	2.6	0.04	0.03	0	0.01	0	0
100038	2010	105603	0.92	53.51	24.51	0.92	5.15	1.29	0.82	1.86	10.76	0.25	0	0	0	0	0
100103	2010	100010	0.91	64.53	24.72	0.23	4.15	0.94	0.03	1.81	2.6	0.04	0.03	0	0.01	0	0
100592	2010	100010	0.91	64.53	24.72	0.23	4.15	0.94	0.03	1.81	2.6	0.04	0.03	0	0.01	0	0
110002	2010	111002	1.29	45.62	13.54	1.85	9.1	2.34	0.14	3.7	16.74	1.75	0.06	0.21	0.08	0	3.57
120061	2010	120068	0.44	60.54	25.97	0.13	3.93	1.77	0.37	1.88	4.59	0.25	0	0	0.13	0	0
120165	2010	120086	0.6	55.61	29.9	0.23	5.07	1.03	0.2	2.96	3.99	0.25	0.08	0.02	0.05	0	0
120186	2010	120086	0.6	55.61	29.9	0.23	5.07	1.03	0.2	2.96	3.99	0.25	0.08	0.02	0.05	0	0
140020	2010	145603	0.54	76.64	18.33	0.07	1.71	0.49	0.12	1.21	0.77	0.06	0	0	0.05	0	0
150107	2010	150062	0.25	78.76	15.23	0.59	2.22	0.35	0.05	0.91	1.57	0.04	0.03	0	0	0	0
150210	2010	150062	0.25	78.76	15.23	0.59	2.22	0.35	0.05	0.91	1.57	0.04	0.03	0	0	0	0
150255	2010	150077	0.34	76.69	15.9	1.09	2.96	0.65	0.07	0.82	1.34	0.04	0.1	0	0	0	0
160220	2010	160138	0.49	71.56	21.62	0.06	2.25	0.69	0.06	1.17	1.84	0.04	0.05	0.04	0.12	0	0
180021	2010	180118	0.24	61.81	18.47	1.18	3.92	1.59	0.66	2.74	8.34	0.98	0.05	0	0.02	0	0
260006	2010	260248	0.91	56.72	36.27	0.09	1.39	0.45	0.01	1.08	2.96	0.06	0.02	0.03	0	0	0
490032	2010	491502	0.73	57.47	32.51	0.94	5.3	0.47	0.03	1.62	0.84	0.06	0	0	0.02	0	0
490100	2010	490125	0.68	59.94	29.6	0.43	4.8	1.48	0.03	2.74	0.22	0.06	0	0	0.03	0	0
510052	2010	511503	0.79	64.71	27.48	0.59	2.97	0.75	0.06	1.36	1.25	0.04	0	0	0	0	0
510063	2010	511604	0.92	57.12	29.8	0.77	4.27	0.81	0.15	1.9	4.14	0.06	0	0	0.04	0	0
540006	2010	542001	0.32	56.09	18.38	0.96	2.93	0.52	0.11	2.04	17.43	0.25	0.54	0.37	0.06	0	0
540049	2010	542001	0.32	56.09	18.38	0.96	2.93	0.52	0.11	2.04	17.43	0.25	0.54	0.37	0.06	0	0
540061	2010	540235	0.67	58.55	29.79	0.35	4.71	0.82	0.03	1.46	3.57	0.06	0	0	0	0	0
550008	2010	550214	0.71	69.08	23.84	0.82	3.25	0.44	0	1.17	0.65	0.04	0	0	0	0	0
554146	2010	553073	0.84	66.08	28.72	0.39	2.73	0.31	0.08	0.62	0.17	0.06	0	0	0	0	0
570044	2010	570088	0.29	68.21	25.39	0.42	3.18	0.69	0.12	0.59	1.04	0.04	0.02	0	0	0	0
570045	2010	570088	0.29	68.21	25.39	0.42	3.18	0.69	0.12	0.59	1.04	0.04	0.02	0	0	0	0
570071	2010	570088	0.29	68.21	25.39	0.42	3.18	0.69	0.12	0.59	1.04	0.04	0.02	0	0	0	0
570073	2010	570088	0.29	68.21	25.39	0.42	3.18	0.69	0.12	0.59	1.04	0.04	0.02	0	0	0	0
720214	2010	720899	0.26	67.85	24.21	0.32	1.71	0.68	0.07	0.73	3.83	0.04	0.21	0.07	0	0	0.01
720263	2010	720899	0.26	67.85	24.21	0.32	1.71	0.68	0.07	0.73	3.83	0.04	0.21	0.07	0	0	0.01
750002	2010	750480	0.39	64.11	26.71	0.92	5.34	0.52	0.06	0.67	1.14	0.04	0.06	0	0.01	0	0.03
750167	2010	750480	0.39	64.11	26.71	0.92	5.34	0.52	0.06	0.67	1.14	0.04	0.06	0	0.01	0	0.03
750552	2010	750480	0.39	64.11	26.71	0.92	5.34	0.52	0.06	0.67	1.14	0.04	0.06	0	0.01	0	0.03
860003	2010	860346	0.44	85.89	9.61	0.36	1.17	0.45	0.44	0.21	0.31	0.25	0	0	0.05	0	0.83
860029	2010	860478	0.43	82.94	14.5	0.84	0.87	0.08	0.01	0.09	0.17	0.06	0	0	0	0	0
860035	2010	860429	0.75	79.25	16.42	0.76	2.1	0.19	0.02	0.35	0.09	0.04	0	0	0	0	0.02
860134	2010	860478	0.43	82.94	14.5	0.84	0.87	0.08	0.01	0.09	0.17	0.06	0	0	0	0	0
860622	2010	860429	0.75	79.25	16.42	0.76	2.1	0.19	0.02	0.35	0.09	0.04	0	0	0	0	0.02
860623	2010	860429	0.75	79.25	16.42	0.76	2.1	0.19	0.02	0.35	0.09	0.04	0	0	0	0	0.02
870631	2010	872539	0.23	85.08	11.47	0.98	1.42	0.48	0	0.15	0.11	0.06	0	0	0.01	0	0.01
870633	2010	871116	0.17	71.02	22.62	0.54	4.62	0.22	0.01	0.57	0.23	0	0	0	0	0	0
870785	2010	872572	0.78	19.61	2.33	75.73	1.36	0.19	0	0	0	0	0	0	0	0	0
870786	2010	872571	0.38	19.27	2.48	75.19	2.1	0.57	0	0	0	0	0	0	0	0	0
870787	2010	872570	0.25	41.23	2.47	54.57	0.74	0.74	0	0	0	0	0	0	0	0	0
890113	2010	890044	0.54	77.17	19.22	0.05	1.46	0.07	0	0.88	0.04	0	0	0	0.01	0	0.55
900095	2010	900623	2.63	68.63	18.05	1.04	5.32	0.54	0.44	1.54	1.72	0	0.08	0.01	0	0	0
900096	2010	900623	2.63	68.63	18.05	1.04	5.32	0.54	0.44	1.54	1.72	0	0.08	0.01	0	0	0
920001	2010	920314	0.85	75.93	15.51	0.23	2	1.94	0.23	0.89	2.16	0.25	0	0	0	0	0
920199	2010	920058	1.26	48	15.13	0.33	3.48	1.72	0.27	3.3	24.68	0.71	0.16	0.34	0.63	0	0
920200	2010	920058	1.26	48	15.13	0.33	3.48	1.72	0.27	3.3	24.68	0.71	0.16	0.34	0.63	0	0
934924	2010	930680	1.6	89.76	6.27	0.15	1.91	0.09	0.09	0.11	0.01	0	0	0	0.01	0	0
940003	2010	940719	2.48	84.21	9.47	0.12	1.52	0.62	0.42	0.94	0.19	0.04	0	0	0	0	0
940004	2010	940719	2.48	84.21	9.47	0.12	1.52	0.62	0.42	0.94	0.19	0.04	0	0	0	0	0
940084	2010	940719	2.48	84.21	9.47	0.12	1.52	0.62	0.42	0.94	0.19	0.04	0	0	0	0	0
940085	2010	940719	2.48	84.21	9.47	0.12	1.52	0.62	0.42	0.94	0.19	0.04	0	0	0	0	0
944008	2010	948528	0.76	62.66	24.4	1.94	3.41	0.14	0.3	5.18	0.18	0.39	0.09	0.24	0.32	0	0

7.1.1.1. Predicted accident counts

The Thompson et al. (1999) model was applied to Florida's state-maintained bridges, considering 4651 bridges in total, after the necessary filtering of the data. As shown in Table 7.6, the predicted accident counts at specific bridge sites varied from a minimum of zero to 31.9 with an average of 1.5 for all vehicles. For trucks the corresponding values were a range of zero to just over six and an average of almost zero (0.1). The overall variation in the predicted annual accidents at bridge locations is shown in Figure 7.3. About 73% of the bridges have two or fewer vehicle accidents predicted for annual occurrence while 95% have six or fewer crashes. Over 10 crashes annually were estimated for 78 bridges, or just over 1.6% of the bridges considered. Using the predicted count of accidents and the traffic volume, a conventional estimate of risk, the annual accidents per million daily vehicles (aamdv) was also estimated. The variation is shown in Figure 7.4.

Table 7.6. Statistics summary of bridge accident parameters

Parameter	Mean	Standard deviation	Minimum	Maximum	Count
Annual Vehicle Accident Count	1.5	2.4	0	31.9	4651
Annual Accidents Per Million Daily Vehicles	388.5	12873.6	0	864513.6	4651
Annual Truck Accident Count	0.1	0.3	0	6.1	4651
Truck Annual Accidents Per Million Daily Vehicles	22.3	803.2	0	53304.9	4651
Prob. of Annual Vehicle Accidents ($x\,10^{-6}$)	1.064	35.270	0	2368.530	4651
Prob. of Annual Truck Accidents ($x\,10^{-6}$)	0.061	2.200	0	146.041	4651

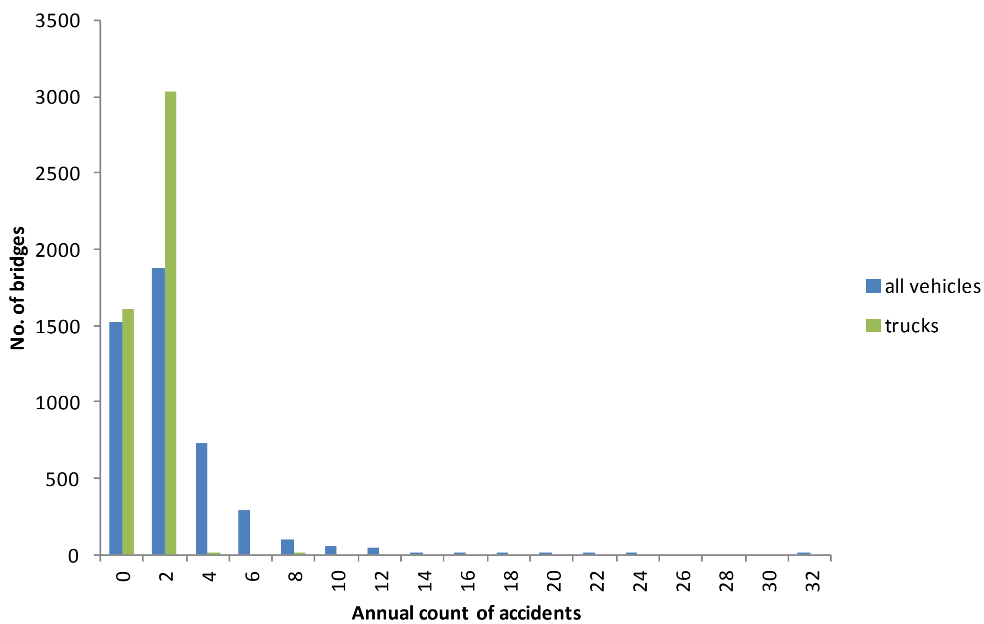


Figure 7.3. Variation in predicted number of annual accidents on Florida bridges

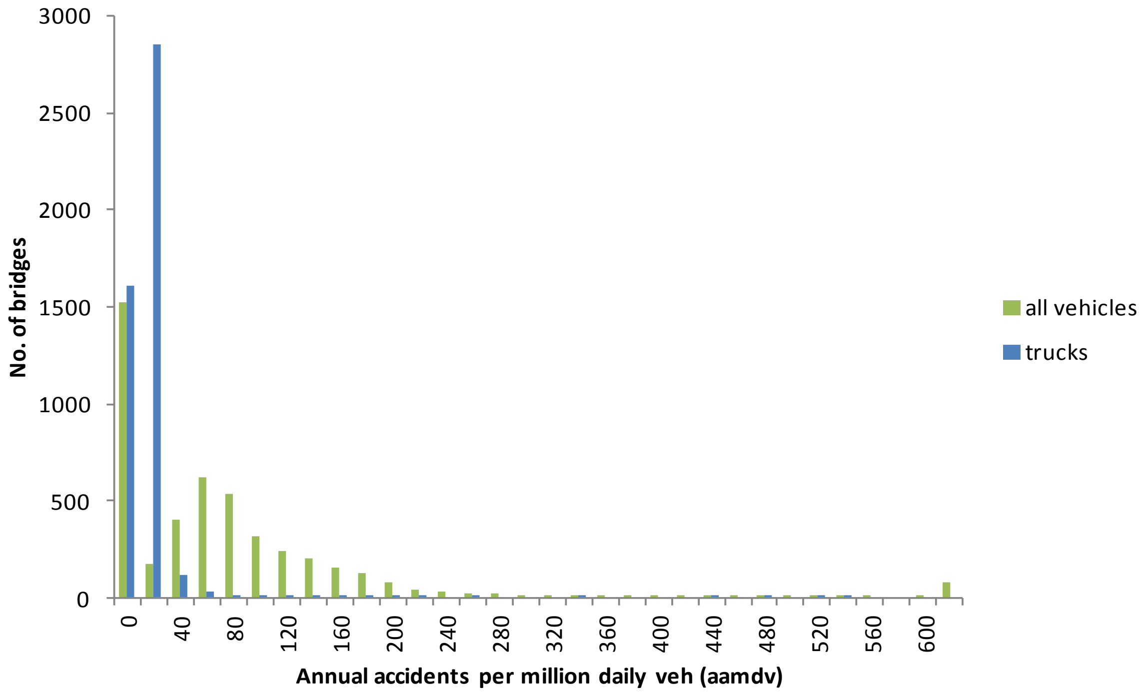


Figure 7.4. Variation in annual accidents per million daily vehicles on Florida bridges

Next the probability of an accident occurring on the bridge was estimated as a ratio of the predicted annual crash count, to the number of vehicles exposed to the accident (ADT x 365), i.e., using equation 1 explained earlier. From the bridge data analyzed, results for those bridges with more than 1 x 10^{-6} annual probability of accident are summarized in Table 7.7. The estimated probabilities for all vehicles varied from a minimum of zero to maximum of 2368.5 x 10^{-6}, and an average of 1.06 x 10^{-6} (Table 7.6). About 66% of the bridges considered have a probability of 2 x 10^{-6} or less of having roadway crashes involving all vehicles, or less than 1 in half a million chances. It was identified that 150 bridges have more than a probability of 10 x 10^{-6} of having roadway crashes. The variations are shown in Figure 7.5.

The probability of roadway crash involving a truck was estimated as simply a product of the probability of roadway crash and the probability of trucks being in the traffic stream, i.e., using equation 2 stated earlier. The percentage truck in the ADT was used as the probability of the trucks being in the traffic. Since some bridge records indicate zero truck percentage, these bridges will have zero probability of a truck being involved in a roadway crash. For accidents involving trucks, the probability of accident at bridge locations ranged from zero to 146.04 x 10^{-6}, and an average of 0.06 x 10^{-6} with the variation shown in Figure 7.5. For accidents involving trucks, most bridges (99.4%) have a probability 2 x 10^{-6} or less of annual occurrence. For a comparison, the TTI study on truck impact crashes on bridge piers, the probabilities of crash where truck runs off road were estimated as 3.799 x 10^{-7} and 2.986 x 10^{-7} respectively, for undivided and divided roadways (Butt et al., 2010).

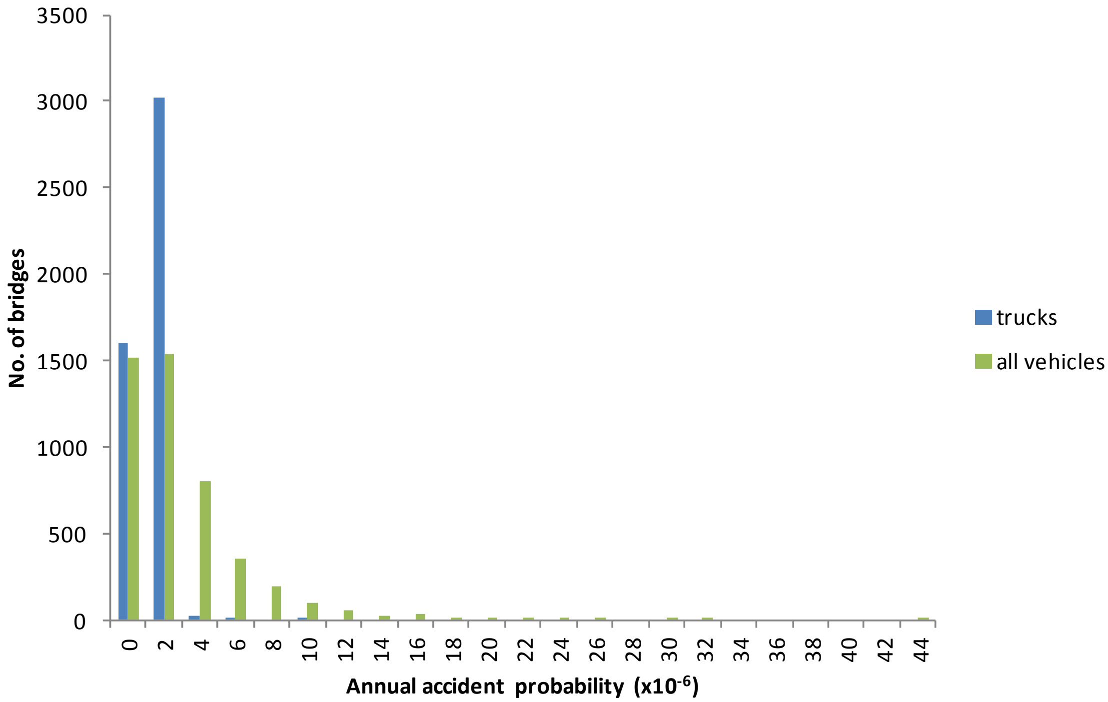


Figure 7.5. Variation in annual probability of accidents on Florida bridges

The last approach considered was to identify the types of trucks responsible for most fire accidents, specifically assumed to be fuel tankers, i.e., vehicle class 10, and apply their probabilities of being in the traffic stream. Obviously this will make the already low probability estimates, even smaller. But it is something that should be considered. For example, the bridges with IDs 150036, 150252, and 170083 from Table 7.7, have data on the vehicle classification. The truck percentage of vehicle class 10 (fuel tankers) are respectively 0.02%, 0.02% and 0.07% respectively for these three bridges. These are much lower than the overall percentage trucks (all trucks) of 3%, 6% and 8% respectively for the three bridges. A revision of the estimated probability of truck in crash can now be done using the percentage of the traffic that is fuel tankers, i.e., vehicle class 10. As shown in Table 7.8, for bridge IDs 150036, 150252, and 170083, the original estimated accident risk (truck annual accidents per million daily vehicles) were 0.009 $x10^{-6}$, 0.012 $x10^{-6}$, and 0.017 $x10^{-6}$respectively. But now with the vehicle class data, the revised estimates are 0.000002 $x10^{-6}$, 0.000002 $x10^{-6}$, and 0.000012 $x10^{-6}$ respectively, showing a significant reduction in the likelihood of such event.

Table 7.7. Bridges with greater than 1 x 10^{-6} annual probability of vehicle accident

Bridge ID	Facility	Functional Class	Percent truck	Year Built	Total ADT for 2012	Annual Vehicle Accident Count	Annual Accidents Per Million Daily Vehicles	Annual Truck Accident Count	Truck Annual Accidents Per Million Daily Vehicles	Prob. of Annual Vehicle Accidents (x 10^{-6})	Prob. of Annual Truck Accidents (x 10^{-6})
024022	SR-951(COLLIER BL)	08	5	1969	1048	0.4	425.4	0.0	21.3	1.165	0.058
105400	SR 684 (Cortez Rd)	19	0	1956	1860	1.0	541.5	0.0	0.0	1.484	0.000
150004	SR60 WB	16	5	1999	2190	1.0	460.2	0.1	23.0	1.261	0.063
554036	US-17 (SR-15)	07	1	1971	101	0.5	4603.4	0.0	46.0	12.612	0.126
554050	US-17 NB (SR-15)	07	3	1973	727	0.3	444.6	0.0	13.3	1.218	0.037
574130	SR-109A (CESERY)	09	0	1966	25	0.1	2279.3	0.0	0.0	6.245	0.000
580816	SR-109(UNIVERSITY)	09	0	1983	105	0.1	648.3	0.0	0.0	1.776	0.000
580817	I-295 SB (SR-9A)	09	0	1994	63	0.4	5779.1	0.0	0.0	15.833	0.000
584016	SR-228 (LEG E)	09	0	1967	58	0.0	634.4	0.0	0.0	1.738	0.000
584025	SR-228	08	0	1967	74	0.5	6550.9	0.0	0.0	17.948	0.000
584048	SR-202 EB (JTB)	09	1	1988	42	0.7	15479.5	0.0	154.8	42.410	0.424
584060	I-295 (9A)DAMES PT	09	0	1989	63	2.0	31601.0	0.0	0.0	86.578	0.000
584115	I-95 NB (SR-9)	09	6	1992	348	0.4	1013.4	0.0	60.8	2.776	0.167
584130	SR-105 NB	09	0	1994	21	0.0	680.1	0.0	0.0	1.863	0.000
584132	SR-13 SB (ACOSTA)	09	0	1991	70	0.4	5355.6	0.0	0.0	14.673	0.000
584133	SR-13 NB (ACOSTA)	09	1	1991	132	0.4	2862.5	0.0	28.6	7.842	0.078
584140	SB ACOSTA N. LEG H	09	0	1991	85	0.1	694.3	0.0	0.0	1.902	0.000
584142	NB ACOSTA N. LEG G	09	0	1991	48	0.4	7486.5	0.0	0.0	20.511	0.000
584146	US-17 ACOSTA LEG K	09	0	1991	51	0.0	692.1	0.0	0.0	1.896	0.000
584189	SR-9A SB	09	0	2005	45	0.6	12216.2	0.0	0.0	33.469	0.000
584190	SR-9A NB	09	0	2005	96	0.6	5789.6	0.0	0.0	15.862	0.000
584198	I-95 (SR-9)NB & SB	09	0	2003	96	0.6	6362.0	0.0	0.0	17.430	0.000
584200	I-95 (SR-9)	09	0	2003	51	0.8	14772.9	0.0	0.0	40.474	0.000
584201	I-95 (SR-9)	09	5	2003	107	0.7	6601.8	0.0	330.1	18.087	0.904
584214	I-95SB to I-295SB	09	0	2004	20	0.5	25891.2	0.0	0.0	70.935	0.000
585001	I-95NB TO I-295NB	17	1	2005	301	0.8	2671.2	0.0	26.7	7.318	0.073
590021	SR-116 E.B. & W.B.	07	2	2003	1057	0.4	384.3	0.0	7.7	1.053	0.021
590023	SR-116 E.B. & W.B.	07	17	2003	2523	1.4	543.3	0.2	92.4	1.488	0.253
590039	SR-10 W.B.	02	9	2000	2418	1.1	448.4	0.1	40.4	1.228	0.111
594001	SR-202 WB TO SR-9A	09	0	2007	48	0.0	615.2	0.0	0.0	1.685	0.000
600006	US-90 EB (SR-212)	08	8	2009	1472	0.6	438.7	0.1	35.1	1.202	0.096
720642	SR-312 EB	17	5	1976	9570	4.5	469.0	0.2	23.4	1.285	0.064
180941	US98 SR30	09	1	2006	1	1.0	864513.6	0.0	8645.1	2368.530	23.685
184000	US98 SR30	09	5	2003	348	1.5	4240.4	0.1	212.0	11.618	0.581
260101	SR 20	14	3	1998	31015	14.7	472.7	0.4	14.2	1.295	0.039
270006	US98 SR30	16	18	1960	11039	15.8	1429.8	2.8	257.4	3.917	0.705
280032	I110 SR8A	07	8	2007	443	0.4	984.1	0.0	78.7	2.696	0.216
280046	I10 SR8	07	5	2006	631	5.9	9290.3	0.3	464.5	25.453	1.273
290083	US98 SR30	06	9	1988	3259	1.5	460.6	0.1	41.5	1.262	0.114
290084	US98 SR30	07	10	1988	1315	6.1	4615.0	0.6	461.5	12.644	1.264
290085	US98 SR30	07	3	1990	666	2.4	3573.7	0.1	107.2	9.791	0.294
290086	SR 300	01	23	2003	23656	15.7	662.4	3.6	152.4	1.815	0.417
320015	I 10, SR 8	08	5	1978	335	2.2	6426.5	0.1	321.3	17.607	0.880
320025	I 10 SR 8	08	10	1975	266	0.1	500.9	0.0	50.1	1.372	0.137
320026	I10 SR8	08	10	1975	266	0.1	434.6	0.0	43.5	1.191	0.119
320047	US27 SR63	07	22	2000	527	0.3	546.6	0.1	120.3	1.498	0.329
320050	US27 SR63	08	10	2001	163	0.2	1503.5	0.0	150.4	4.119	0.412
380037	US 90 SR 10	07	5	1959	579	0.4	733.2	0.0	36.7	2.009	0.100
480105	SR20	08	1	1972	278	0.3	1195.1	0.0	12.0	3.274	0.033
490838	I10 SR8	09	5	1997	26	0.3	12127.5	0.0	606.4	33.226	1.661
490839	I10 SR8	09	5	1997	26	0.3	12124.0	0.0	606.2	33.216	1.661
520082	US331 SR83	08	0	1991	979	3.0	3112.5	0.0	0.0	8.527	0.000
534007	I10 SR8	08	1	1973	201	0.5	2658.1	0.0	26.6	7.283	0.073
734066	I-595 to NB I-95	19	2	1969	1014	0.4	383.5	0.0	7.7	1.051	0.021
734080	SB A1A to WB 820	14	2	1976	239	1.1	4655.0	0.0	93.1	12.753	0.255
735502	SB I-95	14	5	1975	116	1.0	8712.6	0.1	435.6	23.870	1.194
744001	WB I-595 (SR-862)	09	0	1987	52	0.7	13616.7	0.0	0.0	37.306	0.000

Table 7.7. Bridges with greater than 1 x 10^{-6} annual probability of vehicle accident (Cont'd)

Bridge ID	Facility	Functional Class	Percent truck	Year Built	Total ADT for 2012	Annual Vehicle Accident Count	Annual Accidents Per Million Daily Vehicles	Annual Truck Accident Count	Truck Annual Accidents Per Million Daily Vehicles	Prob. of Annual Vehicle Accidents (x 10^{-6})	Prob. of Annual Truck Accidents (x 10^{-6})
744006	EB I-595 (SR-862)	09	0	1987	33	1.8	54521.5	0.0	0.0	149.374	0.000
744007	SB I-95 to I-595	07	5	1987	453	0.5	1207.2	0.0	60.4	3.307	0.165
744009	NB I-95 (SR-9)	07	5	1988	233	0.5	2243.6	0.0	112.2	6.147	0.307
864092	NB I-95 (SR-9)	16	5	1982	1816	1.2	651.0	0.1	32.5	1.784	0.089
864093	SB I-95 (SR 9)	16	5	1982	1816	1.2	666.1	0.1	33.3	1.825	0.091
864104	St Lucie West Blvd	16	5	1989	4360	1.7	387.2	0.1	19.4	1.061	0.053
157127	I-75	19	3	1966	568	1.7	2952.8	0.1	88.6	8.090	0.243
544067	SR-404 EB	09	0	1971	52	0.1	2632.7	0.0	0.0	7.213	0.000
710027	SR-426	16	10	1979	3155	1.6	499.9	0.2	50.0	1.370	0.137
714048	Central Fl Parkway	09	10	1990	159	0.2	1215.1	0.0	121.5	3.329	0.333
720026	SR-417 SB	14	24	1993	4191	1.8	419.5	0.4	100.7	1.149	0.276
724055	SR-44	19	25	1990	369	0.8	2043.9	0.2	511.0	5.600	1.400
724115	SR-430 WB	17	2	1997	1365	0.8	590.0	0.0	11.8	1.616	0.032
724116	SR-430 EB	19	5	1997	1000	0.8	752.5	0.0	37.6	2.062	0.103
724151	US-92 WB	17	5	2001	9598	3.6	370.4	0.2	18.5	1.015	0.051
724153	US-92 EB	19	3	2001	2103	1.2	589.6	0.0	17.7	1.615	0.048
724175	I-4 WB	19	15	2002	2198	0.9	415.9	0.1	62.4	1.139	0.171
794038	CA Ramp	16	5	2007	5309	2.1	400.2	0.1	20.0	1.096	0.055
750556	S MIAMI AVE-I95 NB	11	4	1968	2839	1.4	489.7	0.1	19.6	1.342	0.054
750607	SR-826 WB	12	14	1989	2734	2.9	1057.0	0.4	148.0	2.896	0.405
750801	E/B SR 41 - A1A	09	0	1995	3	0.2	52581.5	0.0	0.0	144.059	0.000
754009	I-95NB HOV(870774)	09	5	1994	28	0.7	23825.3	0.0	1191.3	65.275	3.264
784058	SR-5 (US-1) NB	09	5	1972	159	2.6	16629.5	0.1	831.5	45.560	2.278
784064	SR 5 (US 1)	19	5	1979	309	0.2	744.2	0.0	37.2	2.039	0.102
784065	US 1 (SR 5)	19	1	1979	257	0.6	2223.7	0.0	22.2	6.092	0.061
100381	SR618A (CROSSTOWN)	16	3	2006	11079	9.1	822.8	0.3	24.7	2.254	0.068
124078	I-275 SB (SR 93)	19	2	1980	506	1.6	3194.2	0.0	63.9	8.751	0.175
124079	I 275 NB	09	1	1980	102	1.6	16161.9	0.0	161.6	44.279	0.443
124080	I-275 NB TO I-375	09	1	1977	116	0.1	706.3	0.0	7.1	1.935	0.019
124133	SR 580 EB	17	2	1988	3524	1.9	529.6	0.0	10.6	1.451	0.029
124906	I 275 SB	09	1	1991	261	6.7	25806.6	0.1	258.1	70.703	0.707
124909	I-275 SB	19	3	1994	786	1.0	1240.5	0.0	37.2	3.399	0.102
124910	I-275 NB	17	2	1992	2074	1.1	537.3	0.0	10.7	1.472	0.029
125007	SR 679	19	0	2001	95	0.3	2989.2	0.0	0.0	8.190	0.000
134046	PINELLAS TRAIL	08	10	1999	772	0.4	461.2	0.0	46.1	1.263	0.126
144035	SR 570 POLK WB 6.6	16	5	1998	2301	1.2	525.0	0.1	26.3	1.438	0.072
750077	SR91 WB TPK 55.2	14	5	1990	3200	1.7	519.3	0.1	26.0	1.423	0.071
750515	SR821 HEFT NB 12.4	14	3	1972	2198	1.1	512.1	0.0	15.4	1.403	0.042
134117	SR-60A	14	1	1954	1536	1.0	651.5	0.0	6.5	1.785	0.018
156710	I-75	19	5	1966	1162	1.8	1536.4	0.1	76.8	4.209	0.210
184055	US98 SR30 Westbd	07	5	-1	592	0.5	846.7	0.0	42.3	2.320	0.116
262502	US90 CERVANTES ST	08	10	1940	269	0.1	427.1	0.0	42.7	1.170	0.117
300008	US 98 SR 30	02	9	1935	2103	2.4	1124.2	0.2	101.2	3.080	0.277
370017	US 90 SR 10	07	10	1938	564	1.1	1873.1	0.1	187.3	5.132	0.513
490025	I 10 SR 8	06	16	1968	731	5.7	7849.3	0.9	1255.9	21.505	3.441
490028	I 10 SR 8	02	11	1968	1682	1.0	612.4	0.1	67.4	1.678	0.185
490801	I 10 SR 8	09	25	1968	75	2.7	35511.8	0.7	8877.9	97.293	24.323
490807	I10 SR8	09	5	1968	26	0.3	10625.2	0.0	531.3	29.110	1.456
494000	SR87	08	2	2008	301	1.8	5891.2	0.0	117.8	16.140	0.323
520047	US-331 SR-83	01	32	1940	8464	4.1	481.2	1.3	154.0	1.318	0.422
560804	SR-13	09	5	1921	26	0.3	10299.2	0.0	515.0	28.217	1.411
570028	SR-105 (FT.GEORGE)	07	2	1949	252	0.2	822.2	0.0	16.4	2.253	0.045
574009	I-95 (SR-9)	09	1	1958	101	0.5	4533.1	0.0	45.3	12.419	0.124
584158	I95	09	2	2007	603	1.3	2098.1	0.0	42.0	5.748	0.115
720489	I-4	14	1	1959	2103	1.1	508.6	0.0	5.1	1.393	0.014
720490	I-4	14	1	1959	1998	1.0	514.1	0.0	5.1	1.408	0.014
724236	Ramp GE	09	10	2009	216	0.1	484.7	0.0	48.5	1.328	0.133
790141	US-92	14	50	1938	9	0.9	106609.7	0.5	53304.9	292.082	146.041

Table 7.8. Revised probability of truck accidents involving fire

Bridge ID	Facility	Functional Class	Percent truck	Year Built	Total ADT for 2012	Annual Vehicle Accident Count	Annual Accidents Per Million Daily Vehicles	Annual Truck Accident Count	Prob. of Annual Truck Accidents (x 10^{-6})	Percent vehicle class 10 (fuel tankers?)	Revised Prob. of Annual Truck Accidents (x 10^{-6})
150036	SR-60	14	3	1941	82532	8.8	107.0	0.3	0.009	0.02	0.000002
150252	US-41 (SR-45)	11	6	1923	132998	9.7	72.7	0.6	0.012	0.02	0.000002
170083	US-19 (SR-20)	11	8	1958	135883	10.6	78.2	0.9	0.017	0.07	0.000012

7.1.2. Vessel impact accidents

It is very important to estimate of the likelihood of bridge elements, especially substructures, getting impacted by vessels. To understand the existing methods for designing or evaluating bridges for vessel impacts, two documents were reviewed – the AASHTO Specifications for Vessel Collision Design of Bridges (AASHTO 2009); and report on a recent research about vessel impacts on Florida bridges by Consolazio et al. (2010). The former explains the design and evaluation methods for bridges while the latter performed a finite element modeling of the bridge, recommending some improvements to the AASHTO code.

The focus of Consolazio et al. (2010) is further developing the probability of collapse expressions for bridge piers subject to barge impact loading. During the collision of a traveling water vessel, such as a large barge, with a structural component of a bridge, there is a large horizontal force transferred into the bridge superstructure that could possibly cause structural collapse. This article develops and improves expressions measuring the probability of structural collapse due to this type of collision. These probabilistic collapse expressions developed in this study are meant to serve as an aid in the design of bridges for vessel collision. Through the use of probability analysis, along with the aid of finite element analysis of barge-pier collision simulations, the authors propose new structural component designs to better withstand vessel impact at the critical impact locations they have determined.

Now although vessel impact is not an environmental hazard, it poses a very serious hazard to coastal bridges and needs to be addressed. Also, the methods for the development of the probability expressions for vessel collision can be very similar to those of environmental hazards. The paper quantifies the probability of vessel impact on structural elements of bridges that traverse waterways, specifically, the collapse of bridge piers from vessel impact, not just the general collapse of the entire structure like in the previous articles. The authors go into elaborate detail in developing various finite element models of the bridge piers and the barge-bow representing the vessel that induces the impact, providing detailed results of deformation and collision forces.

One of the major components of this paper is further developing the relationship between vessel impact deformation and the damage inflicted on the structural components of the bridge. This also provides insight into how much detail can be put into developing probability expressions, which under similar methodology, can assist in dealing with risk due to environmental hazards.

One very direct and important application of this paper's methodology is how the authors went about identifying which bridges to look at. In other words, the approach they took to select what spectrum of bridges to choose for study. In this article, it is shown that there is great importance in selecting a set of bridge cases that represent a wide range of bridge types. Many other articles in this literature review apply a method in which only bridges of a certain category are addressed. For example, there are other journals mentioned in this review that focus on addressing the risk a bridge is vulnerable to structural damage from storm surges experienced in hurricanes. The only bridges of interest in addressing that specific environmental hazard were low elevation, concrete slab and girder bridges that cross coastal waterways

The 1980 collapse of the Sunshine Skyway Bridge led to the decision to consider vessel collision risk in the design of bridges, providing for the analysis of existing bridges to determine their vulnerability and potential for retrofit: "bridge structures to be designed to prevent collapse of the superstructure by considering the size and type of fleet,…., available water depth, vessel speed, structure response, risk of collision, and operational classification of bridge." There are three recommended methods for designing or evaluating bridges with regard to vessel collision. Method I is very detailed, involving the selection of design vessel, but maybe too detailed for consideration as a network-based methodology. The Method II risk analysis, which appears more suitable for network-based application, is recommended for existing bridges in a procedure based on estimated annual frequency of collapse. This estimate is used to identify and rank high risk bridges, and prioritize vulnerable bridges for potential rehabilitation, retrofit, pier protection, or replacement. The bridges vulnerable to ship and barge impacts are typically located in coastal areas and along inlet waterways. The Method III analysis involves the evaluation of the benefits of risk reduction efforts relative to the costs of bridge strengthening or protection system; this method is also suitable for network risk of bridges, particularly, if combined with Method II.

According to the AASHTO Guide (2009) from 1960 to 2002 (42 years), 31 major collapses are reported worldwide, with loss of 342 lives; 17 of these collapses occurred in the US resulting in loss of 35 lives. Many vessel collisions caused damage varying from very minor to significant levels but do not necessarily result in bridge collapse or loss of life. Based on a study by the US Coast Guard in 2003, of the 2692 vessel accidents observed between 1992 and 2001, there was no fatality involved, only 61caused bridge damage in excess of $500,000 while 1703 of these accidents caused very minor damage with no repair costs.

The factors identified as contributing to the bridge vulnerability to vessel collision includes its location relating to the alignment of the navigation channel, nearness to waterfront docks, as well as the adequacy of the span (length) over the navigation channel. Poorly sited bridges on unusual bends or turns are vulnerable as well as bridges with short spans over the navigation channel.

The AASHTO Guide (2009) discussed the risk analysis involving extreme event combinations of scour and vessel collision. Two load cases were mentioned: 1) Minimum impact load from vessel collision combined with the long-term scour; and 2) maximum impact load combined with half of the predicted long-term scour. The argument was made that based on historical records, merchant ships or barges do not transit during storms or extreme events, thus vessel impact would probably never occur during hurricanes or storms. Also among the 31 bridges observed to have collapsed worldwide, none was mentioned to have scour concerns.

Bridges are also given an operational classification: Critical/Essential bridges; and typical bridges. The former are bridges that are located on the STRAHNET routes (NBI Item 100), and they must function after impact from a design vessel whose probability of occurrence is smaller than for typical bridges. Their locations always indicate high vehicle volumes (potential of high loss of lives), and connection to civil defense, police, fire department, and health facilities.

The annual frequency of bridge elements collapse (AF) is defined as

$$AF = N*PA*PG*PC*PF \qquad (3)$$

Where,

N = Annual frequency of vessel by type and size

PA = Probability of aberrancy

PG = Geometric probability
PC = Probability of collapse
PF = Protection factor

PA is further defined as

$PA = BR*R_B*R_C*R_{XC}*R_D$

Where,

BR = Empirical values used for barges and ships
R_B = Factor depending on geometry of location
R_C = Factor depending on velocity of current parallel to vessel path
R_{XC} = Correction factor for X-currents acting perpendicular to the vessel path
R_D = Correction factor for vessel traffic density

The Geometric Probability (PG) is the conditional probability that a vessel will hit a bridge pier or span given that it has lost control (aberrant) in the vicinity of the bridge. It is determined based on the pier dimensions and vessel width. The Probability of Collapse (PC) is based on the ratio of bridge's lateral capacity to the collision impact load. PF is estimated as the fraction of the bridge elements not protected. If there is no protection, PF = 1.0; if there is 100% protection, PF = 0.0; and if say there is 70% protection (e.g., use of dolphins), PF = 0.3.

The Method III analysis evaluates of the benefits of risk reduction efforts relative to the costs of bridge strengthening or a protection system. This is estimated as a discounted present worth of the expected disruption costs due to the bridge collapse from vessel collision,

PW = AF*DC*(discount factor)

$$PW = (AF)(DC)\left[\frac{1+g}{1-g}\right]\left(1-\left[\frac{1+g}{1+i}\right]^{Y}\right) \tag{4}$$

Where,

PW = present worth of the disruption cost,
AF = Annual frequency of bridge collapse
DC = Disruption costs associated with bridge collapse (pier replacement cost, span replacement cost, motorist inconvenience cost, and port interruption cost),
g = real annual rate of growth of disruption costs (as decimal, 2%/yr. = 0.02)
i = discount rate (as decimal, 4%/yr. = 0.04),
Y = design life of bridge (years)

The disruption cost associated with the collapse of the bridge is computed as

DC = PRC + SRC + MIC + PIC

Where

DC = disruption cost,
PRC = pier replacement cost,
SRC = span replacement cost,
MIC = motorist inconvenience cost, and
PIC = port interruption cost

Method III may be useful in the risk study for mitigation analyses, i.e., comparing PW above to the cost of a mitigation effort (strengthening, adding a pier protection system, etc.)

The challenge for network level evaluation of vessel impact risk is the availability of the input parameters needed to estimate the annual frequency of collapse (AF). It is necessary to know which data are already available in Pontis or in local district records. It is recommended that FDOT districts develop a database (coastal and inlet bridges) of pertinent information required to perform the Method II analyses on a network basis. The primary parameter to be estimated is the Annual Frequency of bridge collapse, AF defined above. The AASHTO Guide specifications have established the following acceptance criteria for bridge collapses associated with vessel collisions: AF = 0.0001 per year for critical/essential bridges (AF<= 0.01 in 100 years or one failure every 10,000 years); and AF=0.001 for typical bridges (AF<= 0.1 in 100 years or one failure every 1,000 years).

Another question of interest is to identify which bridges are essential or critical. It appears that those on the STRAHNET routes (NBI Item 100) qualify for this class. According to FDOT (2012), the Strategic Highway Network (STRAHNET) is a designation given to roads that provide “defense access, continuity, and emergency capabilities for movements of personnel and equipment in both peace and war.” STRAHNET includes Routes (for long-distance travel) and Connectors (to connect individual installations to the Routes).

STRAHNET Routes include all of the Interstate highways and the following additional routes:

HIGHWAY ROUTE NUMBERS	ROUTE DESCRIPTION	APPROX. MILEAGE	INSTALLATIONS ALONG THE CORRIDOR
US 231	From the Alabama / Florida state border to US 98 in Panama City.	66	Tyndall AFB
FL 91 (Florida Turnpike), FL 528 (Bee Line Expressway)	Take FL 91 from I-75 south of Ocala to FL 528 in Orlando. Then take FL 528 to FL 401 at Cape Canaveral.	106	Cape Canaveral, Patrick AFB
FL 826, US 1	Take FL 826 from I-95 at Biscayne Gardens in Miami to US 1 in southern Miami. Then take US 1 to Key West.	171	Key West Naval Complex

STRAHNET Connectors have been defined for Priority 1, Priority 2, and Priority 3 military installations. Only the connectors for Priority 1 and 2 installations are shown in the STRAHNET Atlas, and only those connectors are also on the National Highway System. The main national source of information on STRAHNET is at the DOD Web page at https://www.tea.army.mil/pubs/res/dod/pmd/STRAHNET.htm

7.1.3. Vehicle overweight hazard

To estimate the likelihood of a bridge experiencing vehicles that are over the legal weight allowed, two approaches are presented. The first approach involves the recorded data of vehicle weight violations on bridges while the other uses bridge operating ratings, a measure of the bridge vehicle gross weight capacity.

7.1.3.1. Likelihood of vehicle weight violation

This approach uses data provided by FDOT on the vehicle weight violations on Florida bridges for the time period January 1, 2007 through December 31, 2008, i.e., two years. The data included information as follows: Total Penalty ($); Weight Penalty ($); Date of Citation; Scale City; Violation Type; Actual Weight; and Legal Weight. From this data, containing 322 records, estimates were made of the excess weight recorded over the bridge’s legal weight limit. A summary of the data and the resulting overweight, as a fraction of the legal weight, are shown in Table 7.8 and Figure 7.6. If the data is assumed to be complete for Florida bridges, then it can be assumed that during the period of two years, the 6500 state-maintained bridges on Florida roadways experienced overweight situations 322 times or 161 annually. In other words, the annual probability of excess weight over the legal limit on each bridge is the ratio 161 to

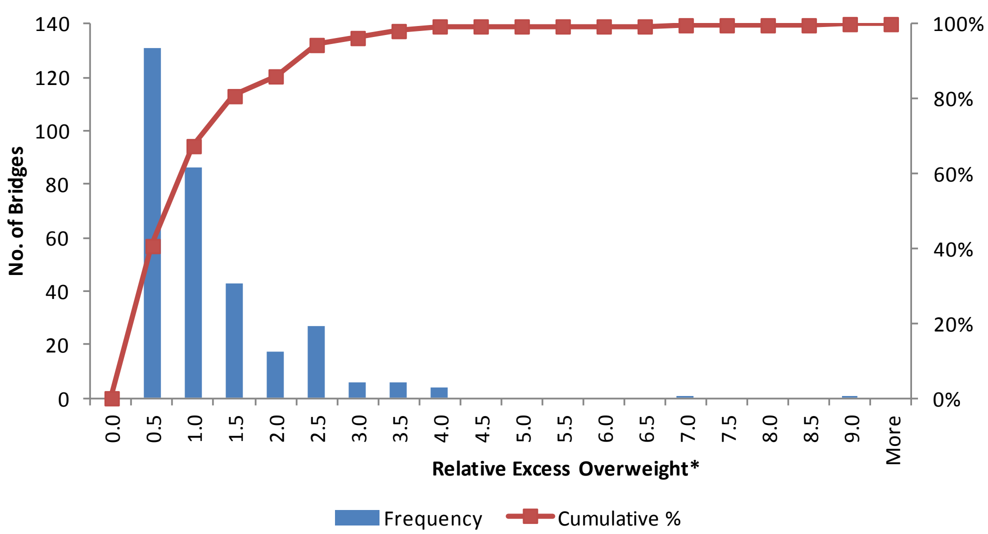
6500 or 0.025. If more explicit data are available, for instance on specific bridges, then the probability estimate can be refined further.

From Figure 7.6 an estimate can be made of the likelihood of getting an amount of excess weight beyond the legal weight on the bridge. For example, there is a 67% likelihood of the vehicle weight exceeding up to an amount same as the legal weight, while 86% of the vehicles will exceed the legal weight by an amount up to two times the legal weight. The data was fitted to a lognormal distribution with the location parameter (μ) and the scale parameter (σ), being estimated as -0.46978 and 0.98629 respectively. The goodness of fit was defined with a reasonable Anderson Darling value of 0.6281. The parameters μ and σ were used to estimate the expected value as 1.02, slightly higher than the arithmetic mean of 0.962 shown in Table 7.9. In other words, vehicle weights on bridges are expected to be excessive by an amount equal to about the legal weight.

It is necessary to again note some limitations in using the results. First, the data on the overweight violations only represent those truckers that get caught, so the number of overweight loads would be higher than shown. In addition, using the average assume that each overweight truck is only crossing one bridge. They would probably be crossing multiple bridges. Therefore, the number of bridges experiencing overweight loads would actually be significantly higher that shown in the report.

Table 7.9. Summary on vehicle weight violations on Florida bridges (2007 to 2008)

Vehicle Parameters	Mean	Std Dev	Min.	Max.	Count
Legal Weight on Bridge (lb)	25074.5	11111.5	10000	68000	322
Actual Weight (lb)	45452.6	19396.1	14800	158200	322
Excess Over Weight (lb)	20378.1	16989.7	400	142200	322
Ratio of Excess to Legal Weight	0.962	0.978	0.006	8.888	322



* Ratio of excess weight to bridge legal weight

Figure 7.6. Variation in vehicle weight violations on Florida bridges

7.1.3.2. Likelihood of reduced operating rating due to age

Based on bridge data for state-maintained bridges that have no record of reconstruction, simple deterioration models are developed for operating ratings relative to the bridge age. Using the functional class designations and the types of material in the superstructure, the various rates of deterioration for the operating rating is summarized in Table 7.10. The regression models for the various sets of bridges are shown in Figures 7.7 to 7.16. The predicted operating rating at a desired age can then be compared to threshold rating (legal load or 35 tons). The blank entries in Table 7.9 are for where there is inadequate data to formulate the regression model while the last column shows the results for all materials in each functional class. By identifying which sets of bridges deteriorate fast, a more detailed model (probabilistic model) can be developed for predicting "overweight" based on the 30 tons legal load, i.e., predicting age when legal load will be exceeded.

Table 7.10. Estimate of the likelihood of reduction in operating rating relative to bridge age

	Superstructure material type						
Functional class	Concrete	Concrete continuous	Steel	Steel continuous	Prestressed concrete*	Prestressed concrete continuous*	All material types
Rural Principal Arterial - Interstate	Slight	None					None
Rural Principal Arterial - Other	Very Significant	Very Significant	None		Moderate	Slight	Moderate
Rural Minor Arterial	Moderate	Very Significant	None	Moderate	Significant	Slight	Moderate
Rural Major Collector	Significant				Very Significant		Moderate
Rural Minor Collector					Significant		Slight
Rural Local					Very Significant		Very Significant
Urban Principal Arterial - Interstate	None	None	None	None	None	None	None
Urban Principal Arterial - Other Freeways or Expressways	Significant		Significant	Very Significant	Very Significant		Very Significant
Urban Other Principal Arterial	Significant	Very Significant	Moderate	None	Moderate	None	Moderate
Urban Minor Arterial	Very Significant	Significant	Significant	None	Slight	Slight	Moderate
Urban Collector	Moderate	Moderate	Moderate	Moderate	Moderate	Moderate	Moderate
Urban Local	Significant	Significant	Significant	Significant	Significant	Significant	Significant

* Post-tensioned concrete coded as prestressed concrete.

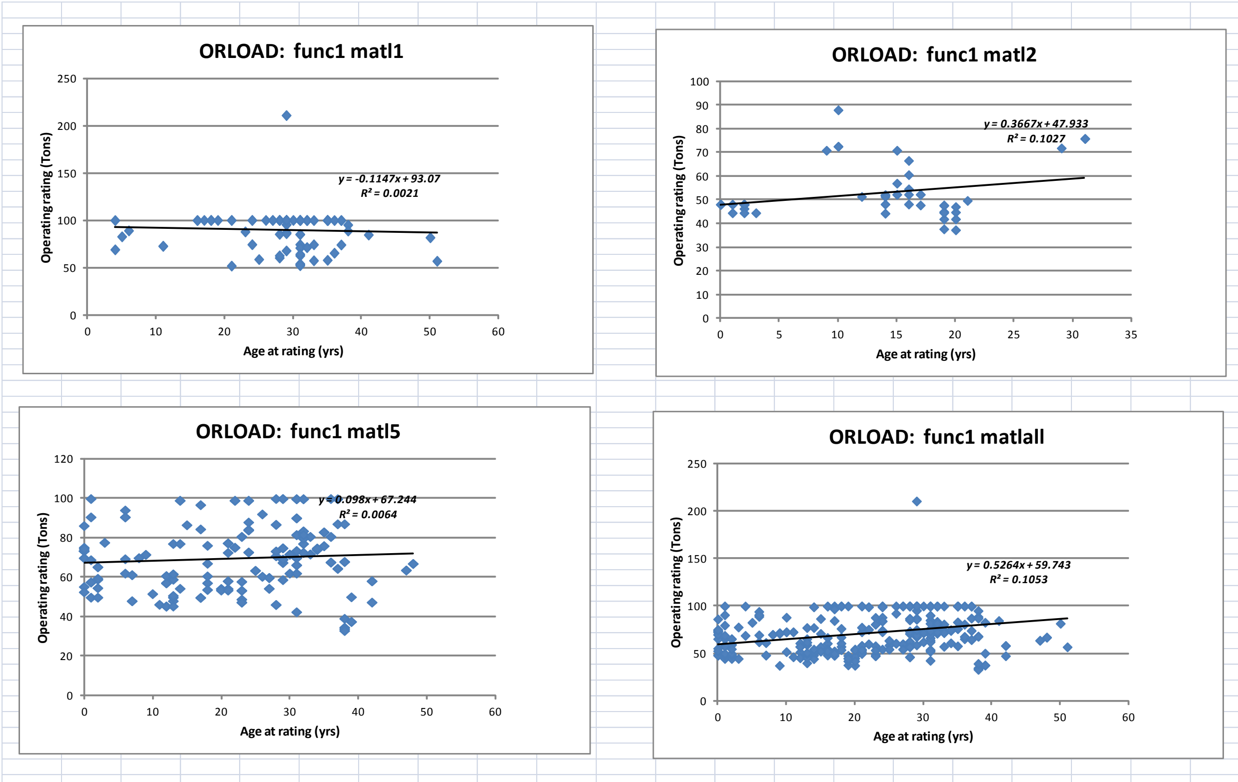


Figure 7.7. Deterioration trends for bridge operating ratings on functional class 1 roadways

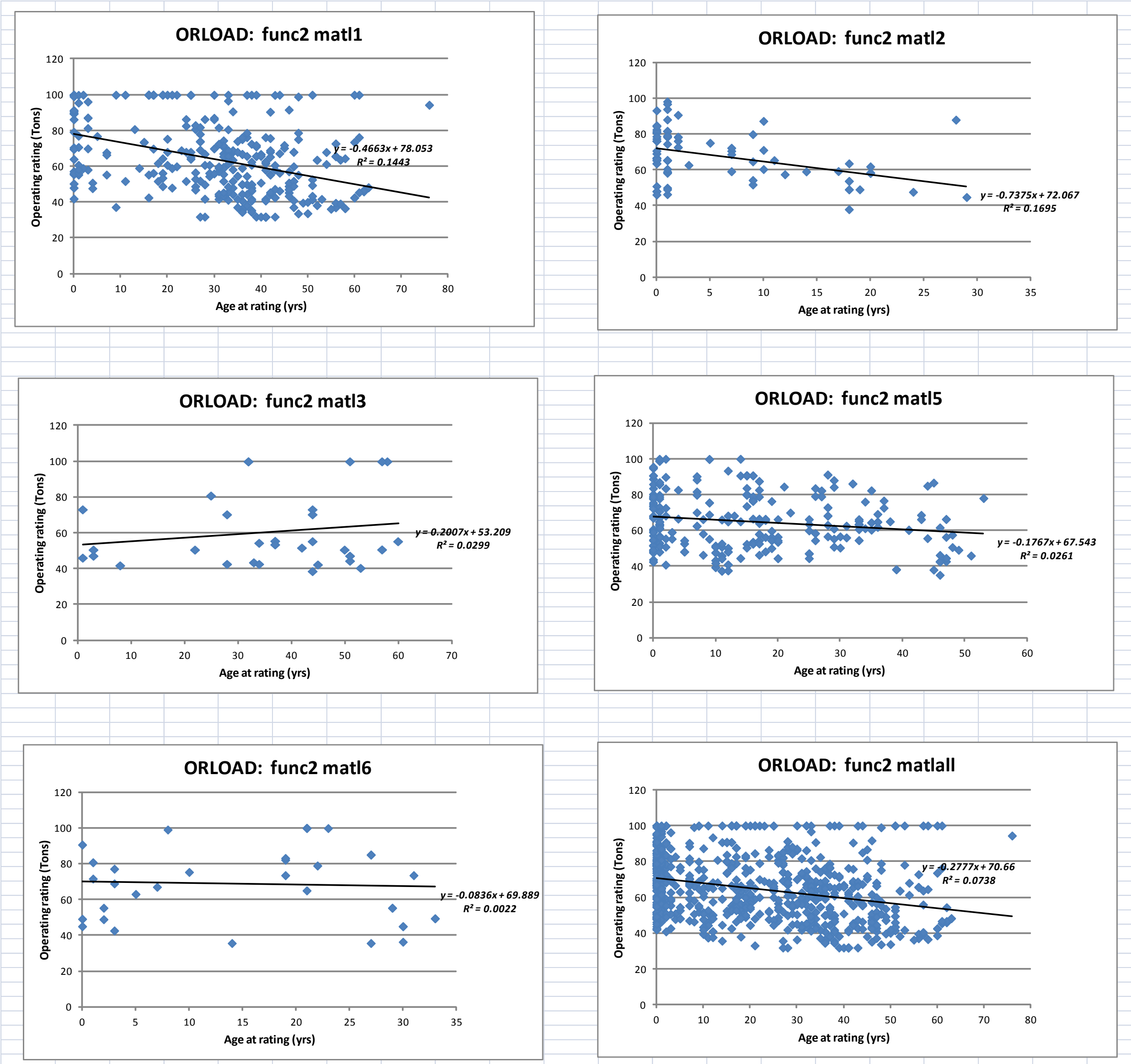


Figure 7.8. Deterioration trends for bridge operating ratings on functional class 2 roadways

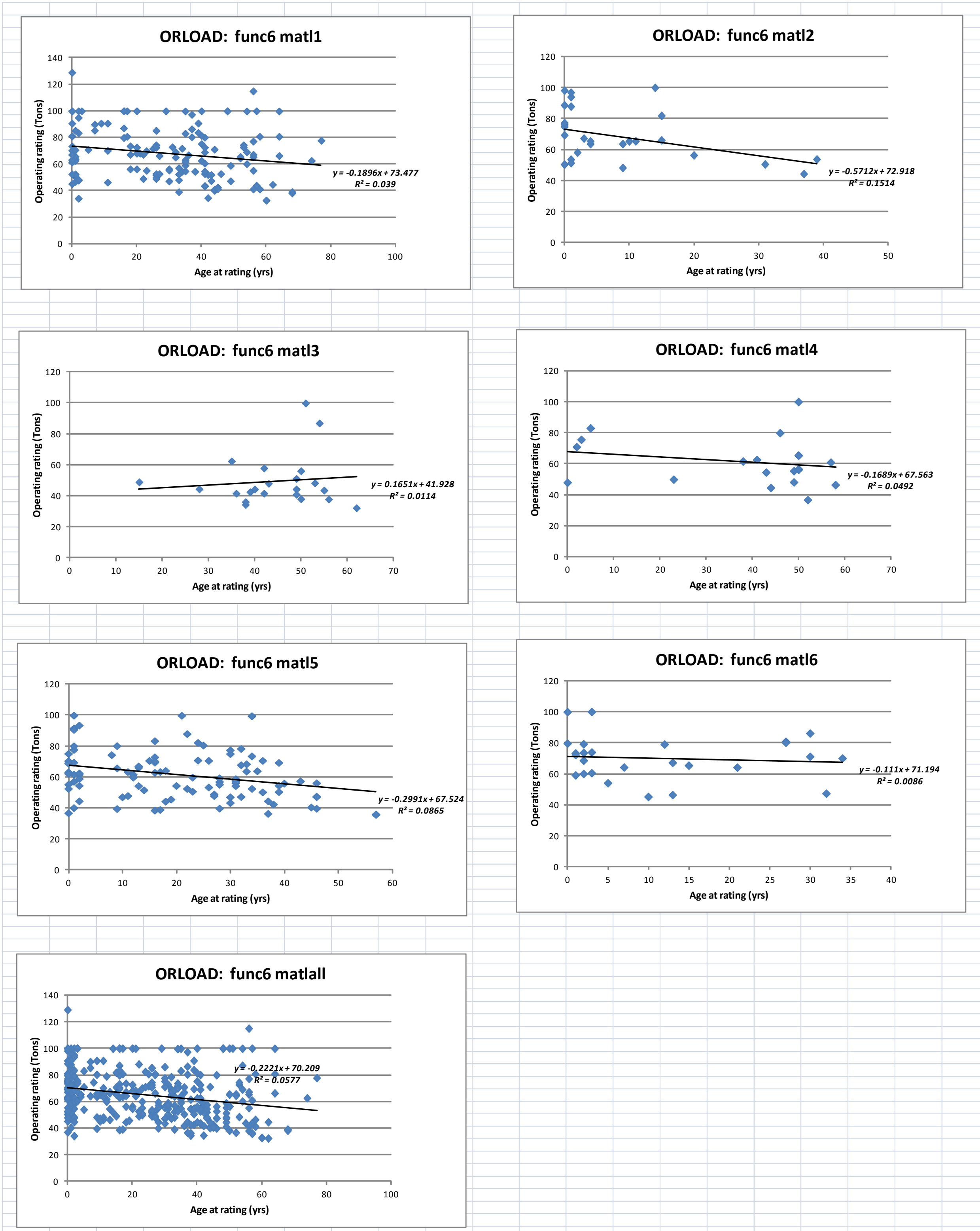
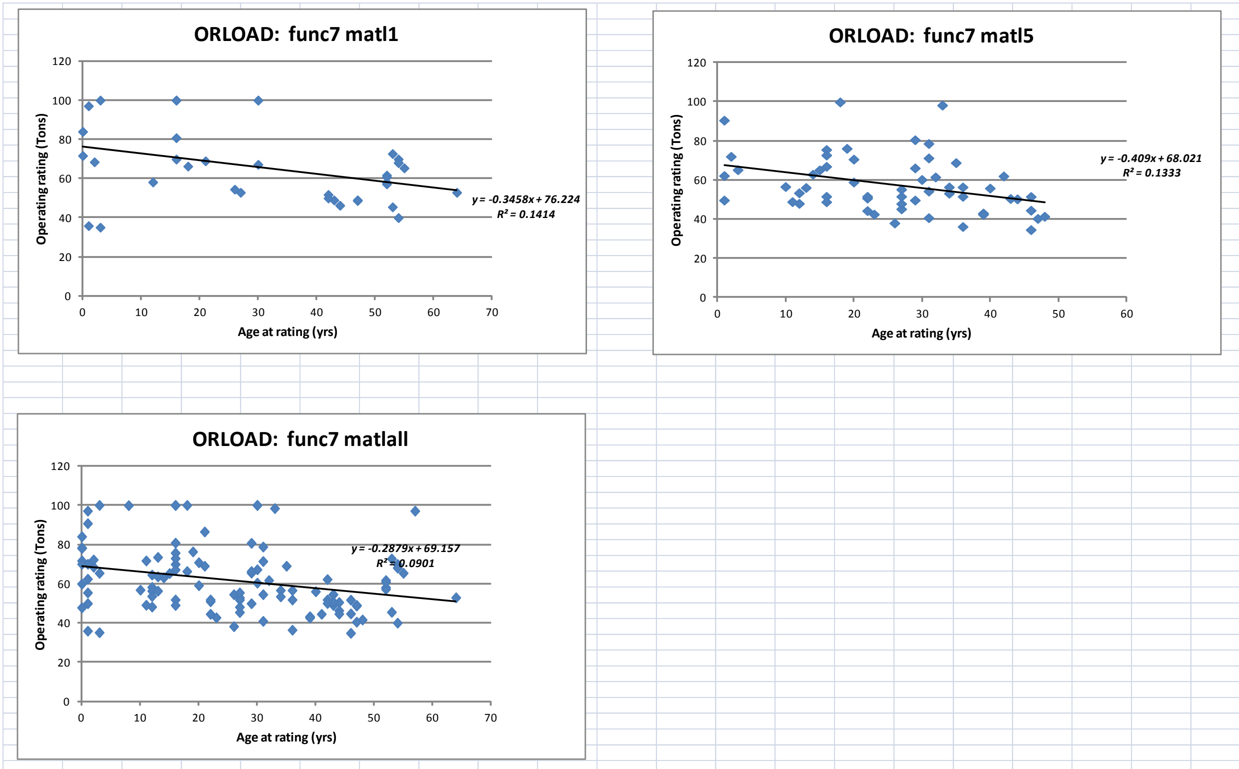
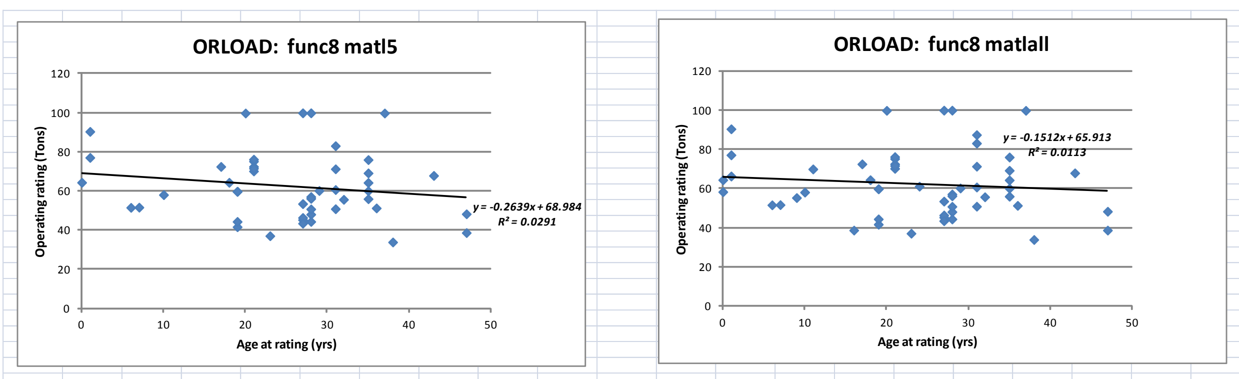


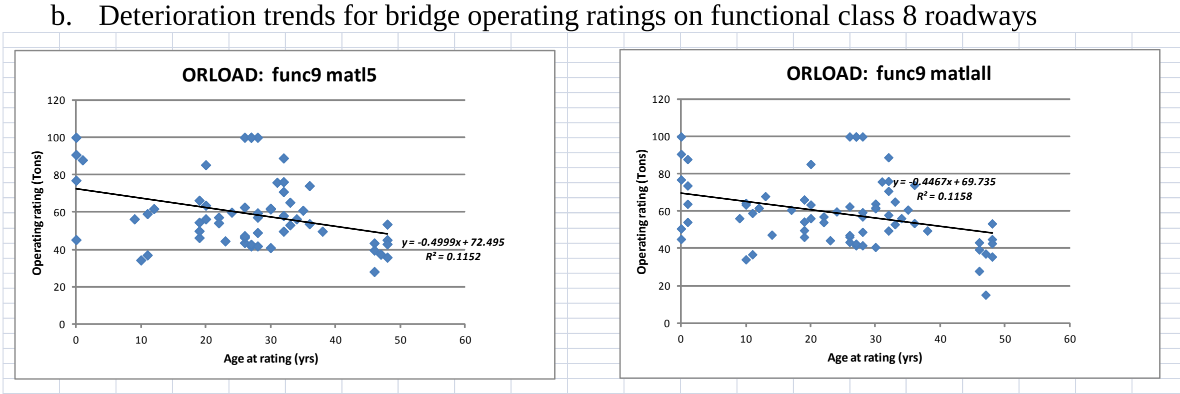
Figure 7.9. Deterioration trends for bridge operating ratings on functional class 6 roadways



a. Deterioration trends for bridge operating ratings on functional class 7 roadways



b. Deterioration trends for bridge operating ratings on functional class 8 roadways



c. Deterioration trends for bridge operating ratings on functional class 8 roadways

Figure 7.10. Deterioration trends for bridge operating ratings on functional class 7, 8, and 9 roadways

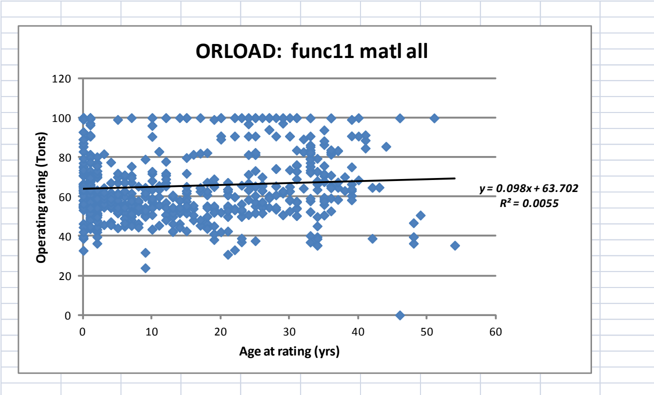


Figure 7.11. Deterioration trends for bridge operating ratings on functional class 11 roadways

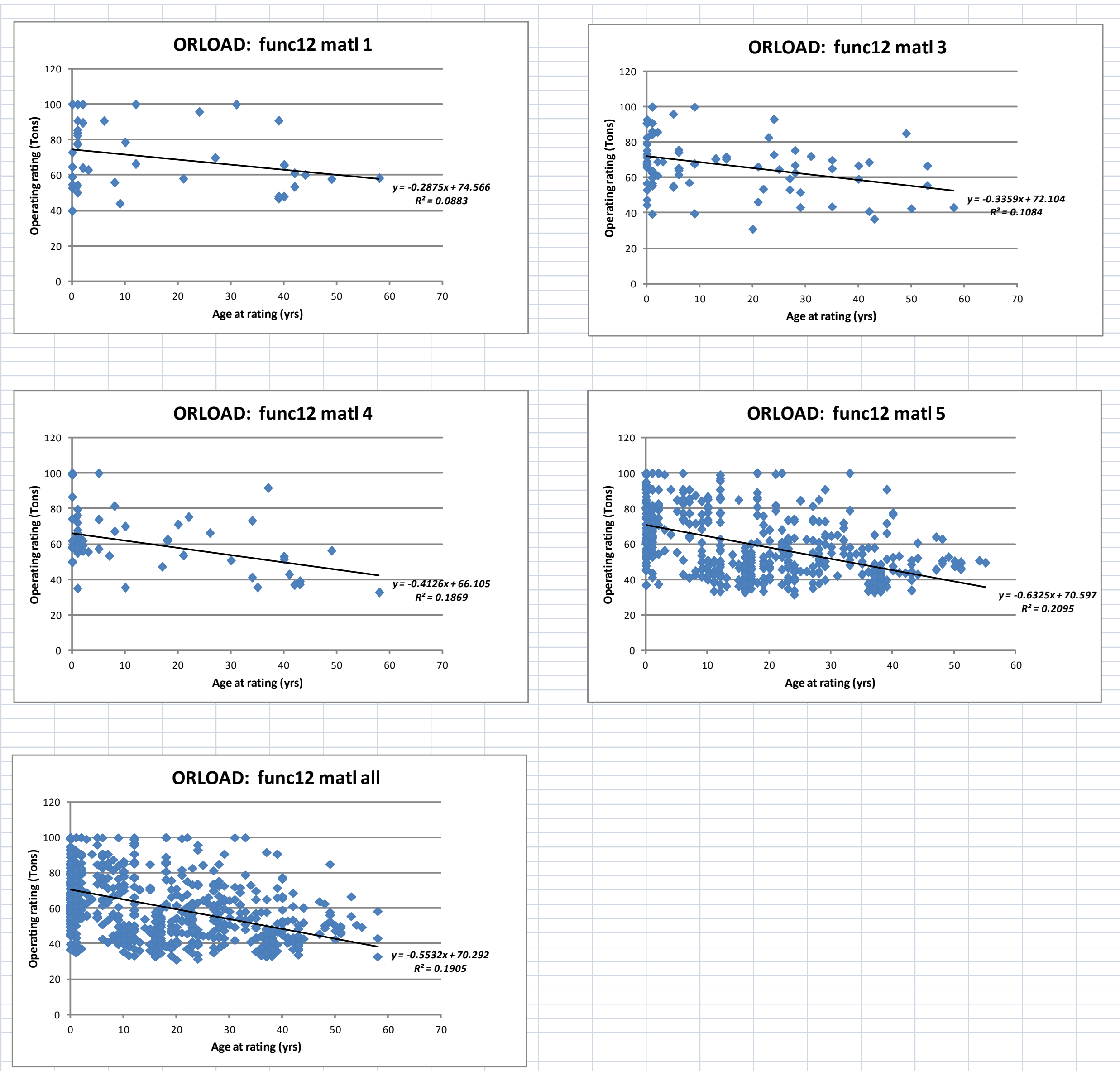


Figure 7.12. Deterioration trends for bridge operating ratings on functional class 12 roadways

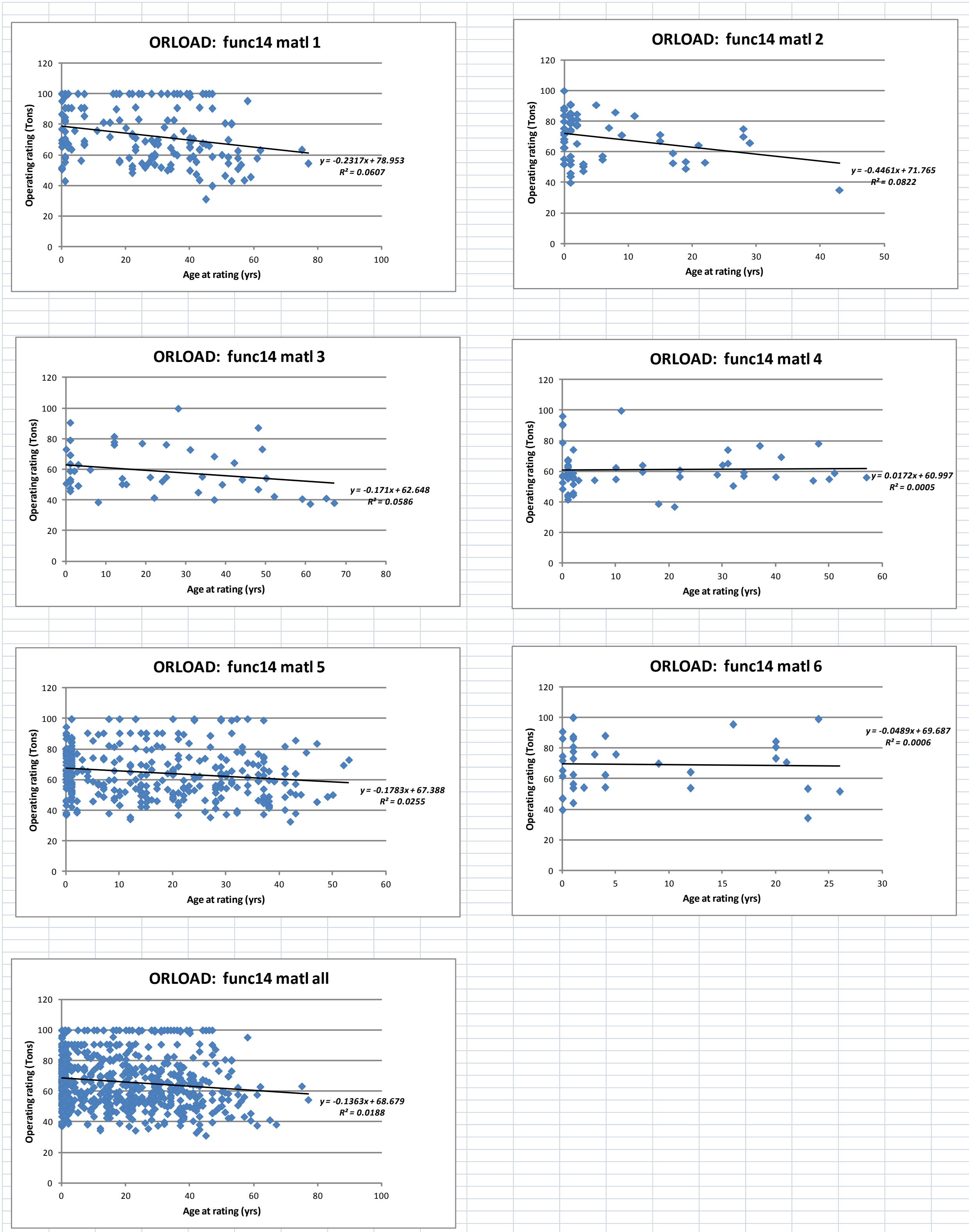


Figure 7.13. Deterioration trends for bridge operating ratings on functional class 14 roadways

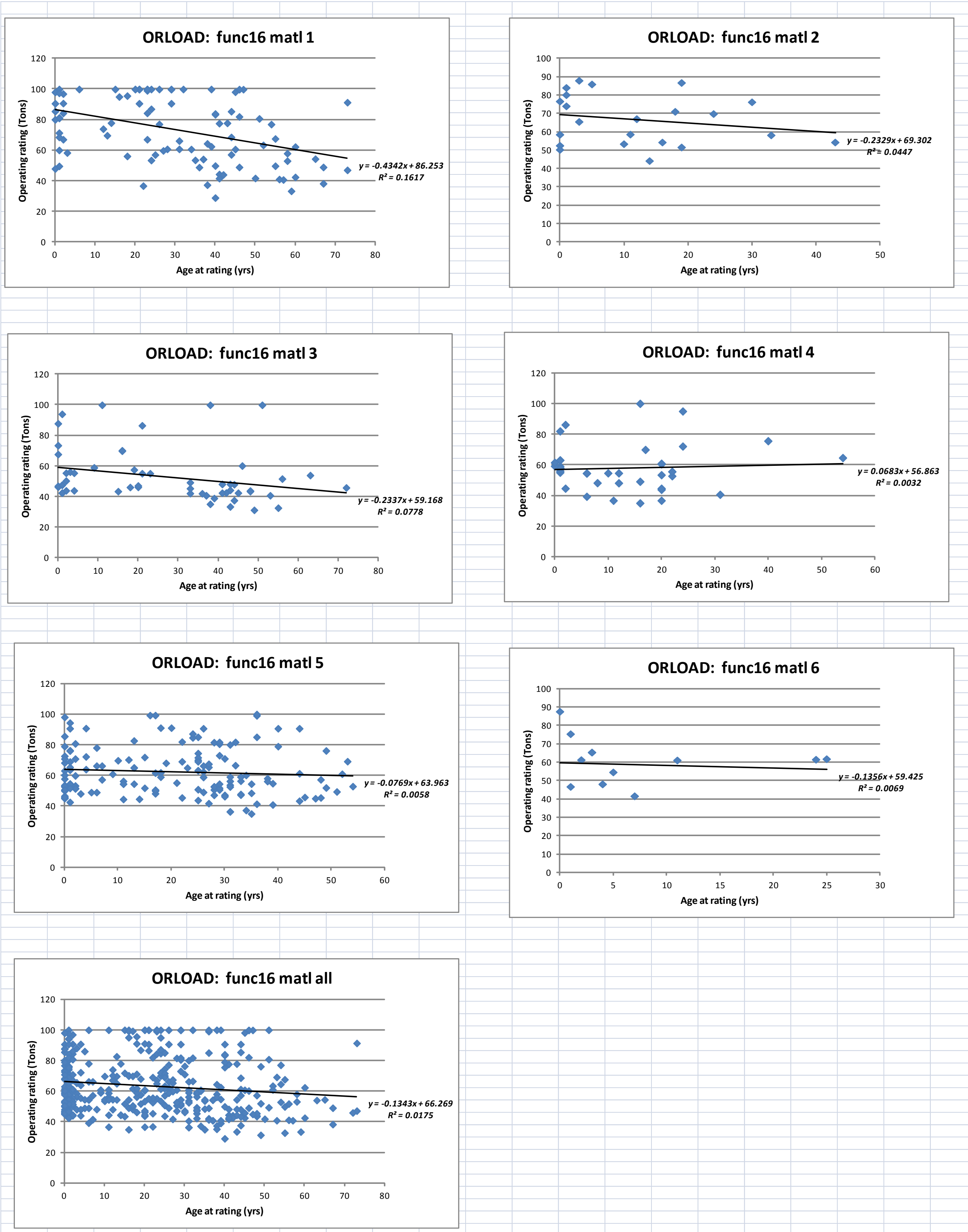


Figure 7.14. Deterioration trends for bridge operating ratings on functional class 16 roadways

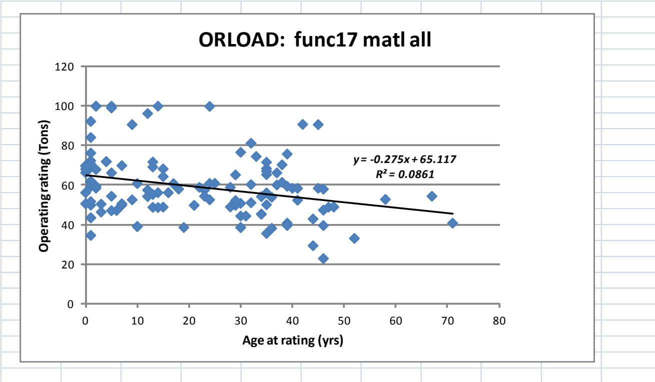


Figure 7.15. Deterioration trends for bridge operating ratings on functional class 17 roadways

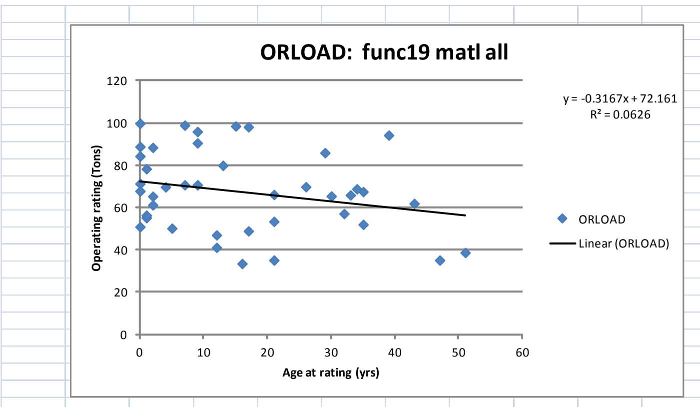


Figure 7.16. Deterioration trends for bridge operating ratings on functional class 19 roadways

7.2. Risk assessment: consequences of accidents and overloads

When hazards occur on bridges, the consequences consist of damage for which the agency is directly responsible (i.e., physical damage to the bridge, which will require repairs or replacement efforts) and other types of consequences related to public user costs (user delays, vehicle operations, and accidents). But first, it is important to note that the extent of the consequences or impact of hazards on bridges is dependent on attributes related to the bridge and its location. This can be referred to as the vulnerability or susceptibility of the bridge to damage from the particular hazard. For instance, a timber bridge is more susceptible to damage during fire from roadway fuel tanker accidents than a reinforced concrete bridge. This measure of vulnerability can be used to enhance the overall estimate of risk of the bridge to a particular hazard. In the examples given above, a bridge that is already very vulnerable will have a higher overall risk than another (less vulnerable) bridge that is exposed to the same likelihood estimate of occurrence of the particular hazard.

It should be noted also that bridges are exposed to fire hazards, with some from minor causes such as electrical equipment malfunctions on movable bridges, vagrants starting fire under the bridge, and wildfires (covered earlier in this report). But the major fire events on bridges have been observed to come from vehicular (truck) accidents. Thus they are presented in this chapter of the report.

7.2.1. Prior reports on consequences

As reported by Lessard (2010), an accident occurred on CR 561 in Lake County, Florida on Monday December 14, 2009 causing fire to an overpass bridge (ID 110070) on the Turnpike highway. A rough sketch of the accident reconstruction diagram is shown in Figure 7.17. This is basically a roadway vehicular accident occurring on the underpass of a bridge, and resulting in fire because it involved a tractor trailer that was carrying flammable material.

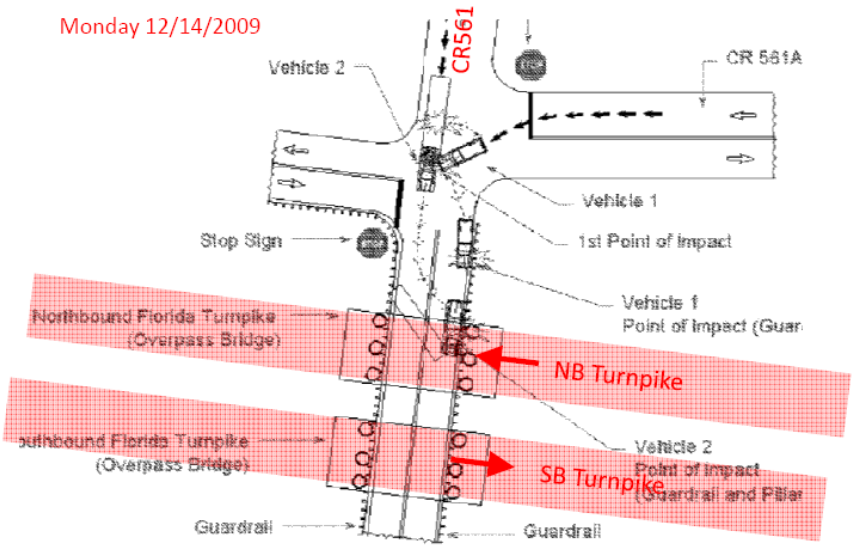


Figure 7.17. Accident reconstruction diagram for fire-related tractor trailer crash on CR 651 in Florida (Lessard 2010)

The damages, partially shown in Figure 7.18, were summarized as follows. On the concrete deck, there were large delaminations with exposed steel, widespread Map Cracking, and areas of Concrete “softness”, easily removed when struck with hammer. Prestressed Concrete Beams had delaminations and strand Exposure along length (5 of 7 beams), and cracking and “softness” along exposed surfaces (webs and flanges). Bearing Pads (Pier 2) appeared “brittle” and flake easily when hit with hammer. Pier Caps and Columns had delaminations with areas of exposed steel. Spans 1 and 3 had Localized hollow areas at beam ends (five to six locations).

The FDOT’s immediate project mission was outlined as follows: Restore Turnpike NB Traffic ASAP by constructing Traffic Diversion with one northbound lane on the southbound bridge; Repair NB Turnpike Bridge 110070 by replacing Span 2 Superstructure; Repair / Strengthen Substructure (Pier Columns and Caps); and Misc. Repairs to Spans 1 and 3. Perform traffic Management by providing 24/7 Roadway. Response through site; Coordinating with Other Agencies; and perform Public Outreach with provision of public Information, Media Briefings, etc.

Figure 7.18. Detailed display of damage to the bridge elements (Lessard 2010)

A study by the New York State Department of Transportation was cited in Kodur et al. (2010) in which reports show that in the U.S., there are three times more bridges that collapse because of a fire as opposed to seismic issues. The majority of fires that occur on a bridge itself are the result of tanker truck collision. If fire engulfs a bridge, the damage that occurs is dependent on the bridge material. Timber is most vulnerable, followed by steel and then concrete.

On April 29, 2007 in Oakland, California, a speeding tanker overturned, dumping 8600 gallons of gasoline which in turn caused an intense fire on I-880 (Figure 7.19). The bridge collapsed after approximately 22 minutes of sustained fire loading. Temperatures during the fire were believed to have reached 2000°F (1100°C). The failure was as a result of the softening of bolts in the connections and girders which caused large deformations resulting in the deck pulling off of its supports (Kodur et al. 2010).

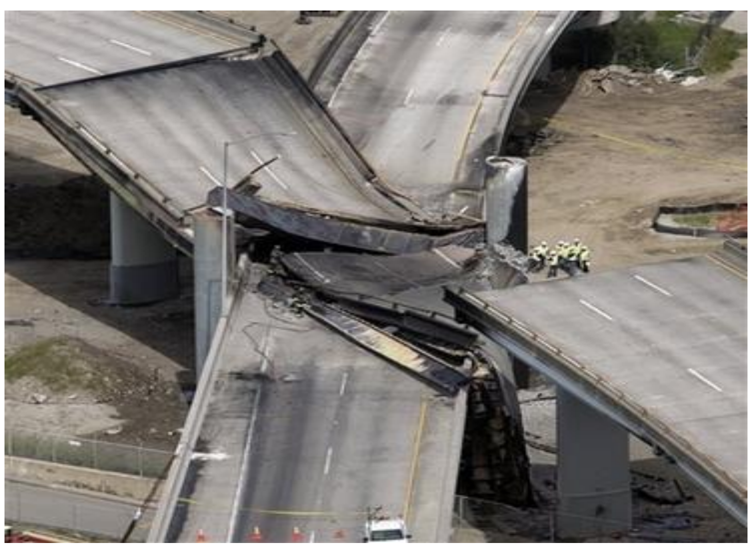


Figure 7.19. San Francisco Oakland Bay Bridge after fire damage

Whenever a bridge is affected by a fire, the bridge is usually closed for an indefinite period of time by the owners regardless of the fire's intensity, so material samples can be tested.

On December 11, 2002, a railroad tanker collision caused a fire under the Puyallup River Bridge, a prestressed concrete girder bridge that consumed 30,000 gallons of Methyl Alcohol (Figure 7.20). A high flame temperature was maintained for about an hour, and span 8 was completely engulfed by the fire. Road closures occurred on the interstate freeway and remained closed pending an all night structural inspection. The bridge displayed no unusual deflections or misalignments and was reopened on December 12th to commuter traffic and legal weight trucks, excluding overweight trucks which were re-routed.

Figure 7.20. Puyallup River Bridge Railroad tanker fire (Stoddard 2004)

Fire damage to the bridge included a 2” deep concrete spalls that exposed the spiral reinforcement for a full height of the pier on Pier 9. Further analysis showed delaminations within the concrete core inside the spiral cage and vertical reinforcement. All the girders were damaged in Span 8, and there were concrete spalls in the top flange and webs of girders (Stoddard 2004).

Timber bridges are used because of the low cost, ease of construction, the reliability of performance, and sometimes aesthetics to match the environment in which the bridge is built in. Although through advances in technology, wooden and timber bridges are being built with the ability to withstand certain temperatures of fire, there are a lot of bridges that were built before the availability of such technologies, and therefore usually in the event of a wooded or timber bridge coming in physical contact with fire, there is usually a total loss of the bridge structure (Figure 7.21). Modern technological design implementations try to ensure that when a larger structural timber is exposed to fire, there is some delay as it chars and eventually flames. As the burning continues, the charred layer has an insulative effect, and the burning slows to an average rate of about 1/40 inch (0.6mm) per minute (or 1 ½ (38mm) per hour), for average structural timber species. These slow rates of fire penetration mean that timber structural members subjected to fire maintain a high percentage of their original strength for considerable periods of time (vermontlocalroads.org).

Figure 7.21. Fire consumption of timber bridge Washington State – WSDOT

The main lesson learned about consequences of fire is that steel and timber bridges are more susceptible to severe damage from fire hazard while concrete bridges may suffer moderate /severe damages. It can be reasonably assumed that timber bridges would suffer the most among the three types of bridges.

7.2.2. Estimate of consequences

As described in the section for hurricane hazards, a review of the inspectors' comments in Pontis and in the survey results was performed to develop a database of the impact of the hazard events on each structure identified. Table A5.2 in Appendix A5 shows a summary of the results, but limited to those structures with cost or roadway closure duration estimates available. After an analysis of the fire damage data, a summary of the count of damages, by specific bridge element or bridge component type, is presented in Table 7.11. It could be seen that the elements that suffer most damage were the deck and superstructure elements, followed by the columns/piles and substructures.

A review of all the described damages due to fire on Florida bridges was utilized to develop the scheme shown in Table 7.12, showing the criteria used in classifying damages experienced by Florida structures during the fire hazard events. This set of criteria was then used to assign levels of fire damage observed on the structures in each event, as summarized in Figure 7.22. Here it is seen that most of the bridges (almost 80%) suffered slight damage, with fewer than 10% having moderate damage, while almost 20% suffered the extensive level of damage. These bridges with extensive damage were all from vehicular accidents on the roadway. From the records available, no bridge was completely replaced due to fire damage. A complete list of the fire damages on the bridges as well as the extent of damage and associated costs (if available) is shown in Table A5.5 of appendix A5. For comparison, the information on damage from fire as reported for bridges outside Florida is shown in Table A5.6.

Records showed that cost of repairs on fire-damaged bridges in Florida were as follows: for a bridge damaged at moderate level it cost $410,000; and for six bridges damaged extensively, it costs $89,852, $375,000, $477,794, $477,794, $1,917,100, and $2,300,000 to repair each bridge. It can be summarized that showed bridges that suffered moderate damage from fire, cost $410,000 per bridge to repair while those at extensive damage cost $939,590 per bridge. In terms of roadway closures resulting from fire events on Florida bridges, there was limited data available. It was reported form six cases of fire events that the average closure duration was 179 hours ranging from six hours to 576 hours. These were all from vehicle accidents-caused fires.

Table 7.11. Summary of bridges by element types damaged during fire hazards in Florida

Element Type	No. of Bridge elements affected
Decks and Slabs	20
Superstructure	19
Columns and Piles	15
Pier Walls and Abutments	7
Substructures, incl. Caps and Footings	11
Expansion Joint	2
Bearings	4
Drainage System	4
Railing	7
Fender and Dolphin	5
Abutment Slope Protection	1
Bulkhead and Seawall	1
Conduit (Elem 572)	1
Control (Elem 574)	1
Navigtional Lights (Elem 580)	3

Table 7.12. Established scheme for classifying levels of damage to structures during fire hazards in Florida

	Levels of Damage			
Hazard Type	**Slight**	**Moderate**	**Extensive**	**Complete**
Fire	Damages such as paint peeling, black soot coloring, charring, small-sized spalls, delaminations (no exposed steel) on structural elements, damages to non-structural elements such as railings. Poses no serious structural problems. Includes minor fire damage to planks/stringers, andelectrical conduit/wiring on fenders. Maybe due to vehicular crashes, electrical fire (fenders), wildfires, gunfires, or vagrants setting fires under bridge. Structure may need minor repairs.	Damages such as moderate size spalls, extensive surface scaling, and delamination on large areas (with exposed steel) on structural elements. Poses serious structural problems. Structure is repairable.	Severe damage to deck, beams, caps, and columns, including widespread spalling, strand loss (prestressed concrete) and major section loss. Damage may include non-structural elements (signs, railings, etc.). Structure is repairable. Poses serious structural/functional problems. May require partial or full replacement of bridge span(s).	Severe damage to all or critical structural and non-structural components. Structure need to be completely replaced.

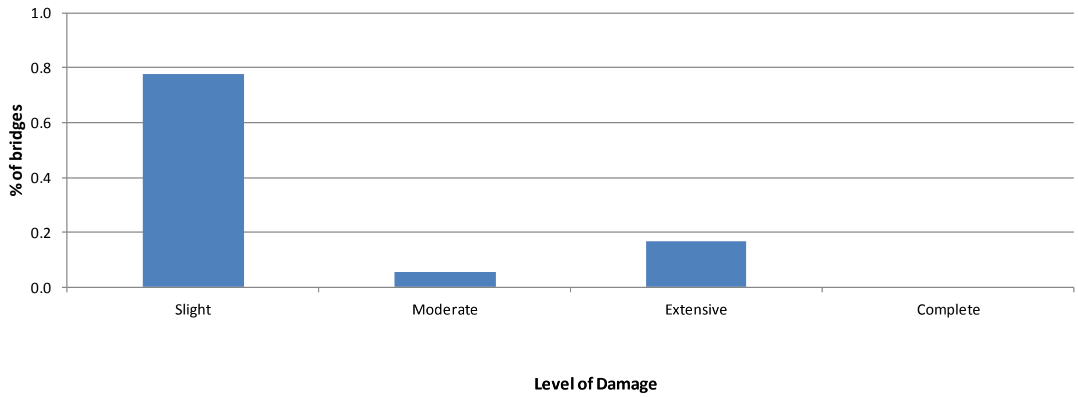


Figure 7.22. Levels of fire damage to bridges

A review of documented accident inspections on bridges (Pontis *inspevent* table) also revealed various types of accidents that could be classified into three categories: roadway (surface) crashes, over-height crashes, and vessel impacts. The roadway crashes imply those crashes that occurred on the traveling surface of the roadway, while over-height crashes mean those involving vehicles that had heights higher than or equal to the available underclearance at the bridges. The vessel impacts are for the cases where vessels (barges, boats, ship, etc.) on the waterway impacted the bridge substructure elements.

The Pontis *inspevent* table data was reviewed, with emphasis on the inspector's notes field, to ascertain and compile for each bridge record, pertinent information as follows: BRKEY (bridge ID), date, time, type of accident, elements inspected, deck/deck overhang/slab Damage, beam damage, barrier damage, guardrail/railing damage, column damage, fire involved, barge impact/fender damage, movable bridge damage, sign/electrical damage, misc. damage, MMS Ref#, no damage, no repair requested, comments, and inspection date. A total of 398 bridge records have been reviewed and the results summarized in the following sections, with Table 7.13 showing the breakdown into the types of accidents. The dates and times were indicated on most of the crash records but for a very few records with no date and time indicated, the inspection date was assumed for the date. The crash records data had dates in the range of 12/16/1998 to 7/8/2008.

Table 7.13. Types of crashes on bridges

Accident Type	Number of crashes	% of crashes
Roadway crash	207	52.0%
Roadway (Under-route) crash	1	0.3%
Vehicle Overheight crash	154	38.7%
Vessel Impact crash	31	7.8%
Unknown (Vehicular Impact)	2	0.5%
Fire (Fender)	3	0.8%
Total	398	

Also reviewed was the frequency of each type of crash at specific bridges, as shown in Figure 7.23. Most bridges experienced only one roadway crash within the period of observation, but there were multiple occurrences at some bridges. Two bridges each experienced more than 10 crashes. For over-height crashes, most of the bridges specifically experienced one crash during the observation period, while three had over 10 crashes each. Most bridges experienced only one vessel impact crash while one bridge experienced 12 crashes.

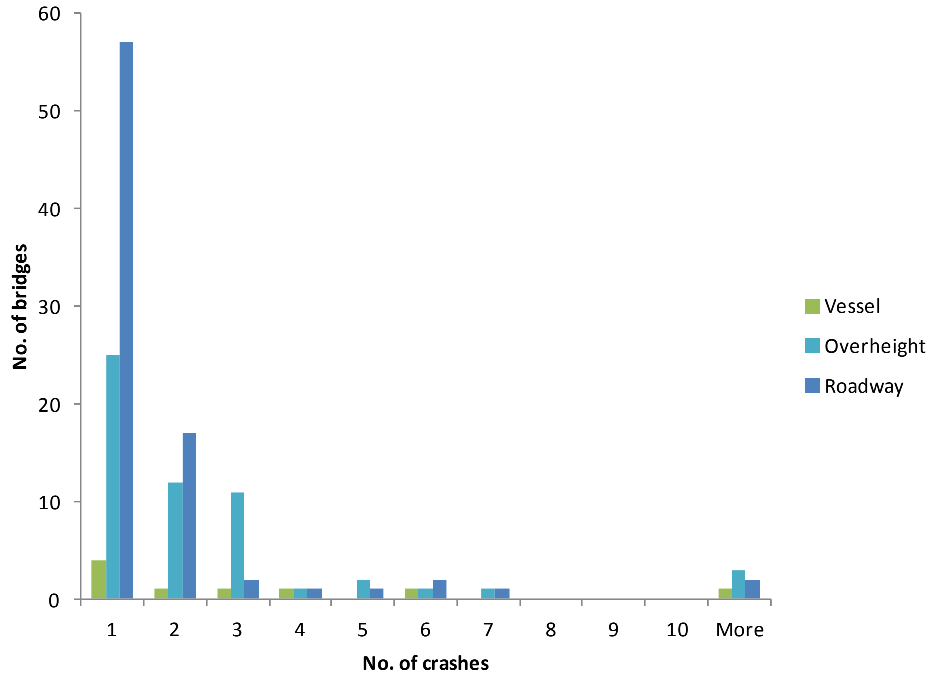


Figure 7.23. Frequency of roadway, over-height, and vessel impact crashes at specific bridge locations

7.2.2.1. Roadway crashes

The consequences of roadway crashes on bridges can be estimated as affecting various element types as shown in Table 7.14. It appears that truss members and railing elements are most affected by roadway crashes.

It was difficult to establish the damage levels of roadway crashes based on the limited information. But some rough inferences can be made. Examples of the roadway crashes are shown in Table 7.15 in a set of criteria that could be used to classify damage levels. Of the 207 crashes, 29 or 14% had clear indication that there was no damage or that there was no need for repairs. Fire was involved in eight (about 4%) of the crashes, with two cases having severe damage to beams, and three crashes causing severe damage to piers, columns, and caps. For 107 of the 207 crashes (about 52%), in which MMS reference numbers were indicated, this may imply that there was an effort to do some repairs. Such crashes can be assumed to have caused moderate level damage.

Though not the ideal statistical situation, the data in Table 7.14 can be approximated as the probability of a bridge element being damaged in a roadway crash, as illustrated in Figure 7.24 Again, it could be seen that handrails, bridge rails, and railings, in general, are damaged about half of the time during roadway crashes; truss members and parts of deck or slab are also affected at about 16% and 7% of the time.

Table 7.14. Effect of roadway crashes on bridge elements

	Element Type Affected	No. of element in crashes	% of all elements in crashes
Core Elements	Decks and Slabs	16	6.7%
	Beams: Girders and Stringers	4	1.7%
	Truss	39	16.3%
	Columns	7	2.9%
	Abutment and Walls	7	2.9%
	Caps	5	2.1%
	Channel	3	1.3%
	Joint/Seal	3	1.3%
	Bearing	2	0.8%
	Approach Slab	4	1.7%
	Handrail/Bridge rail/Railing	117	48.8%
	Dolphin	1	0.4%
	Slope Protection	1	0.4%
	Sign/Sign Structure	1	0.4%
	Moveable bridge elements	7	2.9%
Non-core Elements	Guardrail	5	2.1%
	Barrier	13	5.4%
	Fence	1	0.4%
	Wingwall and backwall	4	1.7%
	Totals all elements in crashes	240	

Table 7.15. Established damage levels to bridges due to roadway crashes

Level of Damage	Bridge ID	Incident Description	Elements Affected	Comments	MMS No.#
Severe	130103	a tanker truck southbound on I-75 crashed into the US-301 median, burning spans 39 and 40 and pier 40.	12, 302, 331, 109, 310, 205 and 234	Spans 39-40 are totally unrepairable.	
Moderate	390009	unknown vehicle traveling South on SR-121 struck the guardrail transition areas at the Northwest and Southwest corners of the structure. This impact caused damage to the concrete bridge railing, guardrail, backwall at Abutment 1, and the curb drainage inlet next to the North approach slab.	331, 334, 215, 321		9190721
Minor	720184	vehicle traveling northbound on I-95 struck the median barrier wall in Spans 1, 2 and 3. There are minor surface spalls at the top of the median barrier wall in Spans 1, 2, and 3.	Barrier	no repairs are being requested.	

FDOT's Maintenance Management System (MMS): work order prepared for repair.

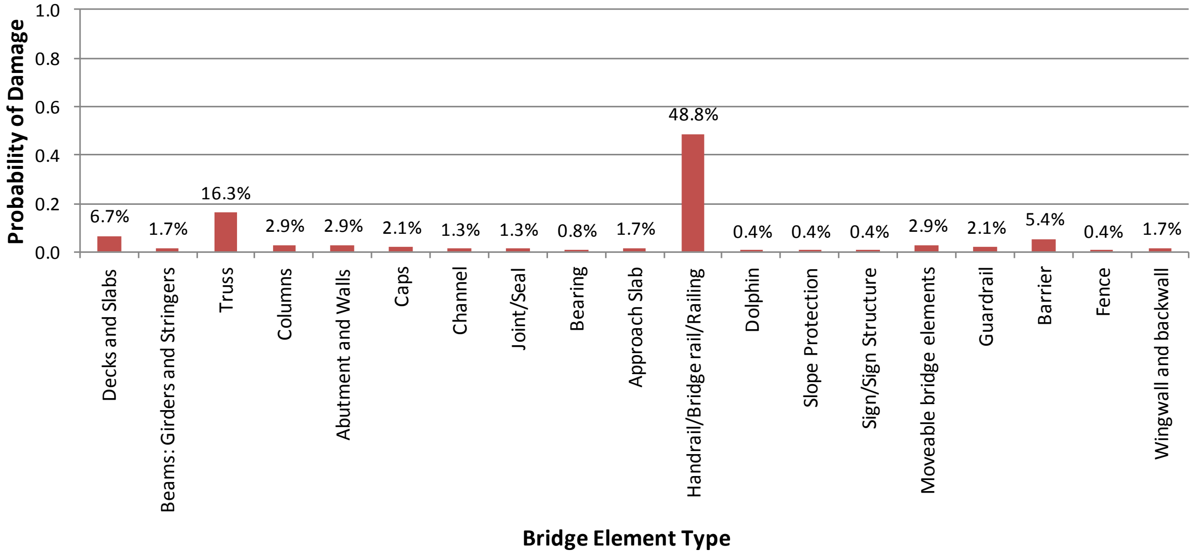


Figure 7.24. Probability of damage on bridge elements during roadway crashes (240 elements/207 crashes)

7.2.2.2. Vehicular over-height crashes (under-route roadways)

Over-height crashes are the vehicular crashes where vehicles, mostly trucks traveling on the roadway under a bridge, are too high for the bridge underclearance. Truck height histograms from prior FDOT BMS studies may be helpful to estimate the likelihood of vehicular over-height crashes, but recognizing that most over-height trucks successfully find alternative routes. Examples of the damage levels as observed in the data are shown in Table 7.16, and the number of bridge crashes at each level is shown in Table 7.17. Of the 154 recorded bridge over-height crashes, about 60% resulted in moderate damage to the bridge elements while about 30% of the bridges suffered minor or no damage. Only two bridge crashes resulted in extreme level damage.

A summary of the bridge elements affected by this type of crash is shown in Table 7.18 and Figure 7.25. As expected, the beams and girders are most affected while the deck or slab may be occasionally affected.

Table 7.16. Established damage levels to bridges due to vehicular over-height crashes

Level of Damage	Bridge ID	Incident Description	Elements Affected	Comments	MMS No.[#]
Severe	010088	track hoe mounted on a semi rig lowboy trailer traveling northbound struck this bridge at span 5 south side, midspan. The collision severely damaged the span necessitating closure of Rampart Blvd over I-75 by first responders. The roadway underneath also sustained some damage.	98, 109		
Moderate	720092	northbound long-bed dump truck struck and damaged/destroyed 2 of 6 beams in Span 3.	109		9190714
Minor	720177	unknown overheight vehicle traveling westbound on 20th St. struck Beams 2-1, 2-2, 2-4, 2-6, 2-14, 2-16, 2-18, 2-20 and 2-24. The bottom east flange of these beams have spalls averaging 12 in. x 4 in. x 1/2 in. with no exposed steel.	Beams	no repairs are requested	
No Damage	290082	an overheight vehicle was reported to be stuck under one of the overpasses at I-75 and US-441 in Columbia County. Beams 2-9 and 2-1appeared to have the most recent damage which consisted of minor scrapes on bottom flange, over outside lane, of Northbound 441.	109	No damage. No evidence of damage/debris on under route roadway.	

[#] FDOT's Maintenance Management System (MMS): work order prepared for repair.

Table 7.17. Levels of damage on Bridges from vehicular over-height crashes

Level of damage	No. of bridge crashes	% of bridge crashes
No Damage	5	3.2%
Minor	43	27.9%
Moderate	90	58.4%
Severe	2	1.3%
Unknown	14	9.1%
Totals all crashes	154	

Table 7.18. Effect of vehicular over-height crashes on bridge elements

	Element Type Affected	No. of element in crashes	% of all elements in crashes
Core	Deck and Slab	12	7.4%
	Beam: Girder and Stringer	137	84.0%
	Column	1	0.6%
	Abutment/Retaining Walls	1	0.6%
	Bearing	1	0.6%
	Railing	5	3.1%
	Signs, Electrical conduit/junction box.	6	3.7%
	Totals all elements in crashes	163	

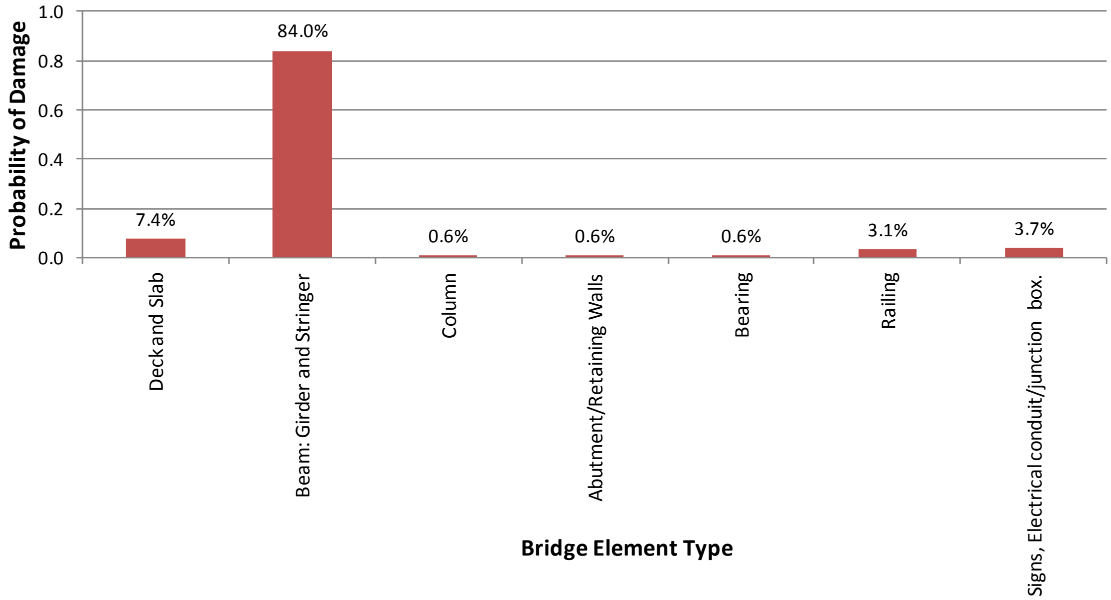


Figure 7.25. Probability of damage on bridge elements during vehicular over-height crashes (164 elements/154 crashes)

7.2.2.3. Vessel impact crashes

Due to the limited size of the data on vessel impact crashes, set it was difficult to establish a level of damage to assess the crash cost, but examples of such levels from the data are shown in Table 7.17. It was observed that of the 31 bridge vessel impact crashes recorded, 26 bridges had their fenders affected, with one case demanding a "declaration of emergency" for removal of damaged portions of the fender system; this could be classified as a severe damage. Most damages to the fenders were described typically as "damage solely to the fender system and consists of broken piling, horizontal wales, catwalk planks, and misalignment of the west fender…" "..struck and damaged the north and south fenders; The southwest fender was destroyed.." The plan to repair, as indicated by MMS numbers on the records, was also noticed on 16 cases of fender damage. In terms of the bridge elements affected, the fenders were clearly identified as the predominant element (Table 7.20 and Figure 7.26), being affected about 60% of the time. Conduit and junction boxes, as well as navigational lights are also slightly affected, in about 14% of the time.

Table 7.19. Established damage levels to bridges due to vessel impact crashes

Level of Damage	Bridge ID	Incident Description	Elements Affected	Comments	MMS No.[#]
Severe	720061	a northbound barge and tug "Miss Sarah" struck and damaged the Fender System. The impacted areas were in good condition prior to this accident. The U.S.C.G. investigated this accident. A Declaration of Emergency was issued on 10/21/04 for removal of damage in the navigational channel to the Fender System.	386, 572, 580		9190624
Moderate	720069	a northbound tug boat "Coption Muller" pushing 2 barges struck and damaged the southwest fender.	fender		9190528
Minor	720005	a barge and tug "Bull Dog" scraped the south fender. No damaged noted. No incident report was sent and no repairs requested.	fender	no repairs requested.	

[#] FDOT's Maintenance Management System (MMS): work order prepared for repair.

Table 7.20. Effect of vessel impact crashes on bridge elements

	Element Type Affected	No. of element in crashes	% of all elements in crashes
Core	Deck overhang	1	2.3%
	Railing	1	2.3%
	Beam: girder, bascule span, chord	3	6.8%
	Fender	26	59.1%
	Conduit & Junc. Box	6	13.6%
	Navigational Lights	6	13.6%
	Platform	1	2.3%
	Totals all elements in crashes	44	

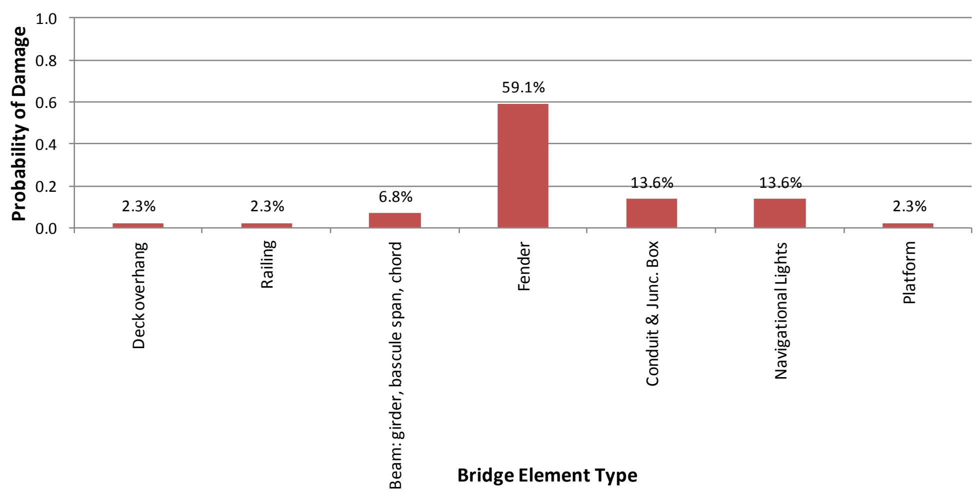


Figure 7.26. Probability of damage on bridge elements during vessel impact crashes (44 elements/31 crashes)

7.2.2.4. Agency costs of accidents

Many of the crash records indicated the MMS numbers associated with the repairs of bridge damage. Unfortunately, the available MMS cost data did not have any of the MMS numbers shown on the data. All the MMS numbers started with "9" and none of the available MMS data (from just prior FDOT BMS study on costs) started with "9." Apparently this MMS numbers indicated inspection effort after the accidents and not the repair effort itself. Available MMS cost data were reviewed using the MMS Activity No. 888, used for "Bridge Damage Repairs" and defined as being done for accident repairs. The repair activities were verified in the activity description of each repair done. Some estimates were derived for the total cost of damage repairs done at each bridge, based on the limited data (100 bridge repairs recorded for this MMS activity number).

The summary is shown in Table 7.21 for the major types of repairs observed in the data and illustrated in Figures 7.27 and 7.28. Bridge beam repair costs are shown because this was the most common type of repair found in the data. Overall there is considerable variation in the costs as indicated by the standard deviations and coefficients of variation. This is not unexpected given that there are many unknown variables behind the costs of these repair activities. But it could be seen that 80% of all bridge accident repairs cost less than about $2,000, while the same percentage costs about $1,600 for bridge beam accident repairs. A detailed list of the repair costs is shown in Appendix A5.

Table 7.21. Summary of bridge accidents repair costs from FDOT's MMS Cost Data

Bridge Element Type	Mean Cost ($)	Min Cost ($)	Max Cost ($)	Std Dev ($)	Coeff. Of variation	Count
Barrier	2,578.41	310.45	6,460.00	2,333.83	0.91	6
Beam	1,203.80	16.64	8,530.13	1,686.86	1.40	41
Fender	1,272.28	51.16	4,224.89	1,993.72	1.57	4
Guardrail/Handrail/Post	1,440.48	33.21	8,045.78	2,081.27	1.44	28
All Bridge Elements	1,307.70	14.55	8,530.13	1,844.27	1.41	100

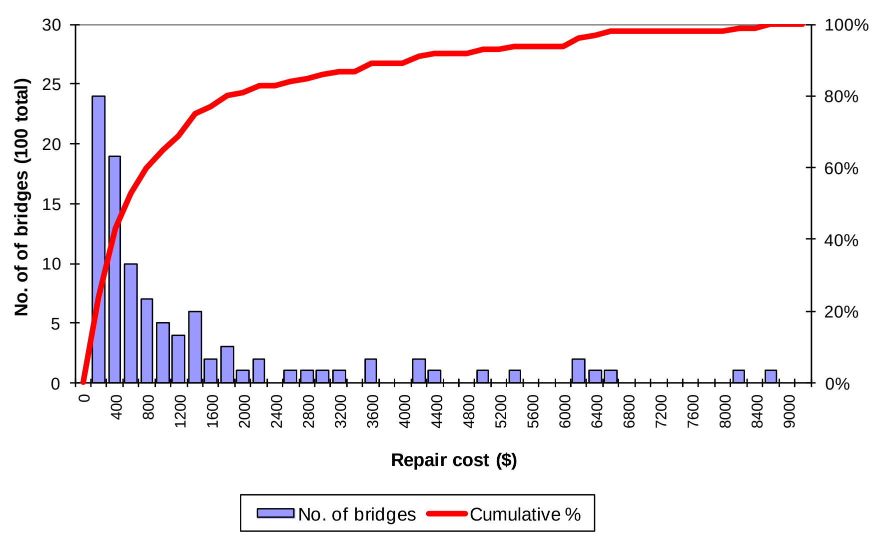


Figure 7.27. Variation in bridge accidents repair costs

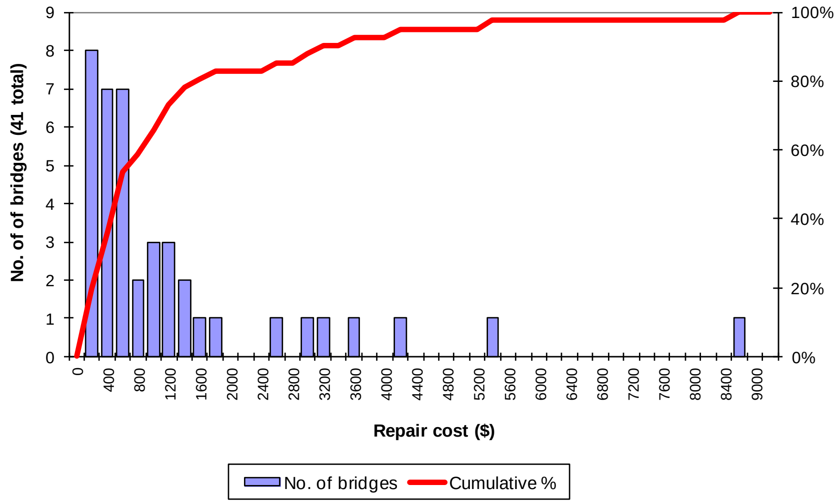


Figure 7.28. Variation in bridge beam accidents repair costs

Additional efforts were made to obtain agency costs of repairs at FDOT. Based on FDOT District Two data for 322 bridge accidents, the following results in Tables 7.22 and 7.23 and Figure 7.29 were estimated. The task description was used to assign a perceived level of damage from the accident, i.e., negligible, slight, moderate, or severe. This can be considered subjective. Cost summaries at each level of damage observed were estimated for each bridge element in the records. It should be noted that these costs had no specific dates assigned to them, but the data was for bridge repairs done from 2002 to 2012. For elements measured in linear feet or square feet, the unit costs are directly useful, but in most cases the total costs are more applicable, especially where the unit of measure is man-hours. Also, although there are a few matches of the bridge IDs on the cost data with those of the accident inspection list, there is no certainty in the dates of occurrence. There is a limitation in terms of small population size for many elements but generally, it could be seen that the costs of repair correlate roughly with the extent of damage on the bridge element.

Table 7.22. Summary of repair total costs for bridge elements after accidents

Bridge element no.	Damage level	mean ($)	stdev ($)	min ($)	max ($)	count
12	slight	1,002.11	NA	1,002.11	1,002.11	1
12	moderate	1,312.85	498.87	776.10	1,762.33	3
107	moderate	772.75	471.57	271.10	1,379.91	6
109	negligible	1,598.48	1,127.77	690.27	2,860.81	3
109	slight	1,165.91	1,582.89	142.95	4,629.87	7
109	moderate	3,085.51	3,579.48	18.85	17,418.25	48
109	severe	2,042.07	872.98	962.68	3,366.02	6
121	moderate	129.97	20.70	109.91	161.87	5
121	moderate	186.08	NA	186.08	186.08	1
205	moderate	3,959.59	NA	3,959.59	3,959.59	1
215	moderate	717.46	822.90	135.58	1,299.34	2
234	slight	562.40	NA	562.40	562.40	1
301	moderate	365.95	NA	365.95	365.95	1
321	moderate	275.70	NA	275.70	275.70	1
330	moderate	529.25	543.18	9.94	3,344.42	38
331	slight	486.66	265.75	298.74	674.57	2
331	moderate	2,703.80	2,594.38	16.31	13,431.80	54
333	moderate	2,478.94	2,628.92	254.01	11,277.61	22
334	moderate	712.00	627.99	18.10	1,542.94	4
386	moderate	3,100.50	4,053.35	497.29	7,770.65	3
387	moderate	539.74	481.76	8.60	948.54	3
394	moderate	1,563.15	1,580.63	339.08	3,315.47	5
396	moderate	910.47	NA	910.47	910.47	1
475	moderate	920.38	1,343.66	94.25	2,926.42	4
482	moderate	704.79	NA	704.79	704.79	1
487	moderate	2,145.15	NA	2,145.15	2,145.15	1
489	slight	433.83	NA	433.83	433.83	1
489	moderate	195.61	NA	195.61	195.61	1
563	moderate	1,495.85	176.56	1,371.00	1,620.70	2
572	moderate	329.03	388.52	8.60	1,234.04	12
580	moderate	309.29	401.46	27.36	1,080.11	6
railings, guardrails & handrails	moderate	3,270.79	2,608.82	461.69	6,988.64	7
Signs and gates	moderate	2,453.64	2,922.90	163.24	6,294.58	4

Table 7.23. Summary of repair unit costs for bridge elements after accidents

Bridge element	Damage level	unit	mean ($)	stdev ($)	min ($)	max ($)	count
12	slight	SF	0.46	NA	0.46	0.46	1
12	moderate	SF	180.05	69.71	125.88	258.70	3
107	moderate	MH	28.55	2.19	25.82	31.67	6
109	negligible	MH	49.31	32.51	27.61	86.69	3
109	slight	MH	61.05	99.25	16.93	285.90	7
109	moderate	MH	37.57	46.14	17.93	345.76	48
109	severe	MH	27.31	4.15	19.85	31.95	6
121	moderate	MH	31.23	3.38	28.29	36.64	5
121	moderate	MH	18.61	NA	18.61	18.61	1
205	moderate	MH	21.70	NA	21.70	21.70	1
215	moderate	MH	26.98	4.45	23.84	30.13	2
234	slight	MH	22.27	NA	22.27	22.27	1
301	moderate	LF	91.49	NA	91.49	91.49	1
321	moderate	SF	275.70	NA	275.70	275.70	1
330	moderate	LF	22.98	38.21	1.74	222.96	37
331	slight	LF	NA	NA	NA	NA	0
331	moderate	LF	115.24	130.01	8.40	854.05	47
333	moderate	LF	53.60	38.79	4.10	132.26	21
334	moderate	LF	41.61	25.13	16.83	67.08	3
386	moderate	MH	28.54	3.07	26.70	32.08	3
387	moderate	MH	32.87	1.66	31.10	34.40	3
394	moderate	MH	34.11	33.23	9.41	92.04	5
396	moderate	MH	10.65	NA	10.65	10.65	1
475	moderate	MH	26.73	1.84	25.00	29.22	4
482	moderate	MH	35.24	NA	35.24	35.24	1
487	moderate	UN	71.51	NA	71.51	71.51	1
489	slight	UN	144.61	NA	144.61	144.61	1
489	moderate	UN	195.61	NA	195.61	195.61	1
563	moderate	MH	36.09	4.87	32.64	39.53	2
572	moderate	MH	31.74	3.84	25.37	37.88	12
580	moderate	MH	64.67	35.11	24.69	121.40	6
railings, guardrails & handrails	moderate	LF	102.12	92.59	9.62	240.99	7
Signs and gates	moderate	MH	27.16	3.34	23.81	31.70	4

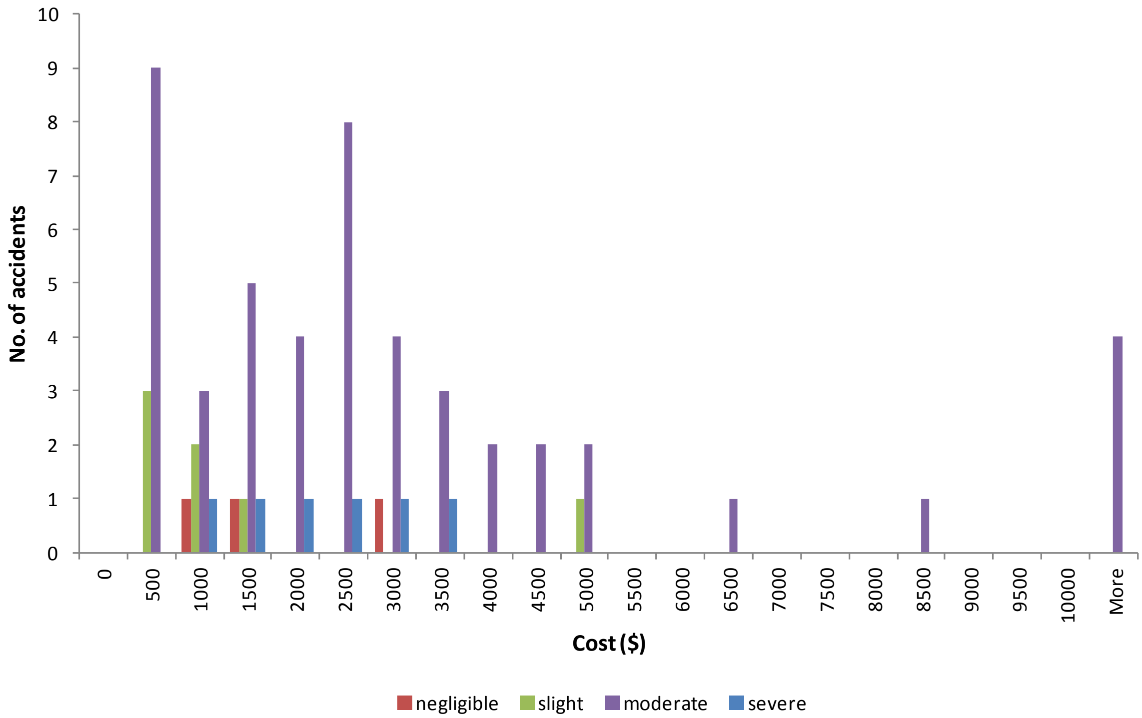


Figure 7.29. Variation in repair costs for bridge accidents at levels of damage for Element No. 109

7.3. Summary

Some reasonable data were available on Florida bridges regarding damage due to accidents from vehicle and vessel collisions. These results are summarized above in various tables and graphs. As shown in Tables 7.24 to 7.26, some bridge elements are more vulnerable than others. For instance, signs, railings, and movable bridge elements are very vulnerable.

Table 7.24. Levels of vulnerability of bridge elements to damage from truck collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	3	216	Timber Abutment	2	476	Timber Walls	2
13	Unp Conc Deck/AC Ovl	3	217	Other Mtl Abutment	2	477	Other Walls	2
28	Steel Deck/Open Grid	3	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	3	230	Unpnt Stl Cap	1	480	Mast Arm Found	1
30	Corrug/Orthotpc Deck	3	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	1
31	Timber Deck	3	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	1
32	Timber Deck/AC Ovly	3	234	R/Conc Cap	1	483	Other Mast Arm Vert	1
38	Bare Concrete Slab	3	235	Timber Cap	1	484	Paint Mast Arm Horzn	1
39	Unp Conc Slab/AC Ovl	3	240	Metal Culvert	1	485	Galvan Mast Arm Horz	1
54	Timber Slab	3	241	Concrete Culvert	0	486	Other Mast Arm Horzn	1
55	Timber Slab/AC Ovly	3	242	Timber Culvert	0	487	Sign Member Horiz	1
98	Conc Deck on PC Pane	3	243	Misc Culvert	0	488	Sign Member Vertical	1
99	PS Conc Slab	3	290	Channel	1	489	Sign Foundation	1
101	Unpnt Stl Box Girder	5	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	1
102	Paint Stl Box Girder	5	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	1
104	P/S Conc Box Girder	5	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	1
105	R/Conc Box Girder	5	301	Pourable Joint Seal	3	498	Other High Mast L.P.	1
106	Unpnt Stl Opn Girder	5	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	1
107	Paint Stl Opn Girder	5	303	Assembly Joint/Seal	3	540	Open Gearing	1
109	P/S Conc Open Girder	5	304	Open Expansion Joint	3	541	Speed Reducers	1
110	R/Conc Open Girder	5	310	Elastomeric Bearing	3	542	Shafts	1
111	Timber Open Girder	5	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	1
112	Unpnt Stl Stringer	5	312	Enclosed Bearing	3	544	Brakes	1
113	Paint Stl Stringer	5	313	Fixed Bearing	3	545	Emergency Drive	1
115	P/S Conc Stringer	5	314	Pot Bearing	3	546	Span Drive Motors	1
116	R/Conc Stringer	5	315	Disk Bearing	3	547	Hydraulic Power Unit	1
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	1
120	U/Stl Thru Truss/Bot	3	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	1
121	P/Stl Thru Truss/Bot	3	330	Metal Rail Uncoated	3	550	Hopkins Frame	1
125	U/Stl Thru Truss/Top	3	331	Conc Bridge Railing	3	560	Locks	1
126	P/Stl Thru Truss/Top	3	332	Timb Bridge Railing	3	561	Live Load Shoes	1
130	Unpnt Stl Deck Truss	3	333	Other Bridge Railing	3	562	Counterweight Suppor	1
131	Paint Stl Deck Truss	3	334	Metal Rail Coated	3	563	Acc Ladd & Plat	1
135	Timber Truss/Arch	3	356	Steel Fatigue SmFlag	0	564	Counterweight	1
140	Unpnt Stl Arch	3	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	1
141	Paint Stl Arch	3	358	Deck Cracking SmFlag	0	570	Transformers	1
143	P/S Conc Arch	3	359	Soffit Smart Flag	0	571	Submarine Cable	1
144	R/Conc Arch	3	360	Settlement SmFlag	0	572	Conduit & Junc. Box	1
145	Other Arch	3	361	Scour Smart Flag	0	573	PLCs	1
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag	0	574	Control Console	1
147	Misc Cable Coated	3	363	Section Loss SmFlag	0	580	Navigational Lights	1
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	1
152	Paint Stl Floor Beam	3	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	1
154	P/S Conc Floor Beam	3	386	Fender/Dolphin Uncoa	0	583	Swing Bridge Spec. E	1
155	R/Conc Floor Beam	5	387	P/S Fender/Dolphin	0	590	Resistance Barriers	1
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	0	591	Warning Gates	1
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	0	592	Traffic Signals	1
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	0			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	0			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	2			
204	P/S Conc Column	3	395	Timber Abut Slope Pr	2			
205	R/Conc Column	3	396	Other Abut Slope Pro	2			
206	Timber Column	3	397	Drain. Syst Metal	1			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	1			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	1			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	1			
215	R/Conc Abutment	2	475	R/Conc Walls	2			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Table 7.25. Levels of vulnerability of bridge elements to damage from vessel collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	1	476	Timber Walls	0
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	1	477	Other Walls	0
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	0
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	1	480	Mast Arm Found	0
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	0
31	Timber Deck	1	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	0
32	Timber Deck/AC Ovly	1	234	R/Conc Cap	1	483	Other Mast Arm Vert	0
38	Bare Concrete Slab	1	235	Timber Cap	1	484	Paint Mast Arm Horzn	0
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	1	485	Galvan Mast Arm Horz	0
54	Timber Slab	1	241	Concrete Culvert	1	486	Other Mast Arm Horzn	0
55	Timber Slab/AC Ovly	1	242	Timber Culvert	1	487	Sign Member Horiz	0
98	Conc Deck on PC Pane	1	243	Misc Culvert	1	488	Sign Member Vertical	0
99	PS Conc Slab	1	290	Channel	1	489	Sign Foundation	0
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	0
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	0
104	P/S Conc Box Girder	2	300	Strip Seal Exp Joint	1	497	Galvan. High Mast L.	0
105	R/Conc Box Girder	2	301	Pourable Joint Seal	1	498	Other High Mast L.P.	0
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	1	499	H. M. L. P. Found.	0
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	1	540	Open Gearing	1
109	P/S Conc Open Girder	2	304	Open Expansion Joint	1	541	Speed Reducers	1
110	R/Conc Open Girder	2	310	Elastomeric Bearing	1	542	Shafts	1
111	Timber Open Girder	2	311	Moveable Bearing	1	543	Shaft Brgs and Coupl	1
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	1	544	Brakes	1
113	Paint Stl Stringer	2	313	Fixed Bearing	1	545	Emergency Drive	1
115	P/S Conc Stringer	2	314	Pot Bearing	1	546	Span Drive Motors	1
116	R/Conc Stringer	2	315	Disk Bearing	1	547	Hydraulic Power Unit	1
117	Timber Stringer	2	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	1
120	U/Stl Thru Truss/Bot	1	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	1
121	P/Stl Thru Truss/Bot	1	330	Metal Rail Uncoated	1	550	Hopkins Frame	1
125	U/Stl Thru Truss/Top	1	331	Conc Bridge Railing	1	560	Locks	1
126	P/Stl Thru Truss/Top	1	332	Timb Bridge Railing	1	561	Live Load Shoes	1
130	Unpnt Stl Deck Truss	1	333	Other Bridge Railing	1	562	Counterweight Suppor	1
131	Paint Stl Deck Truss	1	334	Metal Rail Coated	1	563	Acc Ladd & Plat	1
135	Timber Truss/Arch	1	356	Steel Fatigue SmFlag	0	564	Counterweight	1
140	Unpnt Stl Arch	1	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	1
141	Paint Stl Arch	1	358	Deck Cracking SmFlag	0	570	Transformers	1
143	P/S Conc Arch	1	359	Soffit Smart Flag	0	571	Submarine Cable	1
144	R/Conc Arch	1	360	Settlement SmFlag	0	572	Conduit & Junc. Box	2
145	Other Arch	1	361	Scour Smart Flag	0	573	PLCs	1
146	Misc Cable Uncoated	1	362	Traf Impact SmFlag	0	574	Control Console	1
147	Misc Cable Coated	1	363	Section Loss SmFlag	0	580	Navigational Lights	2
151	Unpnt Stl Floor Beam	1	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	1
152	Paint Stl Floor Beam	1	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	1
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	5	583	Swing Bridge Spec. E	1
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	5	590	Resistance Barriers	1
156	Timber Floor Beam	1	388	R/Conc Fender/Dolphi	5	591	Warning Gates	1
160	Unpnt Stl Pin/Hanger	1	389	Timber Fender/Dolphi	5	592	Traffic Signals	1
161	Paint Stl Pin/Hanger	1	390	Other Fender/Dolphin	5			
201	Unpnt Stl Column	1	393	Blkhd Sewl Metal Unc	1			
202	Paint Stl Column	1	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	1			
205	R/Conc Column	1	396	Other Abut Slope Pro	1			
206	Timber Column	1	397	Drain. Syst Metal	0			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	0			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	0			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	0			
215	R/Conc Abutment	1	475	R/Conc Walls	0			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Table 7.26. Levels of vulnerability of bridge elements to damage from overhead collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	3	216	Timber Abutment	1	476	Timber Walls	1
13	Unp Conc Deck/AC Ovl	3	217	Other Mtl Abutment	1	477	Other Walls	1
28	Steel Deck/Open Grid	3	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	1
29	Steel Deck/Conc Grid	3	230	Unpnt Stl Cap	1	480	Mast Arm Found	0
30	Corrug/Orthotpc Deck	3	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	0
31	Timber Deck	3	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	0
32	Timber Deck/AC Ovly	3	234	R/Conc Cap	1	483	Other Mast Arm Vert	0
38	Bare Concrete Slab	3	235	Timber Cap	1	484	Paint Mast Arm Horzn	0
39	Unp Conc Slab/AC Ovl	3	240	Metal Culvert	1	485	Galvan Mast Arm Horz	0
54	Timber Slab	3	241	Concrete Culvert	0	486	Other Mast Arm Horzn	0
55	Timber Slab/AC Ovly	3	242	Timber Culvert	0	487	Sign Member Horiz	0
98	Conc Deck on PC Pane	3	243	Misc Culvert	0	488	Sign Member Vertical	0
99	PS Conc Slab	3	290	Channel	1	489	Sign Foundation	0
101	Unpnt Stl Box Girder	5	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	0
102	Paint Stl Box Girder	5	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	0
104	P/S Conc Box Girder	5	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	0
105	R/Conc Box Girder	5	301	Pourable Joint Seal	3	498	Other High Mast L.P.	0
106	Unpnt Stl Opn Girder	5	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	0
107	Paint Stl Opn Girder	5	303	Assembly Joint/Seal	3	540	Open Gearing	0
109	P/S Conc Open Girder	5	304	Open Expansion Joint	3	541	Speed Reducers	0
110	R/Conc Open Girder	5	310	Elastomeric Bearing	3	542	Shafts	0
111	Timber Open Girder	5	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	0
112	Unpnt Stl Stringer	5	312	Enclosed Bearing	3	544	Brakes	0
113	Paint Stl Stringer	5	313	Fixed Bearing	3	545	Emergency Drive	0
115	P/S Conc Stringer	5	314	Pot Bearing	3	546	Span Drive Motors	0
116	R/Conc Stringer	5	315	Disk Bearing	3	547	Hydraulic Power Unit	0
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	0
120	U/Stl Thru Truss/Bot	3	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	0
121	P/Stl Thru Truss/Bot	3	330	Metal Rail Uncoated	3	550	Hopkins Frame	0
125	U/Stl Thru Truss/Top	3	331	Conc Bridge Railing	3	560	Locks	0
126	P/Stl Thru Truss/Top	3	332	Timb Bridge Railing	3	561	Live Load Shoes	0
130	Unpnt Stl Deck Truss	3	333	Other Bridge Railing	3	562	Counterweight Suppor	0
131	Paint Stl Deck Truss	3	334	Metal Rail Coated	3	563	Acc Ladd & Plat	0
135	Timber Truss/Arch	3	356	Steel Fatigue SmFlag	0	564	Counterweight	0
140	Unpnt Stl Arch	3	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	0
141	Paint Stl Arch	3	358	Deck Cracking SmFlag	0	570	Transformers	0
143	P/S Conc Arch	3	359	Soffit Smart Flag	0	571	Submarine Cable	0
144	R/Conc Arch	3	360	Settlement SmFlag	0	572	Conduit & Junc. Box	0
145	Other Arch	3	361	Scour Smart Flag	0	573	PLCs	0
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag	0	574	Control Console	0
147	Misc Cable Coated	3	363	Section Loss SmFlag	0	580	Navigational Lights	0
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	0
152	Paint Stl Floor Beam	3	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	0
154	P/S Conc Floor Beam	3	386	Fender/Dolphin Uncoa	0	583	Swing Bridge Spec. E	0
155	R/Conc Floor Beam	5	387	P/S Fender/Dolphin	0	590	Resistance Barriers	0
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	0	591	Warning Gates	0
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	0	592	Traffic Signals	0
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	0			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	0			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	3	395	Timber Abut Slope Pr	1			
205	R/Conc Column	3	396	Other Abut Slope Pro	1			
206	Timber Column	3	397	Drain. Syst Metal	1			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	1			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	1			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	1			
215	R/Conc Abutment	1	475	R/Conc Walls	1			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

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8. Advanced Deterioration

A common and general concern of risk management is the unavoidable disruption of service due to the need to respond pro-actively to impending hazards. If bridge maintenance is deferred for a prolonged period, the condition of the structure reaches a point where the agency is forced to take action to ensure safe mobility. The action may be posting, closure, strengthening, or partial or complete replacement. All of these actions disrupt service, forcing road users to expend more time and fuel in congestion or detours. They also force the agency to expend public funds on the action.

One of the key life cycle tradeoffs in bridge management is the possibility of strategic preventive maintenance actions to postpone the need for more expensive forced activities. A purpose of Pontis and the PLAT is to identify these opportunities. Accurate evaluation of preventive activities requires the use of tools to quantify the negative impacts of allowing conditions to deteriorate. There is no laboratory where a scientific sample of bridges can be allowed to deteriorate to failure under realistic conditions of weather and traffic. Similarly, highly deteriorated conditions in the Florida inventory are uncommon and are routinely avoided by active management processes.

Florida's Pontis database is the only comprehensive source of data on historical conditions and events of service disruption. Therefore a resourceful data mining of Pontis is necessary to develop the required models.

8.1. Identifying service disruptions related to condition

While there are many ways in which bridge characteristics can interfere with mobility, the Pontis database offers three types of service disruption that can be specifically related to condition:

- Evidence of bridge closure or replacement, provided by the assignment of a bridge to fictitious "District 9 – Central Office." FDOT never deletes a structure from the Pontis database, but instead assigns it to District 9 when it is no longer required to be in the active inventory, usually because it has been demolished.
- Evidence of bridge reconstruction, from National Bridge Inventory (NBI) item 106 – Year reconstructed. This is defined as the most recent year in which the bridge underwent work eligible for Federal funding (regardless of how actually funded). This can include total replacement, superstructure replacement, and functional improvements such as widening.
- Operational status of posted or closed, based on a value of NBI item 41 of B, D, E, K, P, or R.

This information is comprehensive for bridges, but not for other types of structures, such as sign structures, high-mast light poles, and mast arms. This is because non-bridge structures were added to Pontis relatively recently and do not yet have a long time series of data. As a result, it was necessary to remove non-bridge structures from the analysis. This reduced the number of structures in the data set from 31,166 to 15,548.

Table 8.1 summarizes the events of service disruption, potentially related to condition, identified in the analysis. For an inspection to be listed in the table, it must have at least one deck, superstructure, or substructure element in the worst or second-worst condition state. This represents 27% of the 97,416 bridge inspections in the database. A bridge inspection is classified as Retired if it is the last element inspection that was recorded before the bridge was moved to District 9. It is classified as Rebuilt if it is the last element inspection recorded before the year indicated in NBI item 106. It is classified as Posted if the operational status indicates posting or closure in the following inspection but not in the current inspection. These definitions arose from the statistical analysis described later in this report.

Table 8.1. Summary of service disruptions

Material	Inspections	Number of events			Percent of inspections		
		Retired	Rebuilt	Posted	Retired	Rebuilt	Posted
Reinforced concrete	9057	175	110	54	1.9	1.2	0.6
Prestressed concrete	9865	193	208	51	2.0	2.1	0.5
Steel	4810	143	86	47	3.0	1.8	1.0
Timber	2802	150	58	8	5.4	2.1	0.3

It can be seen in Table 8.1 that the types of service disruptions can vary substantially by material type, as determined from NBI item 43.

8.2. Identifying deteriorated conditions related to risk

In the Florida Bridge Inspection Manual, as in the AASHTO CoRe Element Manual (AASHTO 2001), the definition of the final (worst) condition state of each structural element includes words such as, "Deterioration is sufficient to warrant structural review to ascertain the impact on the ultimate strength and/or serviceability of either the element or the bridge." This wording indicates the possibility of service disruption in the worst condition state, at least for the primary load-bearing elements.

An analysis of replacement, reconstruction, posting, and closure of bridges in the Florida inventory found that even a tiny fraction of an element in the worst condition state was highly correlated with service disruption, and that a significant fraction of bridges underwent a service disruption even with no elements in their worst condition states. A detailed analysis of central office bridges (next chapter) found that it was also necessary to consider the fraction of primary elements in the second-to-worst condition state in order to obtain a more complete picture of the motivations for posting, reconstruction, and replacement.

In the analyses described in the remainder of this report, each model was tested for significance of the second-to-worst condition state, to see if it was necessary to consider these moderately-deteriorated elements. In almost every case, these elements played a necessary role in obtaining strong statistical results. This may indicate a significantly proactive FDOT response to advanced deterioration.

8.3. Central office bridges

As of August 2011, the FDOT Pontis database contains 5,033 structures in District 9, of which 3,213 are bridges (Table 8.2). Known as "central office bridges," these are structures whose data are not actively maintained for inspection, operations, or planning purposes. Nevertheless, they are retained in Pontis for a variety of other reasons, including possible use in special studies such as the current one. Most of these are structures that were previously in service, but were retired or replaced. The reasons for these retirements or replacements are especially relevant to efforts to quantify the likelihood and consequences of risk factors such as advanced deterioration, fatigue, storms, collisions, and scour. As part of the study, an effort was made to identify the reasons for bridges to be classified as "central office bridges."

Sign structures, high-mast light poles, and traffic signal mast arms are relatively recent additions to the Pontis database, unlikely to have enough data yet to reliably quantify cause-and-effect relationships. As a result, it was decided to omit them from the analysis. To ensure that the data set would have roadway-on data and element inspection data, structures lacking these records were omitted.

In many cases FDOT staff add a new bridge in the central office to reserve a bridge ID for a structure that is not yet built, or not yet open to traffic. Appendix A of the FDOT Coding Guide (FDOT 2011) describes the data items required for these structures. Nearly all of these structures lack element inspection data.

Table 8.2. Structure design types in District 9

Design type		Count
-1	Unknown (P)	75
0	Other (NBI)	298
1	Slab	718
2	Stringer/Girder	1494
3	Girder-Floorbeam	55
4	Tee Beam	202
6	Single/Spread Box	23
7	Frame	1
9	Truss-Deck	1
10	Truss-Thru	18
11	Arch-Deck	24
12	Arch-Thru	3
15	Movable - Lift	4
16	Movable-Bascule	46
17	Movable-Swing	4
18	Tunnel	1
19	Culvert	265
20	Mixed types	2
21	Segmental Box Girder	3
22	Channel Beam	51
89	Sign-Monotube-Cantilever	1
91	Sign-Cantilever	910
92	Sign-Span	361
93	Sign-Butterfly	44
94	Sign-Cable	152
96	High Mast Light	270
97	Traffic Signal Mast Arm	7
	Total	5033

In addition, occasionally a bridge will be assigned to the central office in Pontis if it is found to be open only to private use, or does not satisfy NBI length requirements, or for other administrative reasons. This may occur without any physical change happening at the bridge site. A total of 70 bridges were found to be in District 9 for this reason. After removing these structures, a total of 1,480 bridges remained in the data set for further analysis.

8.4. Classifying the reason for demolition or replacement

It is desired to classify these remaining bridges according to the likely reason(s) that each bridge was removed or replaced. The most common reasons include:

- A roadway section or intersection is widened or reconfigured to increase access, capacity or safety.
- The condition of a bridge has deteriorated to the point where there is an actual or imminent effect on serviceability for legal loads, or where the danger of fracture or other types of sudden failure is unacceptably high.
- A bridge is damaged by the cumulative effect of scour on its foundations, to the point where there is actual or imminent effect on serviceability.
- A bridge presents a safety or access issue because of narrowness, impaired vertical clearance, or low load capacity not caused by active deterioration.
- A bridge is no longer needed because the feature under it is removed, or the bridge's traffic-carrying role is no longer required.
- A bridge is damaged or destroyed by a hurricane, flood, vehicle or vessel collision, or other extreme event.

It is possible for there to be a combination of reasons, particularly a combination of deterioration and functional needs. Some of the reasons are relevant to risk analysis, while others are not.

The Pontis database provides several types of evidence that can assist in this classification:

- Year built or reconstructed (NBI items 27 or 106);
- National Bridge Inventory (NBI) condition assessments for deck, superstructure, substructure, and culvert (Items 58, 59, 60, and 62);
- NBI assessments of channel condition, waterway adequacy, and scour criticality (Items 61, 71, and 113);
- NBI indication of the requirement for fracture critical inspections (Item 92A);
- Element inspection data, particularly the percentage of elements in their worst and second-worst defined condition states;
- Operating rating (NBI item 64);
- Route number and milepost (NBI items 5 and 11), which is especially useful for identifying groups of bridges that are all replaced at the same time along a route for functional reasons;
- Number of lanes on the bridge (NBI item 28A);
- Comments that are logged with the bridge record or with inspection records.

Ultimately each bridge was classified using judgment, based on the preponderance of the available data. The most important factors turned out to be:

- Comments entered in the database directly explaining what happened to the bridge and why;
- For bridges with an identified replacement structure, a change in the number of lanes was taken as a strong indicator of functional reasons for the replacement;
- An NBI condition assessment of 4 or below, or any element (from among the deck, superstructure, or substructure primary load-bearing elements) in the worst condition state, were taken as strong indicators of condition reasons for bridge replacement, especially when in combination with reduced operating ratings;
- Indicators of severe channel deterioration, waterway inadequacy, scour criticality, or fracture criticality were taken as indicators that extreme events, or the possibility of such hazards, may have contributed to the decision to replace a bridge. In addition, some of the bridges had specific mentions of hurricane damage in their final inspection records.

It should be noted that none of these information sources is necessarily authoritative. In many cases there are engineering reports and detailed inspection reports outside of Pontis that provide a deeper justification for replacement. The current analysis seeks network-wide risk estimates, and therefore places higher priority on breadth and comprehensiveness across the inventory, rather than depth of analysis for any specific bridges.

The interpretation of the data is in the context of knowing with certainty that the bridge was replaced or retired: the effect is known and the cause is imputed. For example, if a two-lane bridge in bad condition is replaced by another two-lane bridge, there is a presumption that the bad condition must have contributed to the replacement decision. The converse is usually not true: if a bridge is in bad condition, it usually is not immediately replaced. Instead, it might be repaired or more closely monitored. The risk factor of interest in this study is the small but non-zero likelihood that none of the less extreme options will suffice, and that immediate replacement or load restriction will be necessary, thus interrupting normal service on the bridge.

Table 8.3 shows the results of this classification analysis. In this table, each retired or replaced bridge is classified by what is believed to be the primary reason for retirement, using the logic described above. This information was added to the *userbrdg* table in the researchers' Pontis database for further use.

Table 8.3. Primary cause of bridge retirement

Reason		Count
?	Unknown, unable to determine reason	440
C	Cracking, fatigue, fracture criticality	8
D	Deterioration of structural elements	327
F	Functional, roadway project, add lanes	650
H	Hurricane damage	18
V	Vehicle/vessel collision	2
W	Washout, scour, flood	35
	Total	1480

It can be seen that in 440 cases there was not enough information in Pontis to enable useful speculation about the cause of bridge retirement. However, in all of these cases there were no condition assessments of 4 or below, no primary elements in their worst condition states, and no indication of vulnerability to extreme events. In many cases these structures have one or more primary elements in their second-worst condition state, indicating that condition may have played a role. Functional requirements may also have played a role.

In many cases replacement of a bridge may be motivated by a combination of reasons. For example, in 54 of the cases where deterioration was viewed as the primary reason for replacement, the replacement bridge had a different number of lanes than the original structure.

One of the most useful indicators of the reason for bridge retirement, was the set of data available about the bridge that replaced it. The Pontis data item *userbrdg.repl_strct_id* was added in February 2008 to enable this type of analysis. Prior to that date, the replacement structure had been sporadically noted in bridge or inspection comments. To improve the usefulness of this information, the replacement bridge ID was extracted from all comments where it was noted, and used to further populate the *repl_strct_id* column in the database. This added 642 data points, for a total of 913 bridges whose replacement bridge id was known, out of the 1480 bridges in the analysis. Of the remaining bridges, some were replaced by an unknown bridge ID, and others were not replaced at all.

An SQL update script for the 642 replacement structure ids can be provided for FDOT use if desired.

8.5. Likelihood of service disruption

To develop a predictive model of the likelihood of service disruption, a regression analysis was conducted to estimate the probability of replacement, reconstruction, and operational restriction, as a function of element condition. The analysis attempted to find and quantify a cause-and-effect relationship between advanced deterioration and service disruption.

In order to make the analysis possible using a Pontis data set, the incidence of service disruption was approximated by recorded instances of bridge retirement, replacement, reconstruction, and posting. It was recognized that condition is not the only reason for these activities. It was expected therefore that a significant amount of statistical "noise" would be present in the models, and it was important to select model formulations that would isolate the effect of condition and avoid bias caused by non-condition-related influences on agency decisions.

8.5.1. Effect of condition

It is desired that the risk model to be developed have the ability to work with the results of deterioration models, to show how the risk of service disruption may increase as condition worsens. A previous FDOT study of deterioration showed that reliable deterioration models can be developed for element condition states, but that the forecasting of National Bridge Inventory (NBI) condition ratings was indirect and much less precise (Sobanjo and Thompson 2011). Therefore a statistical relationship with element condition states is desired.

When bridge element deterioration is forecast using Markovian models, a small but non-zero fraction appears in the worst condition state relatively early in the element's life. Table 8.4 shows a typical example of deterioration based on a Markov transition probability matrix. It can be seen that the percent in the worst condition state (state 5 in this example) is non-zero after only three years of deterioration. Obviously the risk of service disruption cannot be considered to be elevated when so little deterioration has taken place. From this reasoning, it follows that small amounts of deterioration should be associated with very small amounts of risk, and that risk should smoothly increase with further deterioration.

Table 8.5 shows the frequency of retirement, reconstruction, and posting for deteriorated bridges for various ranges of condition. In this table the inspections counted as "Decayed" are those that were moved to District 9 for deterioration or for unknown reasons (codes "D" or "?"). The rows in this table represent ranges of the percent found to be in the worst (left side) or second-worst (right side) condition state for the element in worst condition. The left side of the table shows that the probability of a bridge being Rebuilt is already 3.13 percent even after a tiny percentage (more than zero but less than one-tenth of one percent) is found in the worst condition state. As expected, the risk increases as higher amounts are found. The right side of the table shows that the risk is lower but still significant for the second-worst condition state. Experimental regression models also confirmed that it is necessary to consider the second-worst condition state in order to have a risk model with reasonable behavior.

Table 8.5 also hints that the increase in risk is not linear with worsening condition, but is more closely tied to the log of condition. The regression analysis presented later in this chapter shows that there is a good reason for this to be the case.

Table 8.4. Example of Markovian deterioration

Markov transition probability matrix					
State	State probability in one year				
Today	1	2	3	4	5
1	95.3	4.6	0.1	0.0	0.0
2	0	93.2	3.9	1.9	1.0
3	0	0	89.4	7.3	3.3
4	0	0	0	82.8	17.2
5	0	0	0	0	100

Probability of state k next year: $y_k = \sum_j x_j p_{jk}$ for all k

j is the condition state this year and x is the fraction in state j
p is the transition probability from j to k

Future condition forecasts

Year	1	2	3	4	5	Year	1	2	3	4	5	
	Percent by condition state						Percent by condition state					
0	100	0.0	0.0	0.0	0.0	25	30.0	28.1	10.1	6.8	25.0	
1	95.3	4.6	0.1	0.0	0.0	26	28.6	27.6	10.1	6.9	26.8	
2	90.8	8.7	0.4	0.1	0.0	27	27.3	27.0	10.2	7.0	28.6	
3	86.6	12.3	0.8	0.3	0.2	28	26.0	26.4	10.2	7.1	30.4	
4	82.5	15.4	1.2	0.5	0.4	29	24.8	25.8	10.1	7.1	32.2	
5	78.6	18.2	1.8	0.8	0.6	30	23.6	25.2	10.1	7.1	34.0	
6	74.9	20.5	2.4	1.1	1.0	31	22.5	24.6	10.0	7.1	35.8	
7	71.4	22.6	3.0	1.5	1.5	32	21.4	23.9	9.9	7.1	37.6	
8	68.0	24.3	3.6	1.9	2.1	33	20.4	23.3	9.8	7.0	39.4	
9	64.8	25.8	4.3	2.3	2.8	34	19.5	22.6	9.7	7.0	41.2	
10	61.8	27.0	4.9	2.7	3.6	35	18.5	22.0	9.6	6.9	42.9	
11	58.9	28.0	5.5	3.1	4.5	36	17.7	21.4	9.5	6.9	44.7	
12	56.1	28.8	6.1	3.5	5.5	37	16.8	20.7	9.3	6.8	46.4	
13	53.5	29.5	6.6	3.9	6.6	38	16.1	20.1	9.1	6.7	48.0	
14	51.0	29.9	7.1	4.3	7.7	39	15.3	19.5	9.0	6.6	49.7	
15	48.6	30.2	7.6	4.6	9.0	40	14.6	18.8	8.8	6.5	51.3	<< Median life expectancy
16	46.3	30.4	8.0	5.0	10.4	41	13.9	18.2	8.6	6.4	52.9	
17	44.1	30.5	8.4	5.3	11.8	42	13.2	17.6	8.4	6.2	54.5	
18	42.0	30.4	8.7	5.5	13.3	43	12.6	17.0	8.2	6.1	56.0	
19	40.1	30.3	9.0	5.8	14.8	44	12.0	16.5	8.0	6.0	57.5	
20	38.2	30.1	9.3	6.0	16.4	45	11.5	15.9	7.8	5.9	58.9	
21	36.4	29.8	9.5	6.3	18.1	46	10.9	15.3	7.6	5.7	60.4	
22	34.7	29.4	9.7	6.4	19.7	47	10.4	14.8	7.4	5.6	61.8	
23	33.0	29.0	9.9	6.6	21.5	48	9.9	14.3	7.2	5.5	63.1	
24	31.5	28.6	10.0	6.7	23.2	49	9.5	13.8	7.0	5.3	64.4	
25	30.0	28.1	10.1	6.8	25.0	50	9.0	13.3	6.8	5.2	65.7	

Table 8.5. Frequency of service disruption for ranges of condition (in percent)

Range	Worst element in its worst condition state					Worst element in 2nd-worst condition state				
	Count	Decayed	Rebuilt	Posted	Total	Count	Decayed	Rebuilt	Posted	Total
0.0<0.1	32	0.00	3.13	0.00	3.13	228	0.44	3.95	0.00	4.39
0.1-0.5	200	0.50	3.00	0.50	4.00	948	0.11	1.48	0.11	1.69
0.5<1.0	212	1.42	1.42	0.94	3.77	1255	0.80	1.51	0.40	2.71
1<5	1071	1.96	2.61	0.09	4.67	5232	0.88	1.85	0.59	3.33
5<10	738	2.98	1.49	1.22	5.69	2556	1.13	1.68	0.74	3.56
10<50	1182	5.41	2.37	0.93	8.71	5061	1.64	1.70	0.81	4.15
50<100	1227	5.38	1.55	1.06	7.99	8454	1.77	1.87	0.53	4.18

8.5.2. Potential explanatory variables

Table 8.6 shows the variation in service disruptions by functional class of the roadway on each bridge. Bridges carrying more important functional classes are more likely to be replaced or reconstructed when they reach poor condition, and less likely to be posted.

Table 8.6. Frequency of service disruption by functional class

Functional class	Count	Decayed	Rebuilt	Posted	Total
01 Rural Interstate	573	0.00	2.27	0.00	2.27
02 Rural Other Princ	1706	2.17	1.47	0.12	3.75
06 Rural Minor Arterial	1103	2.63	1.09	0.45	4.17
07 Rural Mjr Collector	2058	1.26	0.39	0.63	2.28
08 Rural min Collector	1446	0.69	0.62	0.69	2.01
09 Rural Local	4359	2.84	1.33	0.76	4.93
11 Urban Interstate	1567	0.32	6.89	0.38	7.59
12 Urban Fwy/Expwy	1088	0.55	3.95	0.00	4.50
14 Urban Other Princ	2449	1.10	2.12	0.29	3.51
16 Urban Minor Arterial	2574	1.13	1.17	0.93	3.22
17 Urban Collector	2530	0.83	1.78	0.87	3.48
19 Urban Local	3250	1.02	1.11	0.86	2.98

Table 8.7 shows how the frequency of service disruptions does not exhibit a clear overall pattern when bridges are grouped by traffic volume. The lowest-volume bridges are more likely to be retired, while the highest volume bridges are more likely to be reconstructed. The incidence of posting increases as volume decreases, but this is because of condition rather than because of traffic volume.

Table 8.7. Frequency of service disruption by ADT range

Range of ADT	Count	%Bad*	Decayed	Rebuilt	Posted	Total
0<100	2678	49.94	3.40	1.27	0.64	5.30
100<500	3043	48.64	1.64	1.18	1.02	3.84
500<1000	1741	47.18	0.69	0.80	0.86	2.35
1000<5000	5441	46.54	1.31	0.99	0.74	3.03
5000<10000	2836	41.11	1.34	1.23	0.81	3.39
10000<50000	7389	34.40	1.04	2.33	0.30	3.67
50000+	1578	21.58	0.51	5.96	0.13	6.59

*Percent of the worst element in the worst or 2nd-worst condition state

Analysis of disruption frequency by bridge design type (NBI item 43) did not show any clear relationships, but the analysis by material type was very clear, as shown in Table 8.8. Most of the timber bridges that are in deteriorated condition are already posted for reasons unrelated to condition, so the incidence of posting is even higher than the table would suggest.

Table 8.8. Frequency of service disruption by material type (based on NBI item 43A)

Material	Count	Decayed	Rebuilt	Posted	Total
Concrete - Prestressed	9108	0.82	2.18	0.53	3.54
Concrete - Reinforced	8397	1.05	1.25	0.60	2.90
Steel	4551	1.80	1.80	0.99	4.59
Timber	2650	3.85	2.00	0.26	6.11

8.5.3. Model form

Regression models were developed for a variety of functional forms that might be justified by the problem structure. The model types and their rationale are:

Constant probability. This model assumes that a bridge has a fixed probability of service disruption if any of its elements reaches its worst or second-worst condition state. Such a model works fairly well when only bridge decks are considered, because they are inspected as "eaches". In the historical database of inspections, a bridge deck has either 100% or 0% in the target condition state. Unfortunately, this model lacked explanatory power for elements that are not inspected as "eaches". Also, the forecasts output by a Markovian model are not limited to 100% or 0%. In addition, it was considered unreasonable that a tiny fraction in a deteriorated condition state should yield the same disruption probability as a much larger fraction in that state.

Linear probability. This model assumes that a bridge's probability of service disruption is linearly proportional to the fraction and/or probability of the deteriorated condition state. This model form is implicit in Pontis, where the proportional relationship is represented by the "failure probability," and is also used in the PLAT. The regression analysis showed that the linear model was a reasonable approximation for some of the materials and some of the disruption types, but could readily be improved upon by other model forms. It is the only model form currently accepted by Pontis, however.

Weibull model. This model assumes that the probability of disruption is related to bridge age, using a Weibull distribution. A variation of the model uses a measure of condition, such as health index, as a proxy for age. This is the same model that proved to be very effective in describing the onset of deterioration, in the previous FDOT study. There would be an attractive consistency and symmetry if the end of a bridge's life could be described using the same model as the beginning of its life. Unfortunately, the Weibull form did not work well for the risk model. Determining the actual effective age of a bridge is inexact, introducing a considerable amount of error because of unknown past rehabilitation work. Health index was a poor proxy that did not produce compelling goodness-of-fit statistics from the regression model.

Lognormal model. Like the Weibull model, this model also assumes that the probability of disruption is related to age, and uses condition as a proxy for age. However, it models condition as the result of a multiplicative process (multiplying a transition probability by itself some unknown number of times), such that the predictive variable can be expressed as the log of the fraction in the deteriorated condition state. This model has the theoretical attractiveness that it fits the phenomenon being modeled in a very intuitive way. Moreover, it was the only model form that consistently produced strong goodness-of-fit measures in the regression analysis.

The lognormal model is very commonly used in reliability analysis in combination with Markov models, and is common in many fields where compound rates of change are used. For example, the Black-Scholes model for pricing of financial instruments is based on the lognormal distribution.

Using these alternative model forms in the regression analysis, models were attempted using either the worst condition state, or a combination of the worst and second-worst states, or using a weighted sum of the worst and second-worst states. Models were developed separately for each type of disruption (retirement, reconstruction, or posting), and for combinations of the disruption types. Explanatory variables were investigated in various reasonable configurations, including constant “dummy” variables for material and functional class, and variations on ADT suggested by the exploratory data analysis. Model stratification was also investigated in place of the use of dummy variables.

In the development of this type of statistical model, censoring is a significant issue. In a population of several thousand bridges, only a relatively small fraction are close to the end of their lives. The remaining bridges will eventually reach this point at an unknown time in the future. Future life extension activity may postpone this eventuality for a considerable length of time. It is significant to know that certain bridges have reached various stages of advanced deterioration without encountering any service disruption, so this valuable information should not be discarded.

To ensure that the model estimation process is unbiased while still making maximum use of censored data, the analysis was organized in the form of a hazard model. A hazard model estimates the instantaneous probability, at a given condition level, that a service disruption will occur (Figure 8.1).

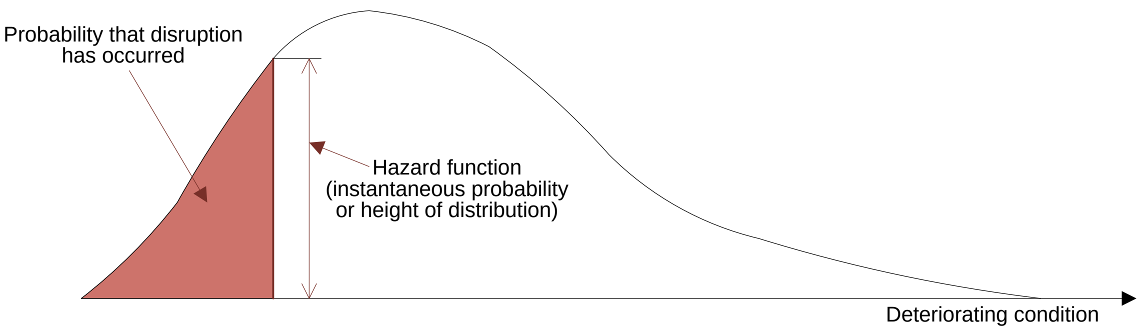

Figure 8.1. Schematic depiction of a hazard model

When the analysis is organized as a hazard model, the unit of analysis is the inspection, rather than the bridge. Rather than following each bridge through its life to find out when replacement, reconstruction, or posting occurs, the process looks at each inspection to see what happens immediately afterward, before the next inspection. The possibilities are that the bridge may be retired, reconstructed, or posted; or it is possible (and most likely) that no disruption will happen at all.

8.5.4. Data filtering

In the estimation data set, inspections were included only if they contain at least one element in the worst or second-worst condition state, from among deck, superstructure, and substructure elements (those with elemkey<300). It is assumed that the disruption probability is zero if no primary load-bearing elements are in deteriorated states. In addition:

- Inspections having the inspkey value of ‘STRT’ were omitted since they represent FDOT’s initial experiments with element inspection in the mid-1990s, which have questionable accuracy.
- Inspections after 7/1/2009 were omitted, to provide an opportunity to examine the following inspection for posting data. This ensures that the hazard model is not censored.
- Certain additional quality control checks were performed, such as ensuring that element quantities are non-zero.

The final data set used in model estimation had 26,534 bridge inspections.

8.5.5. Estimation and model evaluation

The Weibull and lognormal models were estimated using an iterative maximum likelihood estimation procedure, similar to what was used in the previous study for the onset of deterioration. Excel's Solver module provided the fitting procedure. It was configured to maximize the sum of a log-likelihood function for the normal distribution, a measure of the probability of the observed data arising, given the model under investigation.

To facilitate visualization and evaluation, a procedure was developed to group the data points into bins. A tableau using 20 bins of equal population provided the most consistent results. Figure 8.2 shows an example, for steel bridges.

Model parameters	
SpW	40
SbW	40
DkX	0.5
SpX	0.5
SbX	0.5
c	-0.00014
b	0.001962
a	3000
μ	15.5453
σ	3.997625
Std Dev	0.207604
Sum Log LF	737.1525
C1	-5.6E-08
C2	0.213828
Population	4812
Disruptions	218
Avg rate	4.53
X2-model	28.49
X2-uniform	73.07

Bin	Count	DecayIx	ActCnt	EstCnt	Actual	Estimate	X2-mod	X2-unifo
C1	1	1E-06	0	0.0	0.000	0.000	0.00	10.91
0	240	0.01669	3	5.8	1.250	2.400	1.32	5.74
1	241	0.05823	8	7.8	3.320	3.248	0.00	0.78
2	240	0.10813	10	8.5	4.167	3.548	0.26	0.08
3	241	0.17639	12	9.0	4.979	3.735	1.00	0.11
4	240	0.27524	15	9.3	6.250	3.869	3.52	1.53
5	241	0.43885	9	9.6	3.734	3.971	0.03	0.33
6	241	0.65789	6	9.7	2.490	4.035	1.43	2.21
7	240	0.91264	8	9.8	3.333	4.071	0.32	0.78
8	241	1.24123	7	9.9	2.905	4.098	0.84	1.40
9	240	1.76118	8	9.9	3.333	4.132	0.37	0.78
10	241	2.48915	12	10.1	4.979	4.179	0.37	0.11
11	241	3.5524	6	10.3	2.490	4.265	1.78	2.21
12	240	4.62675	7	10.5	2.917	4.369	1.16	1.40
13	241	6.07297	14	10.9	5.809	4.531	0.87	0.87
14	240	7.99632	13	11.5	5.417	4.774	0.21	0.40
15	241	10.1107	6	12.2	2.490	5.067	3.16	2.21
16	241	12.0712	18	12.9	7.469	5.356	2.01	4.61
17	240	15.1367	15	14.0	6.250	5.832	0.07	1.53
18	241	19.4858	27	15.8	11.203	6.540	8.01	23.73
19	239	29.8487	14	19.9	5.858	8.337	1.76	0.87
C2	1	100	0.21383	0.2	21.383	21.383	0.00	10.49

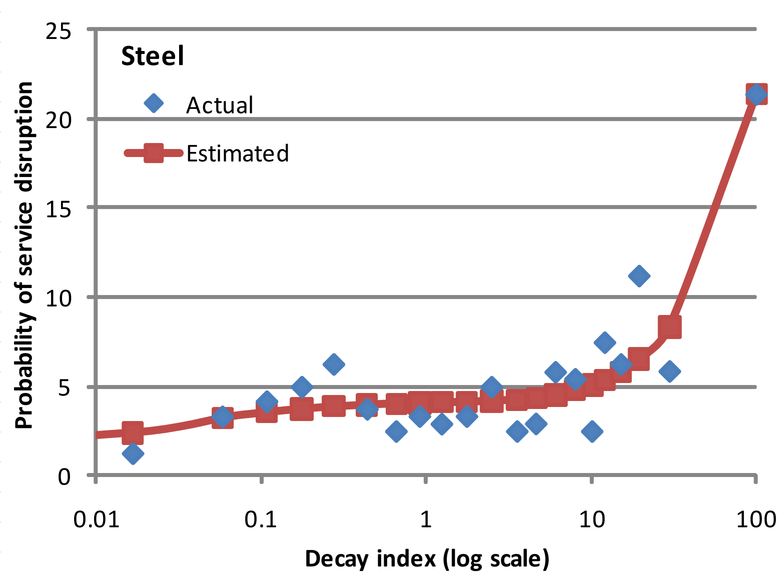

Figure 8.2. Example of model estimation tableau, for steel bridges

In addition to the 20 bins, the model had two control points. One control point was set at a very small but non-zero level of deterioration, and constrained to produce a hazard probability of zero at that point. This was to ensure a smooth risk transition as a bridge deteriorates under a Markov model. A second control point was set at a condition level where 100% of deck, superstructure, and substructure elements were in their worst condition state. This control point was constrained to ensure that the hazard function never exceeded 100% probability of service disruption.

The choice of 20 as the number of bins was made to ensure that nearly all bins have at least 5 instances of service disruption. This was necessary in order to ensure the usefulness of a chi-squared test for goodness of fit. The Pearson's chi-square statistic was calculated from equation 8-1.

$$X^2 = \sum_{i-1}^{N} \frac{(O_i - E_i)^2}{E_i} \tag{8-1}$$

Where O_i = the observed number of disruptions in the i^{th} bin
E_i = the predicted number of disruptions according to the model
N = the total number of bins (20)

While this statistic has limited usefulness as an absolute measure of goodness-of-fit, it is very useful in combination with graphic visualization for comparing two or more alternative models. It aided in the search for the most representative model specification.

8.6. Model results

Maximum likelihood estimation using Excel's Solver is a very general iterative search method that makes few assumptions about the model being developed. This is valuable when there is a desire to investigate several very different model forms. The same estimation framework was adapted to all of the combinations of model formulation alternatives discussed above. The Chi-squared test enabled a fair comparison among all of them. For consistency, the constant (average) probability model was computed for every formulation and used as a baseline against which each new model was compared. For a model to be a useful improvement over simpler models, it would need to produce a significantly lower chi-squared value.

The model estimation process determined that the best statistical properties were achieved if reinforced concrete, prestressed concrete, steel, and timber models were developed entirely separately (as classified by NBI item 43A), using the same functional form but with different model coefficients. Therefore these results are reported separately in the following sections. This is reflective of the fact that these four material categories have very different repair and rehabilitation possibilities when bridges reach a highly deteriorated state.

8.6.1. Decay index

One of the key questions to be addressed in the modeling process was the means of describing bridge condition at the bridge level, as built up from the element level. The maximum likelihood estimation process was adapted to investigate the various alternatives to see which formulation and model coefficients would consistently yield the best predictions of service disruption frequency. The best results came from a two-stage process modeled on the concept of the bridge health index (Shepard and Johnson 2001). Since the formulation emphasized the two worst condition states of each element and gave a value of 100 to the worst possible condition, it was termed the Decay Index. The decay index is computed using equation 8-2.

$$D = 100 \sum_{c} \frac{W_c}{TEV_c} \left(\sum_{s} w_{cs} CEV_{cs} \right) \tag{8-2}$$

$$CEV_{cs} = \sum_{e \in c} (P_{es} Q_e C_e)$$

$$TEV_c = \sum_{e \in c} (Q_e C_e)$$

Where P_{es} = Fraction of element *e* observed or forecast to be in condition state *s*
Q_e = Quantity of element *e* on the bridge
C_e = Unit replacement cost of element *e*
W_c = Relative weight (importance) of component c (Table 10.3)
w_{cs} = Relative weight (importance) of condition state *s* of component *c*

Equation 8-2 is organized into three components, deck, superstructure, and substructure. Current element value, total element value, and decay index are computed separately for each component, before being combined into the bridge-level Decay Index. The three components are combined as a weighted average, using W_c as the weight. This weight is determined in the model estimation process as the weighting that is most predictive of service disruptions. Since a bridge may have more than one element in a given component, the *CEV* and *TEV* are computed separately for each element before being used in the computation of Decay Index at the component level.

Equation 8-2 considers two condition states for each element. First it considers the worst defined state, which may be either state 4 or state 5 in the element inspection manual. (All elements having elemkey<300 have at least 4 condition states.) The worst condition state is always given full weight, w_{c1} = 1.0. Second the model considers the second-worst condition state, which is either state 3 or state 4 in the element inspection manual. The weight of this state is determined in the model estimation process and must be less than or equal to 1.0.

The component and condition state weights were developed as a part of the same process used in selecting a disruption model. Their optimal values did vary with different model formulations. One model selection criterion was to give preference to models that used intuitively reasonable weights of components and condition states, which were as consistent as possible across components. In the final stages of model estimation, the weights were constrained to round numbers in order to improve the ease of use of the product. Table 8.9 shows the final weights.

Table 8.9. Final coefficients for the decay index

	Component weights W_c				2nd-worst state w_{cs}		
Material	Deck	Super	Substr		Deck	Super	Substr
Concrete – prestressed	20%	40%	40%		50%	50%	50%
Concrete – reinforced	20%	40%	40%		50%	50%	50%
Steel	20%	40%	40%		50%	50%	50%
Timber	40%	40%	20%		10%	50%	50%

The models were not very sensitive to variations of any of these weights within a tolerance of 10-20 percentage points. The loss of predictive power was insignificant in constraining concrete and steel to use the same round-number weights. Only timber bridges showed significantly different behavior that could not be fit with the same parameters as other bridges. All of the models performed badly if the weight of the second-worst state was set to zero.

8.6.2. Disruption likelihood model

The disruption likelihood model predicts the probability of a service disruption as a function of decay index. The three types of service disruptions (bridge retirement, reconstruction, and restriction) were investigated separately and in various combinations. The simplest and most robust model was found to be a combination of all three types of disruption into the same model.

It was found that the choice of the type of agency response to deteriorated conditions was a difficult phenomenon to model with any of the available data other than material type. As a result, modeling the three types of disruption separately gave in each case an incomplete picture of the agency response to deteriorated conditions, which failed to produce reliable predictive models. It was only when the three responses were combined into a single model that a clear and consistent picture emerged.

As described above, four alternative model forms were evaluated, each with a valid intuitive rationale. It turned out that a combination of the linear and lognormal models produced consistently strong chi-squared evaluations with the actual data. The final model for service disruption in a given inspection is described in equation 8-3.

$$P^L = \frac{a\varphi\left(\frac{ln(D)-\mu}{\sigma}\right)}{\sigma D\left(1-\Phi\left(\frac{\ln(D)-\mu}{\sigma}\right)\right)} + bD + c \qquad (8\text{-}3)$$

where: *D* = decay index as computed below
ln(D) = natural logarithm of the decay index
μ = mean of *ln(D)* (Table 8.10)
σ = standard deviation of *ln(D)* (Table 8.10)
$\varphi((ln(D)-\mu)/\sigma)$ = probability density function of the normal distribution
= NORMDIST(ln(D),μ,σ,FALSE) in Excel
$\Phi((ln(D)-\mu)/\sigma)$ = cumulative distribution function of the normal distribution
= NORMDIST(ln(D),μ,σ,TRUE) in Excel
a,b,c = regression coefficients (Table 8.10)

Note that the exact value of the probability density function, and an approximation of the cumulative distribution function, can both be computed using standard formulas. But it is simplest to use the built-in Excel functions. In this formulation, *ln(D)* is conceptualized as roughly proportional to the length of time spent in the worst or second-worst condition states. The first term of the equation, with regression coefficient *a*, is the textbook hazard function of a lognormal probability distribution, representing the cause-and-effect relationship between advanced deterioration and service disruption. The second term in the equation, with the coefficient *b*, represents a linear growth of disruption probability with advancing deterioration. It was found to represent a period of increasing risk where a bridge might be posted and monitored, while waiting for funding for bridge reconstruction or replacement.

The coefficients *a* and *c* are not strictly necessary for this model. The value of *a* is related to the values of *b*, μ, and σ and merely provides convenient scaling. The value of *c* is always near zero in the best-fit models. However, these coefficients were found to aid the Excel solution algorithm in avoiding numerical problems while searching for an optimal model. Table 8.10 shows the final coefficients and parameters of the models. Figure 8.3 shows the models graphically.

Table 8.10. Coefficients and distribution parameters of the final model

Material	Coefficients				Distribution	
	a	b	c		μ	σ
Concrete – prestressed	3000	0.00199	0		15.461	3.982
Concrete – reinforced	3000	0.00047	0		15.385	3.928
Steel	3000	0.00196	0		15.545	3.998
Timber	3000	0.00539	0		15.077	3.902

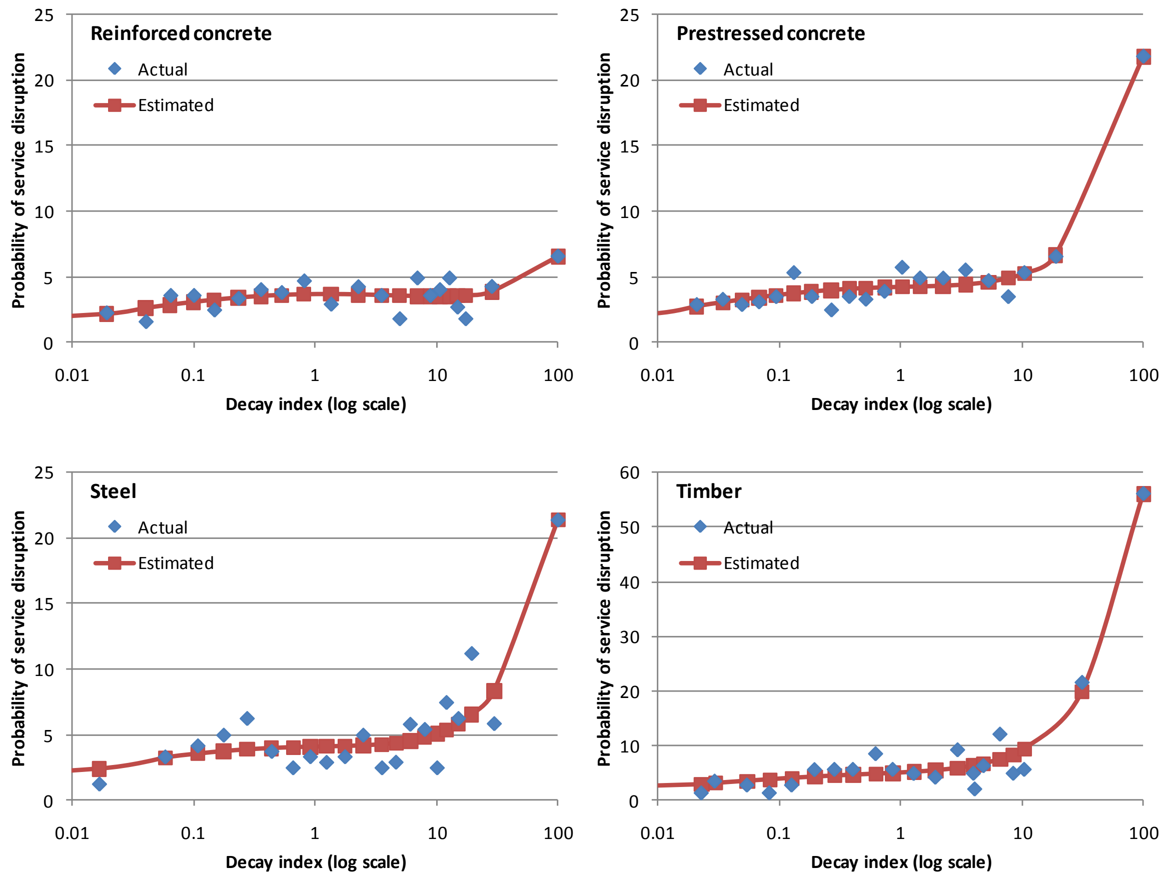

Figure 8.3. Final hazard functions (“estimated”) plotted with binned observations (“actual”) in percent

In this regression analysis, the lognormal probability distribution is unknown because the vast majority of data are on just the left tail of the distribution. This is why the parameters μ and σ must be estimated rather than computed directly. A significant fraction of Florida bridges are replaced or reconstructed relatively soon after significant quantities of elements reach the worst condition state. But for bridges that are not addressed right away, the linear model provides the longer-term pattern of increasing risk.

8.6.3. Life cycle cost of service disruption

In a life cycle cost model, the hazard function provides the likelihood that service will be disrupted by bridge replacement, reconstruction, or posting. In a year-by-year simulation of future costs, the life cycle cost of service disruption can be computed using equation 8-4.

$$\mathrm{LCC}_y = \mathrm{discrate}^y \times \mathrm{P}^L(\mathrm{D}) \times \mathrm{impact} \tag{8-4}$$

where:	discrate	= discount rate = 1/(1-*i*) where *i* is the real interest rate
	y	= year of the condition forecast (number of years to discount)
	D	= decay index (equation 8-2) computed from forecast element conditions
	$P^L(D)$	= disruption probability for the forecast decay index D (equation 8-3)
	impact	= total social cost (agency+user+nonuser) of a full service disruption

As long as no deck, superstructure, or substructure elements on a bridge are in their worst or second-worst condition state, there is no risk of service disruption and the life cycle disruption cost is zero. When Markovian deterioration causes an element to enter its second-worst condition state, the decay index will be very small, and the disruption index will also be very small. The decay index builds up more quickly when elements begin to enter their worst condition state. Table 8.4 above shows the typical buildup of conditions in the worst condition states with a Markovian model. Most of the Florida deterioration models build up more slowly than the example in Table 8.4.

As the binned data points in Figure 8.3 show, relatively few bridges in the Florida inventory, even at the local level, are allowed to deteriorate past a decay index in the 10-20 range without receiving some sort of maintenance. This is the area where the risk of service disruption starts to build up more rapidly. If maintenance is not performed and deterioration continues, the risk of service disruption becomes a significant part of the life cycle cost of the bridge. This creates a substantial penalty for deferred bridge maintenance.

If a bridge is not maintained for a very long time, the decay index approaches 100 asymptotically under Markovian deterioration. The hazard function continues to increase (moderated by the discount rate), and life cycle cost continues to build up without limit. As a result, there is no need for a “minimum failure cost” when this model is used, because it is always optimal to intervene at some point sooner or later in the deterioration process.

8.6.4. PLAT modification

The Project Level Analysis Tool (PLAT) currently contains a “failure risk” model based on the Pontis failure probability. This acts as a penalty for allowing a bridge to deteriorate excessively. It is recommended that the disruption likelihood model replace the failure risk model in the PLAT software. This will improve the realism of the life cycle cost analysis. The framework described earlier in this memorandum will also accommodate the remainder of the risk analysis being prepared under this current study.

8.6.5. Cumulative disruption

For certain applications it may be useful to estimate the total life cycle cost impact of the buildup of risk on a bridge that is allowed to deteriorate. In a full-scale analysis such as PLAT, the buildup can be computed year by year as described earlier, which is the most precise way to do it. For a quicker analysis, however, the cumulative hazard function can be used.

The cumulative disruption probability at a particular level of deterioration $D_i=D$, based on equation 3 above, is the area under the hazard function curve up to decay index D. This is shown in equation 8-5.

$$cdp(D) = \int_{D_i=0}^{D} \left(\frac{a \times \varphi\left(\frac{\ln(D_i) - \mu}{\sigma}\right)}{\sigma D_i \left(1 - \Phi\left(\frac{\ln(D_i) - \mu}{\sigma}\right)\right)} + b \times D_i + c \right) dD_i \tag{8-5}$$

With c=0 and a=3000 this integral works out to equation 8-6.

$$\mathrm{cdp(D)} = -3000 \times \ln\left(1 - \Phi\left(\frac{\ln(\mathrm{D}) - \mu}{\sigma}\right)\right) + \frac{\mathrm{b} \times \mathrm{D}^2}{2} \tag{8-6}$$

where:

D_i	= decay index as computed in equation 2 in inspection i
$ln(D_i)$	= natural logarithm of the decay index
μ	= mean of $ln(D_i)$
σ	= standard deviation of $ln(D_i)$
$\varphi((ln(Di)-\mu)/\sigma)$	= probability density function of the normal distribution
	= NORMDIST($ln(D_i)$,μ,σ,FALSE) in Excel
$\Phi((ln(Di)-\mu)/\sigma)$	= cumulative distribution function of the normal distribution
	= NORMDIST($ln(D_i)$,μ,σ,TRUE) in Excel
a,b,c	= regression coefficients

Figure 8.4 shows equation 8-6 graphically. The cumulative disruption probability is the total fraction of service disruption consequences to be recognized in a life cycle cost model, if condition reaches a given level of decay index. As conditions deteriorate, the cumulative likelihood of disruption becomes higher and can exceed 1.0 if the bridge is not replaced. This model is based on condition, not time, and does not consider discounting. Depending on the application, approximations of the effect of discounting and deterioration rates can be developed to scale the cumulative disruption model appropriately.

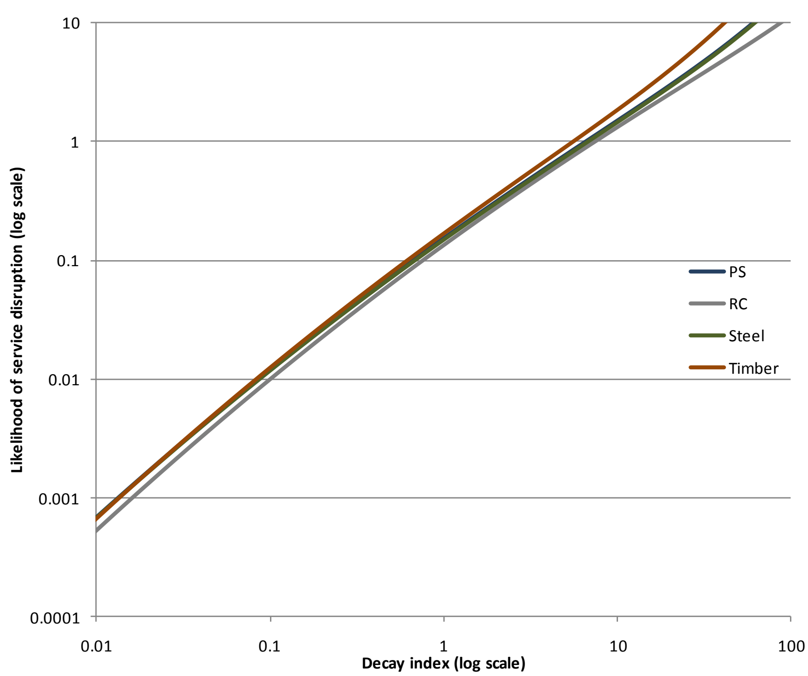

Figure 8.4. Cumulative disruption probability

8.6.6. Pontis failure probability

The analysis for this report produced, as a by-product, the data necessary to compute failure probabilities for Pontis. Pontis is more rigid than PLAT in its risk analysis and has some very specific requirements for the failure probability, which differ from the assumptions made up to this point:

- Failure probability only applies to elements in their worst condition state. The second-worst state cannot play a role.
- Each element accounts for failure separately from all other elements.
- The failure probability model cannot depend on any bridge level data. It can rely only on the type of element and the fraction in the worst condition state.
- The model must be linear. The probability of a bridge failure is a direct multiple of the fraction in the worst condition state.

For the purpose of this analysis, it is valid to define the Pontis term "failure" in the same way that service disruptions are defined for the risk analysis. So the same three disruption types are used, computed in the same way:

- Evidence of bridge closure or replacement, provided by the assignment of a bridge to fictitious "District 9 – Central Office." The reason for bridge retirement must be deterioration or an unknown reason (reason codes "D" and "?"), thus omitting bridges replaced for functional or administrative reasons.
- Evidence of bridge reconstruction, from National Bridge Inventory (NBI) item 106 – Year reconstructed. This is defined as the most recent year in which the bridge underwent work eligible for Federal funding (regardless of how actually funded). This can include total replacement, superstructure replacement, and functional improvements such as widening.
- Operational status of posted or closed as of the following inspection, based on a value of NBI item 41 of B, D, E, K, P, or R, provided that the bridge is not already posted in the current inspection.

To compute the failure probability, a data set was prepared of element inspections having a non-zero quantity in the element's worst condition state. Element inspections were excluded if they had a zero quantity, had an inspkey value of 'STRT', or took place after 7/1/2009 (since some of these would not yet have a follow-up inspection in the database used in the analysis). All element types were included.

Element types were grouped into categories, mainly by material and role in the bridge structure. The purpose was to ensure valid sample sizes while combining elements likely to have similar failure probabilities. For each category, the failure probability is computed as in equation 8-7.

$$\mathrm{FP}_c = 100 \times \frac{\left(\sum_{i \in c} \mathrm{Fail}_i\right)}{\left(\sum_{i \in c} \mathrm{PW}_i\right)} \tag{8-7}$$

where: $Fail_i$ = 1 if the bridge suffered service disruption before the next inspection, 0 otherwise
PW_i = fraction in the worst condition state in element inspection i

Table 8-11 shows the results of this computation. An SQL UPDATE script will be provided so FDOT can enter these results into the condumdl table in the Pontis database.

Table 8-11. Computed Pontis failure probabilities

Material	Count of element inspections			Failure
	Total	Failed*	Worst**	Probability
Deck/Slab	356	43	356	12.1
Steel	1148	82	386	21.2
Concrete	3357	151	783	19.3
Other	4146	299	1322	22.6
Joint	13860	474	8219	5.8
Bearing	901	39	370	10.6
Appurtenance	4172	172	1451	11.9
Smart	380	35	380	9.2
Movable	1957	118	1433	8.2

* Bridges replaced, rebuilt, or posted before next inspection

** Total of fraction in the worst condition state

8.7. References

AASHTO. (2001). *AASHTO Guide for Commonly-Recognized (CoRe) Structural Elements*, Interim revision to the 1997 document. Washington: American Association of State Highway and Transportation Officials.

FDOT. (2011). Bridge Management System Coding Guide. Florida Department of Transportation.

Shepard, R.W., and Johnson, M.B., (2001). California Bridge Health Index: A Diagnostic Tool to Maximize Bridge Longevity, Investment. *TR News* 215, available online at http://onlinepubs.trb.org/Onlinepubs/trnews/trnews215full.pdf.

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9. Fatigue

Fatigue is a deterioration process where a material flaw, initially microscopic in size, develops into a larger defect and eventually a crack. This occurs because of a concentration of stress in the vicinity of the flaw, which is cyclically applied by the passage of live loads on the structure, or by distortion of the structure. As the crack grows, the stress concentration becomes more severe, which may further accelerate the growth of the crack. This unstable situation may quickly cause complete failure of the structural member (Yen et al., 1990).

For bridge management analysis, fatigue of structural steel is of great significance. This may occur on any steel members that are primarily under tension. Steel members and their connections are modeled as having a finite, quantifiable, probabilistic limit on the number of cyclical loadings they may endure before the probability of the onset of cracking becomes unacceptable. Based on traffic studies, structural analysis, and empirical research, each member and connection is designed so that its fatigue life is not reached prior to its design life (AASHTO 2012).

9.1. Level of detail

Fatigue design of bridges is heavily focused on the connections between steel members, because these welded, riveted, or bolted details are often associated with stress concentrations and construction defects. The fatigue resistance at a given location is dependent on forces and their variation under live loads at the given location, member sizes, fastener and weld characteristics, and quality assurance processes. These are designed and managed during the design and construction phases of a bridge’s life with great attention to detail.

Once a bridge is placed into service, the characteristics that affect fatigue resistance are subject to a variety of changes, which are difficult to measure and track in an economical way. The weight and composition of traffic may differ from design assumptions, as may temperature changes, winds, and other live loads. Development of material defects is a random variable, as is the discovery of such defects. The fatigue life of a structure as a whole is influenced by the fatigue life characteristics of many individual details.

A bridge management system generally does not have reliable access to the structural computations that affect design detailing, nor to a comprehensive condition survey of design details on in-service bridges. Therefore a risk analysis in bridge management is necessarily less rigorous, using simplifying assumptions to reduce data requirements.

For guidance on the identification of reasonable simplifications, this study relies especially on the framework of NCHRP Report 495, “Effect of Truck Weight on Bridge Network Costs” (Fu et al., 2003). Although Report 495 was developed for an entirely different purpose (analysis of increased truck weight limits on routes or networks), it relies on the AASHTO fatigue life model, which is an integral part of current bridge design and maintenance practices across the nation. In a manner similar to Report 495, the current study uses the following simplifications:

- Each bridge is represented by its most critical fatigue-prone details, with criticality determined both by the category of detail, and the effective stress range.
- A very rough determination is made, based on available Pontis data, of whether each bridge is likely to have highly fatigue-prone details. It is assumed that the FDOT will be able to refine this determination later as a part of implementing the new model.

- Similarly, a default stress range assumption for critical details is made for initial analysis. It is assumed that once the most significant bridges are identified by the initial analysis, the stress ranges can be refined further for individual structures, using the FDOT's routine load rating procedures.
- Fracture criticality is assessed in a very cursory way from NBI data, but can be refined later where needed.
- Traffic growth is assumed to follow a continuous geometric progression as described by the applicable NBI traffic count fields.
- The likelihood of fatigue cracking is modeled using a simple probability distribution derived from the AASHTO fatigue life model.
- A simple cost and effectiveness model is used in order to quantify consequences and impacts, using a set of prototypical repair options.

Given the combined effect of these assumptions, the initial modeling results may be useful primarily as a screening tool, to identify the bridges most in need of further analysis. However, the framework permits refinement so the addition of a few additional data items can provide a much more precise risk assessment.

9.2. Elements of fatigue risk

In this study, risk is measured using the product of likelihood and consequence, and described as vulnerability to fatigue cracking as the applicable hazard. A risk mitigation action, crack repair, is developed in order to reduce the likelihood of member failure. Replacement of the superstructure (or of the entire bridge) is an action which can restore the fatigue life of the structure. Resilience is the remaining fatigue life of the bridge. (Figure 9.1).

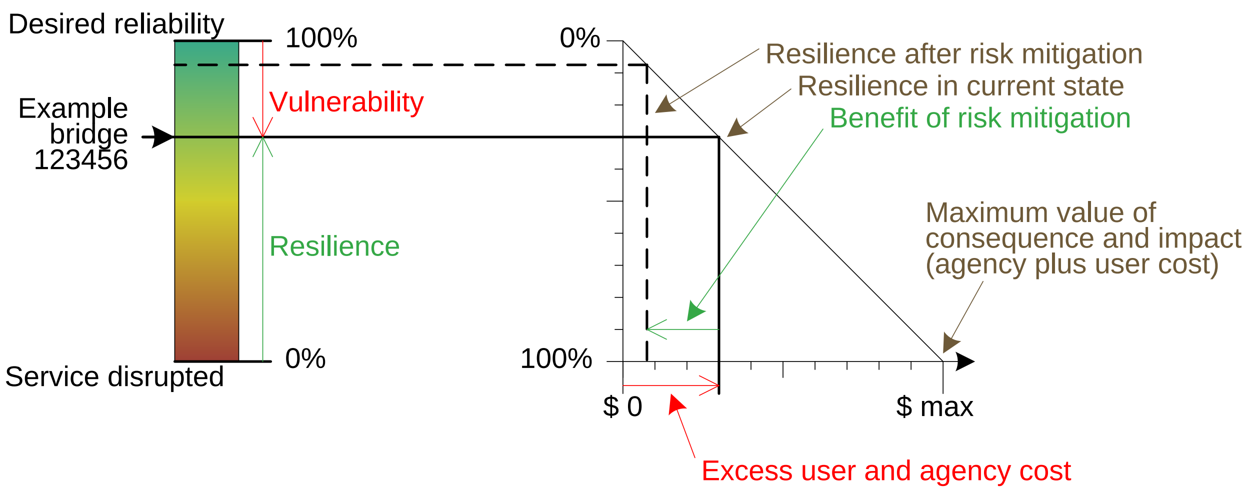

Figure 9.1. Framework for quantifying risk

Since it is desired to express performance in the form of economic measures if possible, a lack of resilience, at either the asset level or the network level, will be expressed as an excess social cost. For redundant structures, the social cost is merely the probabilistic agency cost of crack repairs. For non-redundant structures, a user cost of traffic disruption is also applied, to represent the possibility that a bridge may need to be restricted while repairs are underway. When resilience is increased due to superstructure replacement, the result is a decrease in expected value of social cost. The positive contribution of a risk mitigation action will therefore be measured as a social benefit.

9.3. Categories of fatigue-prone details

Bridge inspectors are trained to recognize a variety of fatigue-prone details, which may receive special attention for condition evaluation. The details are described in categories as follows (Ryan et al., 2012).

Category A includes "base metal" or plain material with rolled or cleaned surfaces, away from welded, riveted or bolted connections. This condition has the best fatigue resistance. It is not common practice to examine these base metal regions for fatigue cracks unless the regions are susceptible to distortion, because cracks usually develop at nearby connection details with lower fatigue strength categories.

Category B includes the following welded structural details and high strength bolted joints:

- Longitudinal continuous welds in built-up plates and shapes.
- Transverse full penetration groove welds with weld reinforcement ground smooth and weld soundness established by non-destructive testing (NDT).
- Groove welded attachments with a transition radius not less than 610 mm (24 inches).
- High strength bolted connections.

Category B' includes details similar to those of Category B, but found to be more sensitive to fatigue:

- Longitudinal continuous welds in built-up plates and shapes not detailed in Category B.
- Transverse full penetration groove welds with reinforcement ground smooth to provide straight transition in width or thickness, slopes of transition not steeper than 1 to 2.5, and base metal A514 or A517.

Category C and C' includes transverse stiffeners, very short attachments, and transverse groove welds with reinforcement not removed.

- Base metal at welds connecting transverse stiffeners or vertical gusset plates to connection and gusset plates of girder webs or flanges.
- Transverse full penetration groove welds, weld reinforcement not removed, but with weld soundness established by NDT.
- Groove or fillet welded horizontal gusset or attachment, the length of which (in the direction of the main member) is less than 50 mm (2 inches).
- Groove welded attachments 150 to 610 mm (6 to 24 inches) in length with transition radius.
- Intersecting plates connected by fillet welds with the discontinuous plate not more than 13 mm (½ inch) thick.
- Shear connectors.

Category D includes welded short attachments, welded connections with sharp transition curves, and riveted joints.

- Welded attachments with a short groove or fillet weld in the direction of the main member between 50 and 100 mm (2 and 4 inches) long but less than 12 times the plate thickness.
- Groove welded attachments with transition radius between 50 and 150 mm (2 and 6 inches).
- Groove welded attachments with unequal plate thickness, weld perpendicular to attachment, weld reinforcement removed, and a transition radius of at least 50 mm (2 inches).
- Longitudinally loaded fillet weld with length in the direction of stress of 50 to 100 mm (2 to 4 inches).
- Riveted connections, net section.

Category E and E' includes details that have the lowest fatigue strength in comparison to those in other categories. Generally, for welded details in this group with the same configurations, Category E' applies if the flange plate thickness exceeds 20 mm (0.8 inch) or if the attachment plate thickness is 25 mm (1 inch) or more.

- Ends of partial length cover plates on girder or beam flanges.
- Welded attachment, with groove or fillet weld in the direction of the main members, more than 100 mm (4 inches) or 12 times the plate thickness.
- Welded attachment with curved transition radius.
- Welded attachment with loads transverse to welds.
- Intermittent fillet welds
- Shear stress on the throat of a fillet weld (Formerly classified Category F)
- Deck plate at the connection to floorbeam web.

Of all the details, those in Categories E and E’ are the most susceptible to fatigue crack growth. These details should be closely examined at every inspection. NCHRP Report 495 (Fu et al., 2003) recommends that categories E and E’ be emphasized for evaluation of fatigue life and costs, but provides the capability to include other categories if desired. The AASHTO LRFD Design Spec and the AASHTO Manual for Bridge Evaluation (AASHTO 2011) provide fatigue life and resistance factors for all of these categories.

The categorization of fatigue details has a significant effect on the likelihood of the onset of cracking. Details in the more fatigue-prone categories are able to withstand fewer loading cycles at any given stress range.

9.4. Redundancy

Redundancy in a structure provides more load paths or elements of support than are necessary for stability. Thus, if one load path is interrupted, the alternative paths are able to pick up this load without instability or failure. Redundancy primarily influences the consequences of fatigue: in a non-redundant structure a fatigue crack can lead to significantly larger risk to road users. There are three types of redundancy:

- Load path redundancy typically exists if a bridge has three or more main load-carrying members between substructure supports. In some cases more than three members may be required for stability. This is determined by structural analysis.
- Structural redundancy may exist if there is continuity of a load path from span to span, typically over three or more spans. If one of the middle spans is broken, the adjacent spans may be able to pick up the load without loss of stability.
- Internal redundancy may exist if a member consists of three or more elements, mechanically connected to provide alternative load paths. Riveted box shapes in truss members often exhibit internal redundancy, as do wire ropes.

A bridge is fracture critical if it contains at least one fracture-critical member in tension. A member is fracture critical if its failure would cause a “collapse,” defined as a change in the bridge geometry that would render it unfit for use. Usually only load path redundancy is considered when making a determination of fracture criticality of a bridge.

When a fatigue crack occurs in a known location, a determination can be made about the consequences of the crack in terms of any of the three types of redundancy. However, in a risk analysis the location of future cracks is unknown, so it is impossible to be certain whether a future crack would be protected by structural or internal redundancy. Therefore, only load-path redundancy is considered.

Similarly, not all cracks in a fracture-critical member necessarily have consequences to bridge users, in terms of restricting service. This of course would depend on the location and severity of the crack. However, since these characteristics are unknown in a risk analysis, a conservative approach assumes that all such cracks have consequences to road users, or will have consequences within a short time if not repaired.

9.5. Available data

Fatigue and fracture criticality have been identified as a significant issue on a relatively small number of Florida's bridges. Table 9.1 summarizes the data available. A total of 519 structures have been identified as requiring fracture-critical inspections, according to NBI item 92AA (Pontis field inspevnt.fcinspreq).

Table 9.1. Summary of fracture criticality and fatigue

		Bridges		Fatigue smart flags	
		Fracture critical inspection required:			
Fracture-critical configurations		Y	N	Y	N
A	Steel 1- or 2-grider systems	206	17	24	1
B	Pin and hanger systems	1	1	0	0
C	Steel bent caps in tension	59	1	1	0
D	Steel trusses	87	17	1	0
F	Suspension or cable struc	1	2	0	0
G	Steel box girders	74	3	3	0
J	Horizontally-curved girders	2	2	0	0
L	Electroslag welding	52	21	0	0
O	Super/sub integral framing	7	0	0	0
P	Tied arches	1	0	0	0
	None	29	30583	0	66
	Total	519	30647	29	67

FDOT also maintains the Pontis field bridge.fc_detail, which indicates bridge configurations lacking load-path redundancy, which are potentially fracture critical. A total of 554 bridges have this indication. In addition, FDOT uses smart flag 356 to indicate observed fatigue cracking. Only 96 bridges have this flag in their most recent inspection, and most of these do not have fracture critical configurations or fracture critical inspections. The Pontis database does not have any information on fatigue detail categories, nor stress ranges at critical locations.

Of bridges that have either fracture-critical configurations or inspections, 68% are on the state highway system. Table 9.2 shows the breakdown by structure design type. All of the high-mast light poles in this group are there because of electroslag welding, with 71% having fracture critical inspections required.

Of the 96 structures with fatigue smart flags, 2 are in state 3 and 27 in state 2, of which only 4 are bridges. The rest of the smart flags are in state 1.

Table 9.2. Fracture criticality or fatigue, by design type

Design type (main)		Fracture critical bridges*	Fatigue smart flags
0	Other (NBI)	3	0
1	Slab	10	0
2	Stringer/Girder	56	14
3	Girder-Floorbeam	43	3
4	Tee Beam	5	0
6	Single/Spread Box	98	3
7	Frame	2	0
9	Truss-Deck	1	0
10	Truss-Thru	90	0
12	Arch-Thru	1	0
13	Suspension	2	0
14	Stayed Girder	1	0
15	Movable - Lift	7	0
16	Movable-Bascule	175	22
17	Movable-Swing	12	1
22	Channel Beam	1	0
91	Sign-Cantilever	0	19
92	Sign-Span	1	31
93	Sign-Butterfly	0	2
96	High Mast Light	73	1
97	Traffic Signal Mast Arm	2	0
	Total	583	96

* Structures with fracture critical inspections or fracture critical configurations identified

9.6. Identifying structures of interest

Appendix A of NCHRP Report 495 (Fu et al., 2003) identifies several common fatigue-prone bridge details that are especially likely to affect fatigue life. These details can exist on any type of bridge, not limited to bridges having fracture-critical configurations or inspections.

- End welds of partial-length coverplates. Commonly found on rolled-beam bridges built before 1975.
- Termination of longitudinal web stiffeners. Common on plate girder bridges with spans longer than 130 feet.
- Longitudinal connection plates. Common on bridges with spans longer than 150 feet, built before 1980.

Other types of details can also be critical for fatigue life, particularly category E or E' details. In general, any steel bridge with truck traffic on it, built before 1980, could potentially have fatigue details that are near the end of their fatigue life. Non-bridge structures are not included, since the fatigue life model depends on truck volume as a critical input. Excluding district 9 (retired bridges) and bridges missing data in critical fields (*adttotal*, *adtyear*, *yearbuilt*, *truckpct* (if *funcclass* is also missing), *trafficdir*, and *lanes*), there are 518 such bridges in the Pontis database. In addition, there are 12 bridges not designated as steel in NBI items 43 or 44, but marked as requiring fracture critical inspections in NBI item 92. So the total number of bridges of interest is 530.

It is likely that a significant fraction of these bridges have an infinite (or very long) fatigue life, and therefore no fatigue risk, due to:

- Lack of fatigue-prone details.
- Low stress ranges on all fatigue-prone details.
- Low truck volume.

For screening purposes, all of these bridges will be included. However, the fatigue risk analysis readily eliminates most of them from further concern due to low truck volume or relatively young age.

9.7. Likelihood of cracking

The recommended risk assessment methodology uses an adaptation of the approach described in NCHRP Report 495, which is based on the AASHTO fatigue life model. The methodology uses the probability distribution implicit in the AASHTO model to quantify the risk of new crack development. Consequences are defined as repair actions that are necessary to arrest or repair cracks, to ensure that each crack doesn't grow and become a fracture. The expected value of agency and user costs is then a measure of risk.

9.7.1. AASHTO fatigue life model

AASHTO's fatigue life model was first developed in NCHRP Project 12-28(03) and published in Report 299 (Moses et al., 1987). It was first codified in the 1990 AASHTO Guide Specification for Fatigue Evaluation of Existing Steel Bridges. With minor revisions, the model was carried over to AASHTO's current Manual for Bridge Evaluation (MBE, AASHTO 2011, section 7.2.5). In the AASHTO model the total finite fatigue life of a fatigue-prone detail, in years, is determined as

$$Y = \frac{R_Y K_Y}{365n(ADTT)_{SL}\left[(\Delta f)_{eff}\right]^3} \tag{9-1}$$

where R_γ = Resistance factor for evaluation or mean fatigue life from MBE table 7.2.5.2-1
K_γ = Detail category constant, from LRFD design spec table 6.6.1.2.5-1
n = Number of stress range cycles per truck passage, from LRFD table 6.6.1.2.5-2
$(ADTT)_{SL}$ = Average number of trucks per day in a single lane averaged over the fatigue life
$(\Delta f)_{eff}$ = Effective stress range, estimated from structural analysis or *in situ* measurement

The resistance factor for mean fatigue life was calibrated for the mean (and median, since the distribution is symmetrical) of the probability density function of the natural log of fatigue lifespans observed in the research data set. It provides the expected value of time until the onset of cracking for a given detail, and varies according to the fatigue detail category. For evaluation fatigue life, the resistance factor is determined from the same probability density function, but taken one standard deviation earlier than the mean fatigue life, where the probability of failure (onset of cracking in the given detail) is 16 percent. These are tabulated in Table 9.3.

Table 9.3. Fatigue model parameters

Fatigue Category	R_γ^M	R_γ^E	K_γ (ksi^3)
A	2.8	1.7	2.50 E+10
B	2.0	1.4	1.20 E+10
B’	2.4	1.5	6.10 E+09
C	1.3	1.2	4.39 E+09
C’	1.3	1.2	4.39 E+09
D	1.6	1.3	2.20 E+09
E	1.6	1.3	1.10 E+09
E’	2.5	1.6	3.90 E+08

In order to adapt the AASHTO model to a network-wide risk evaluation process, it is necessary to make certain simplifications to accommodate the data available for risk assessment. To do this, the following definitions are used:

A_M = Mean age at onset of cracking (years), an estimate of Y in equation 9-1.

R_γ^M = Resistance factor (unitless) for detail category identified by subscript γ and for the mean of the log fatigue life distribution, equal to R_γ in equation 9-1.

K_γ = Empirical fatigue constant for detail category identified by subscript γ (ksi to the third power), equal to K_γ in equation 9-1.

C = Number of stress range cycles per truck passage, simplified to be 1.0 for bridges having maximum span length (bridge.maxspan) of greater than 12m (39.372 feet), and 2.0 for shorter-span bridges, an estimate of n in equation 9-1.

T_A = Average annual single-lane truck volume estimated over the mean expected fatigue life of the bridge, equal to 365 × $(ADTT)_{SL}$ (trucks per year).

S = Effective stress range at the detail to be analyzed (ksi), including any applicable load factor as described in MBE section 7.2.2.

With these slightly refined definitions, the basic equation for mean fatigue life is

$$A_M = \frac{R_Y^M K_Y}{C T_A S^3} \tag{9-2}$$

This equation is quite sensitive to stress range, and also sensitive to the fatigue detail category. Because of this sensitivity, as well as the scarcity of data on fatigue details, it should be adequate to select the one most critical detail on each bridge.

A group of District Structure Maintenance Engineers (DSMEs) was polled by email to gain insight into reasonable default values for the fatigue detail category and stress range. It was found that fatigue details in the E and E' categories with cracking problems are extremely rare, and are typically under some form of monitoring in order to reduce their risk. It was decided therefore to use category D as more typical for newly-discovered fatigue problems.

A conservative default value of 5 ksi was selected for the stress range. Stress ranges can vary widely, so bridges found to have significant risk should be evaluated to assign a more accurate detail category and stress range.

The definition of T_A above is easy to use only if future traffic volume is constant. If traffic is forecast to grow over time, then the fatigue life will be shorter. The analysis then requires a refinement to take into account the fact that fatigue life depends on T_A, which in turn depends on fatigue life. The next section describes how this can be handled.

9.7.2. Truck traffic volume and growth

Pontis provides, for each bridge roadway, a past traffic count and a future one. This enables the calculation of a growth factor. Most bridges in the Florida inventory have a growth projection assigned to them. The traffic counts are for all lanes in both directions. A truck percentage is usually given separately. With this set of data, the cumulative traffic over a bridge's fatigue life can be computed as the area under a traffic growth curve defined as a geometric progression. Then the annual average single-lane truck volume over the mean fatigue life is:

$$T_A = \frac{1}{A_M} T_1 \sum_{a=1}^{A_M} g^{(a-1)} \tag{9-3}$$

Where A_M = Mean fatigue life as computed in Equation 9-2.
T_1 = Single-lane truck volume in the first year of the bridge's life, computed below.
a = Index of age of the bridge.
g = Traffic growth factor, as computed below.

In equation 9-3 the growth factor can be determined from Pontis data, as follows:

$$g = \left(\frac{adtfuture}{adttotal}\right)^{1/(adtfutyear - adtyear)} \tag{9-4}$$

where all of the symbols are fields in the Pontis *roadway* table. If *adttotal* is zero or missing, the bridge should not be included in the fatigue analysis. If any of the other variables are zero or missing, a growth factor of g = 1.0 can be assumed.

The first-year truck volume is determined from the latest traffic count and the growth factor, and adjusted to estimate the maximum single-lane volume:

$$T_1 = 365 \times \frac{adttotal}{g^{(adtyear - yearbuilt)}} \times truckpct/100 \times SLF \tag{9-5}$$

where *adttotal*, *adtyear*, and *truckpct* are in the Pontis *roadway* table, and *yearbuilt* is in the Pontis *bridge* table. If *truckpct* is missing, the AASHTO LRFD spec provides the following default values in Table C3.6.1.4.2-1:

Functional class 01 (rural interstate)	20
Functional classes 02-11 (all other rural, and urban interstate)	15
Functional classes 12-19 (all other urban)	10

If any other fields are missing, the bridge should be excluded from the analysis. SLF is the single-lane factor, based on LRFD spec table 3.6.1.4.2.1, as follows:

If trafficdir=3 or lanes=1	1.000
If trafficdir=1 and lanes=2	0.850
If trafficdir=1 and lanes>2	0.800
If trafficdir=2 and lanes=2 or 3	0.500
If trafficdir=2 and lanes=4 or 5	0.425
If trafficdir=2 and lanes>5	0.400

where *trafficdir* and *lanes* are in the Pontis *roadway* table. If either of these database fields are missing or zero, the bridge should be excluded from the analysis.

The summation of the geometric progression in equation 3 above can be transformed using basic calculus to yield a simpler version:

$$T_A = \frac{1}{A_M} T_1 (g^{A_M} - 1)/(g - 1) \tag{9-6}$$

Substituting equation 9-6 into equation 9-2 and rearranging gives the following equation:

$$A_M \times \frac{T_1 (g^{A_M} - 1)}{A_M (g - 1)} = \frac{R_\gamma^M K_\gamma}{CS^3} \tag{9-7}$$

This can be simplified and rearranged to

$$g^{A_M} = \frac{R_\gamma^M K_\gamma (g - 1)}{T_1 C S^3} + 1 \tag{9-8}$$

Taking the log of both sides and rearranging gives an expression for mean fatigue life under a constant rate of traffic growth

$$A_M = log\left(\frac{R_\gamma^M K_\gamma (g - 1)}{T_1 C S^3} + 1\right)/\log(g) \tag{9-9}$$

This equation has two special cases to be handled. If traffic volume is constant (no growth), then $g = 1$, rendering equation 9-9 singular. In this case $T_A = T_1$, so

$$A_M = \frac{R_\gamma^M K_\gamma}{C T_1 S^3} \tag{9-10}$$

If the argument of the log function in the numerator of equation 9-9 is less than or equal to zero, the equation becomes undefined. This can occur with a negative growth rate ($g < 1$), and corresponds to the situation where traffic volume is falling so rapidly that it never reaches the fatigue limit. In this case the fatigue life is infinite, and there is no fatigue risk.

9.7.3. Accounting for uncertainty

At any given time, the likelihood of the onset of cracking is modeled using a probability density function developed in NCHRP Report 299 (Moses et al., 1987). The model form assumes that the failure probability is normally distributed with *ln(Age)*. A_M is an estimate of the mean (and median) fatigue life for this distribution, and the report also provides a method for computing the evaluation fatigue life as one standard deviation of *ln(Age)* sooner. Using the transformations described in the preceding section, the evaluation fatigue life can be computed as

$$A_E = \frac{R_\gamma^E K_\gamma}{C T_1 S^3} \; if \; g = 1 \tag{9-11}$$

$$A_E = log\left(\frac{R_\gamma^E K_\gamma (g-1)}{T_1 C S^3} + 1\right) / \log(g) \; otherwise \tag{9-12}$$

where all variables are as defined above, and R_γ^E and K_γ given in Table 9.3 above. As with the computation of mean fatigue life, the evaluation fatigue life is infinite, and fatigue risk is zero, if the argument of the log function in the numerator of equation 9-12 is less than or equal to zero.

As a bridge ages, the probability of failure (onset of a crack) increases. When the age reaches A_E, the failure probability is about 16%; when it reaches A_M, the probability is 50% (AASHTO 2011, comment on section 7.2.5.1). If a crack is repaired, but nothing is done to prevent further cracks (such as bridge replacement), the probability is unchanged, and subsequently continues to increase with further aging. Therefore the cumulative probability function for the lognormal distribution is an appropriate estimate of crack probability.

Since the risk model is to be implemented in Excel, the simplest way to compute the lognormal distribution is to use the LOGNORMDIST worksheet function. At a given age A, the probability of cracking is

$$P_f(A) = LOGNORMDIST(A/A_M, 0, \ln(A_M/A_E)) \tag{9-13}$$

where the first argument is age, the second is the mean of the natural log of age in the probability distribution, and the third is the standard deviation of the distribution. In all of the arguments, age is normalized by dividing by A_M, which is why the mean is $ln(A_M/A_M) = 0$. The standard deviation is computed as $ln(A_M/A_M) - ln(A_E/A_M)$, which reduces to $ln(A_M/A_E)$.

9.7.4. Computing likelihood, step by step

The recommended approach for the likelihood model consists of two phases: a screening phase to determine which bridges merit further scrutiny; and a refinement phase to ensure that the fatigue details are accurate, for the bridges of greatest concern. The step-by-step process is described as follows.

Step 1. Create a list of all of the bridges of interest, using the methodology described in Section 1.6 above. The following SQL query will perform this function to populate the accompanying Excel file:

```
SELECT br.*,ie.flag356,ie.truss
FROM

(SELECT b.brkey,yearbuilt,designmain,designappr,mainspans,appspans,
      maxspan*3.281 as maxspan,funcclass,
      bypasslen/1.609 as bypasslen,det_speed/1.609 as det_speed,
      adttotal,adtyear,adtfuture,adtfutyear,truckpct,trafficdir,lanes,
      facility,featint,fc_detail,i.fcinspreq
FROM bridge b,roadway r,inspevnt i
WHERE b.brkey=r.brkey and on_under='1' and district<>'09'
      and i.brkey=b.brkey and i.inspkey=
            (SELECT max(ii.inspkey)
            FROM inspevnt ii
            WHERE ii.brkey=b.brkey and ii.inspdate=
                  (SELECT max(iii.inspdate)
                        FROM inspevnt iii
                        WHERE iii.brkey=b.brkey))
      and ((materialmain in ('3','4')
            and 1*designmain>=2 and 1*designmain<=17)
      or (materialappr in ('3','4')
            and 1*designappr>=2 and 1*designappr<=17)
      or i.fcinspreq='Y')
      and yearbuilt<1980 and adttotal>0 and adtyear>0
      and yearbuilt>0 and trafficdir>'0' and lanes>0
      and not (truckpct<0 and 1*funcclass<1)
      and truckpct<>0) br

LEFT JOIN

(SELECT e.brkey,
      max(if elemkey=356 then
            if qtystate5>0 then 5 else if qtystate4>0 then 4
                  else if qtystate3>0 then 3 else if qtystate2>0 then 2
                  else 1 endif endif endif endif else 0 endif) flag356,
      max(if elemkey in (120,121,125,126,130,131,135) then 1 else 0
            endif) truss
FROM inspevnt i, eleminsp e
WHERE e.brkey=i.brkey and i.inspkey=
      (SELECT max(ii.inspkey)
       FROM inspevnt ii
       WHERE ii.brkey=e.brkey and ii.inspdate=
            (SELECT max(iii.inspdate)
                  FROM inspevnt iii
                  WHERE iii.brkey=e.brkey))
GROUP BY e.brkey) ie

ON br.brkey=ie.brkey
ORDER BY br.brkey
```

This query produces all of its results in US Customary units.

In the 2011 Pontis database used in this research, there are 530 bridges meeting these criteria. The main data-related issue noted was that 25 of the bridges have truckpct=0 and are therefore omitted from the list even though they meet all the other criteria. These bridges are shown in a separate worksheet in the accompanying Excel file.

Step 2. Determine R_γ^M, R_γ^E, and K_γ from Table 9.3, assuming that the critical detail on each bridge is of category D unless better information is available. Also determine C (stress range cycles per truck passage) as it is defined for equation 9-2.

Step 3. Compute g and T_1 as described in equations 9-4 and 9-5.

Step 4. Compute A_M using equations 9-9 and 9-10. Note the special cases described for those equations.

Step 5. Compute A_E using equations 9-11 and 9-12. Note the special cases described for those equations.

Step 6. Compute $P_f(A)$ using equation 9-13. Sort the bridges by this cracking probability in descending order, to indicate the relative priority of a more detailed investigation.

Step 7. For the highest priority bridges in the step 6 list, consult Virtis or other load rating or design files to determine an appropriate fatigue detail category and critical stress range. Also review the fracture criticality indicator, which is used to determine whether the bridge is sensitive in terms of road user impacts of cracking.

Step 8. Repeat the preceding steps using the more accurate fatigue data.

At Step 6 in this process, out of the 530 bridges, there are 36 with at least a 50% probability of cracking, 62 with at least a 16% probability, and 92 with at least 1% probability. This might indicate the number of bridges needing a closer look to refine the fatigue detail category and stress range.

It is recommended that three fields be added to the Pontis userbrdg table, for the critical fatigue detail category, stress range, and user impact sensitivity. This will make it possible to retain the data over time for use in PLAT.

9.8. Consequence of cracking

Once a fatigue crack appears, it is standard operating procedure to schedule a repair action within a reasonable time frame. The cost and urgency of the repair can vary widely depending on the type and severity of the crack, its location on the structure, traffic conditions, difficulty of access, and many other factors. In a risk analysis, the particular characteristics of each repair are impossible to predict. Therefore it is necessary to define a broadly typical repair action based on available data, which can include the detail category, type of structure, number and length of spans. Since the likelihood model is probabilistic, the agency cost portion of the risk assessment is computed as

$$Fatigue\ risk\ agency\ cost = P_f(A) \times Typicalrepair\ cost \tag{9-14}$$

where $P_f(A)$ is the crack probability computed in the previous section.

Appendix A of NCHRP Report 495 (Fu et al., 2003) provides some useful guidance for estimating fatigue repair costs. The following analysis builds on this information to adapt it to the data available in a risk analysis. FDOT's Maintenance Management System does not provide cost data specifically associated with fatigue repairs. However, input from the District Structure Maintenance Engineers has been helpful in refining the estimates.

The recommended framework for estimating costs allows the repair cost to vary by type of bridge, fatigue category, and size of bridge. Since the main and approach units can have different structure types, they are estimated separately and then combined. However, only one fatigue category is used to represent the entire structure. The cost formula is as follows:

$$C^{FR} = \sum_u M_u N_u \left(F_{\gamma u} U_{\gamma u} + I\right) \tag{9-15}$$

where:

C^{FR} = Total agency cost (dollars) of repair of all fatigue cracks on a bridge in a year, given that at least one crack has been observed.

M_u = Span multiplier, indicating the number of cracks repaired per span in a typical year's fatigue repairs on a typical bridge. A value of 0.2 produces average estimated repair costs consistent with the guidance provided by the DSMEs. In addition to making the model sensitive to the length of the bridge, this multiplier also reflects the typically more difficult access found on longer bridges.

N_u = Number of spans in unit *u*. For this purpose a bridge has one main unit and may have one approach unit. Pontis provides separate data for number of spans (*mainspans* and *appspans*) and structure design type (*designmain* and *designappr*) for the main and approach units.

$F_{\gamma u}$ = Cost impact factor (unitless), which depends on the fatigue detail category γ, the design type of unit *u*, and the maximum span length on the bridge

$U_{\gamma u}$ = Agency cost, in dollars, of repairing one typical crack on a bridge with fatigue details of category γ on unit *u*, given the design type of unit *u*

I = Indirect cost, in dollars, per crack repaired, consisting of traffic control, access, and mobilization

NCHRP Report 495 (Fu et al., 2003) documents eight common fatigue repairs on steel superstructures. The authors selected a typical bridge for each repair, and prepared a cost estimate to respond to one crack. They also indicated the variance of costs with span length, as a cost impact factor. Table 9.4 summarizes this information.

The FDOT Pontis database doesn't have enough detail to assign a specific type of repair according to this table. However, the repairs can be associated with detail categories and structure design types, as suggested in Table 9.4. Using some engineering judgment about the relative likelihood of the different repairs, some average unit costs can be developed. In addition, since the repair costs were estimated in 1999, they need to be updated to 2013 dollars. This reasoning gives the following suggested repair costs:

Trusses or non-trusses, category D	$2000
Non-trusses, category E or E'	4400
Trusses, category E or E'	5200

The ENR Construction Cost Index was used for the inflation computation, providing a multiplier of 1.54. A cost of zero is suggested for the other fatigue categories in the fatigue risk computation. Truss structures (typical *designmain* or *designappr* values of 9, 10, 15, 17) are distinguished from non-trusses (*designmain* or *designappr* values of 2, 3, 4, 5, 6, 7, 8, 11, 12, 13, 14,16). It was found that the more difficult access found with truss structures was already largely reflected in the span multiplier.

Table 9.4. Typical fatigue repairs on steel superstructures from NCHRP 495

Repair	Detail	Detail cate-gory	Found on trusses	Found on non-trusses	Repair cost ($/crack)	Range of cost impact factors
B1	Welds of partial-length coverplates	E	N	Y	775	0.71-1.64
B2	Transverse connection plates welded to the tension flange	E	Y	Y	2445	0.75-2.03
B3	Welded transverse stiffener	C	N	Y	340	1.0
B4	Termination of welded longitudinal web stiffener	B	N	Y	360	1.0
B5A	Welded longitudinal connection plates - bottom flange	E	Y	Y	3415	0.75-2.03
B5B	Welded longitudinal connection plates - web	E	Y	Y	4015	0.75-2.03
B6	Riveted truss members	D	Y	N	1075	0.75-1.2
B7	Riveted built-up girder or floorbeam	D	N	Y	1380	0.51-2.7

NCHRP Report 495 (Fu et al., 2003) provides cost impact factors to reflect the higher costs on long-span bridges. Pontis provides only the maximum span length for the bridge as a whole (*bridge.maxspan*), without distinguishing main unit from approach. So that is the information used in the recommended risk model. The following cost impact factors are suggested:

For trusses with fatigue detail category D:

Up to 120 feet in span	0.75
Up to 200 feet in span	1.00
Greater spans	1.20

For non-trusses with fatigue detail category D:

Up to 65 feet in span	0.51
Up to 100 feet in span	1.00
Up to 120 feet in span	1.33
Up to 140 feet in span	1.70
Up to 160 feet in span	2.16
Greater spans	2.70

For trusses or non-trusses with fatigue detail category E or E':

Up to 100 feet in span	0.75
Up to 120 feet in span	1.00
Up to 140 feet in span	1.28
Up to 160 feet in span	1.63
Greater spans	2.03

The report also provides a separate allowance for indirect costs, described as incorporating traffic control and access, typically $1000 per repair. Allowing for inflation, an indirect cost of $1500 per crack is suggested.

Using these parameters, an average total estimated project cost for fatigue detail category D was $22,500 for trusses and $13,500 for other bridges.

9.9. Impact of cracking

It is recommended that the impact of fatigue cracking on road users be limited to fracture-critical bridges. For such bridges, it may be necessary to limit traffic while a repair is accomplished. To provide reasonably typical user costs, it is assumed that all trucks are forced to use the detour route for a period of 8 days while repairs are made, based on DSME guidance.

The recommended user cost model is similar to the one used in the 1999 FDOT User Cost Study (Thompson et al., 1999) and the 2003 failure cost analysis (Thompson 2003). It is calculated as follows:

$$C^{FD} = (CV \times DD + CT \times DD/DS) \times ADTT \times Dur \qquad (9\text{-}16)$$

where:

CV = Vehicle operating cost per detour distance. In 1999 this was estimated as \$0.27/km. Updated to 2013 dollars with the consumer price index (using a factor of 1.3828), and converted to US Customary units (1.609 km per mile), this value is 60 cents per mile.

DD = Detour distance, in miles, from the Pontis field *roadway.bypasslen*.

CT = Travel time cost per hour of detour. Updated to 2013 dollars, this cost is \$36.55.

DS = Detour speed, in miles per hour. This is provided in Pontis in the *roadway.det_speed* field.

Dur = Duration of the repair action. A default value of 8 days is recommended.

$ADTT$ = Average daily truck traffic for the analysis year. This can be computed using the information developed for the likelihood model above.

$$ADTT = adttotal \times g^{(year-adtyear)} \times truckpct/100 \qquad (9\text{-}17)$$

where g is the growth factor computed in equation 3a, and *year* is the analysis year. The fields *adttotal* and *adtyear* are taken from the Pontis roadway table for the roadway on the bridge.

With these parameters, the average estimated user cost during fatigue repair projects was \$67,600.

9.10. Final risk model

Given the likelihood, consequence, and impact models already described, the final social cost of fatigue risk is computed from

$$RiskCost = P_f \times (C^{FR} + C^{FD}) \qquad (9\text{-}18)$$

Where P_f = Cracking probability from equation 9-13.

C^{FR} = Repair cost from equation 9-15.

C^{FD} = User cost from equation 9-16.

9.11. Analysis of FDOT structures of interest

Accompanying this report is an Excel file that implements the methods described here. Since it contains all relevant bridges in the FDOT inventory, it can be used to gauge the overall effect of the various input parameters. The key inputs are highlighted in yellow in the spreadsheet. The key results for evaluation are the probability of cracking, and the final cost numbers: agency cost, user cost, and total risk cost for each bridge (columns highlighted with orange).

Using the recommended model parameters, the total social cost of fatigue risk in the 530-bridge data set is $1.9 million. For the 92 bridges having at least a 1% probability of cracking, the average risk cost is $20,400 per bridge.

9.12. References

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10. Summary of Methods for PLAT

Florida DOT implements the products of its bridge management research in the Project Level Analysis Tool (PLAT; Sobanjo and Thompson, 2004; and Sobanjo and Thompson, 20042011), an Excel spreadsheet model built on the AASHTOWare Pontis database (Cambridge 2003). The PLAT, in turn, contributes estimates of cost and effects to the Network Analysis Tool (NAT; Sobanjo and Thompson, 2007), which is used for priority setting and programming of bridge work on a district and statewide basis.

Philosophically, the performance management approach taken in the PLAT and NAT is to attempt to quantify all costs and benefits in dollar terms at the project and network levels. Each project may affect transportation system performance in a variety of ways: initial cost, life cycle cost, safety, mobility, and risk. These project benefits are considered together in a multi-objective optimization framework (Patidar et al., 2007). In the FDOT models, the utility function for this multi-objective framework is social cost, consisting of agency, user, and non-user costs.

Previous FDOT studies have quantified initial and life cycle costs, safety, and mobility, using social costs to measure ongoing performance and project benefits. The current study measures risk in the same framework.

10.1. General approach

A variety of bad things can happen to good bridges in Florida: hurricanes, tornadoes, wildfires, floods, collisions, advanced deterioration, and fatigue. The causes are, at least in part, outside agency control and subject to random external factors. They are considered to be hazards, which are quantified in terms of the likelihood of hazard occurrence. All of these hazards can cause a bridge to be damaged or destroyed, delivering a consequence to the agency (the cost to repair or replace the structure) and an impact on the public (disruption of transportation service and of the larger economy). Figure 10.1 shows the basic ingredients.

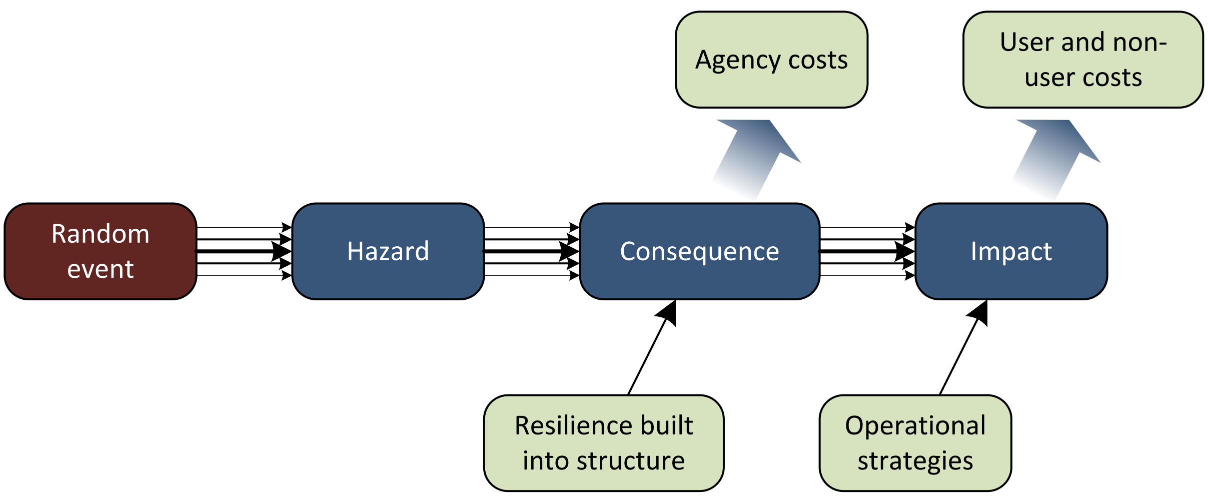

Figure 10.1. Basic ingredients of risk analysis in PLAT

Hazards are modeled probabilistically. At a given bridge site, the hazard can strike with various levels of severity that can be forecast only with a broad concept of probability distribution. An F2 tornado 500 feet wide may touch down near a bridge, pass 1000 feet from the structure, and do no damage. The same tornado with stronger winds or a slight variation in its path, may destroy the same bridge. Tornadoes can happen anywhere in Florida, and do occasionally damage bridges. It is impossible to forecast future events on one given bridge, but it is possible to quantify a general level of risk based on regional records of tornado occurrence and statewide tornado damage.

Once a hazard strikes, the damage to the structure and impact on the public are also probabilistic, subject to a limited degree of agency control. A wildfire near a bridge may engulf and destroy the structure, or may cause varying levels of repairable damage, or may spare the structure and merely disrupt traffic with a pall of smoke. Efforts by emergency crews to save the structure or to minimize the impact on traffic have varying effectiveness, depending on random factors. When a hurricane strikes, the FDOT may close bridges pre-emptively to protect life, even if the bridge is not ultimately damaged.

10.1.1. Projects and benefits

For bridge management purposes, the main decision variable in the risk analysis is the selection and timing of programmed actions to increase the resilience of the FDOT's structures, thus indirectly influencing the social costs caused by hazards. The controllable costs of structure resilience and operational strategies, are combined with the more random future outputs of agency, user, and non-user costs due to hazards, to produce forecasts of life cycle costs.

When a risk mitigation or replacement action is being considered as a way of increasing structure resilience, it is impossible to know what future benefits, if any, may accrue to that specific project. For example, when a bridge pier in the center of a river is reinforced with rip-rap, it is impossible to know whether a severe flood will occur during the life of that improvement, it is impossible to know whether such a flood would be able to damage the unimproved bridge foundation, and it is impossible to know whether the rip-rap will be effective in preventing damage. It is known that these events happen somewhere in the state every year, and every year a few structures are damaged or destroyed.

In risk analysis, the regional or statewide historical records of hazards, consequences, and impacts are summarized and used as a gross indication of future risk. This risk is allocated to specific bridges in a way that is reflective of structure resilience and significance. A bridge is assigned more risk if it has less resilience, if it is expensive to replace, or if it is used by a large number of people (Figure 10.2).

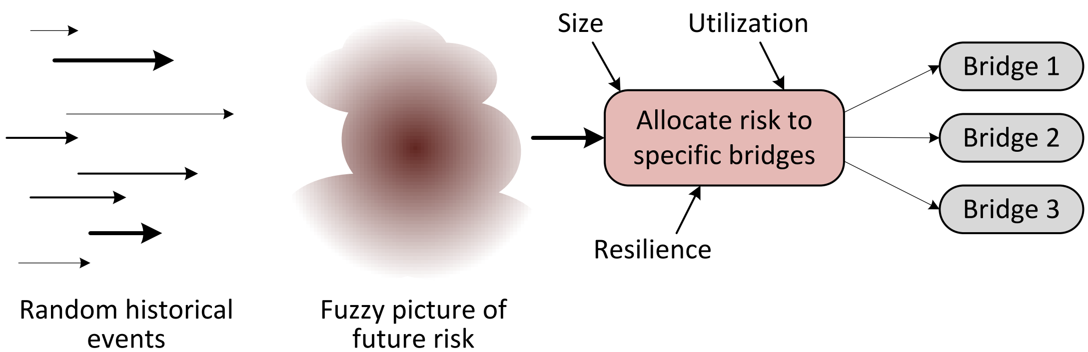

Figure 10.2. Allocation of risk based on historical events

Using this perspective, risk is spread uniformly among bridges, and from year to year over time. Risk may gradually increase over time because of traffic growth and deterioration. If a risk mitigation or replacement action takes place, resilience improves and risk is reduced for the time subsequent to the action (Figure 10.3). The life cycle cost (LCC) of this scenario is the sum of discounted social costs incurred throughout the life of the crossing served by this bridge. Risk-related costs are high without the mitigation action, and lower once the action is applied. The action itself also has a cost. If the life cycle that includes the action has lower total LCC than a life cycle without the action, then it is attractive to perform the work.

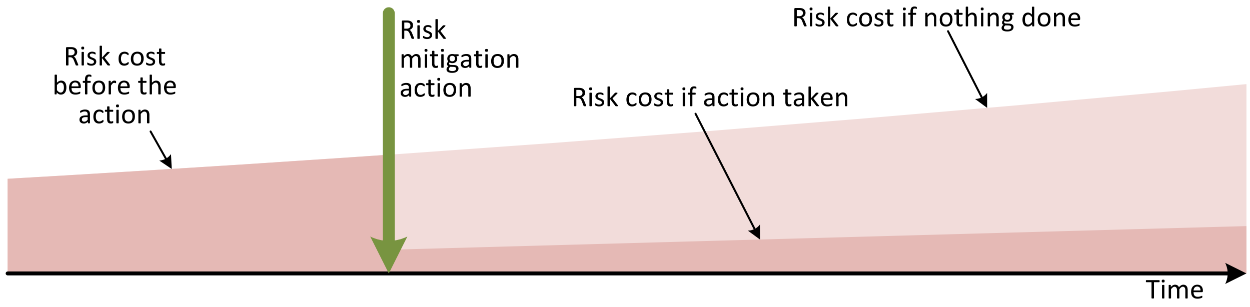

Figure 10.3. Life cycle activity profile for risk

For project selection purposes in any given year, LCC can be computed for a variety of feasible actions, including doing nothing, to select the action which minimizes LCC. The total benefit of a project is the savings in LCC relative to doing nothing.

If a project is delayed, this lengthens the period of higher risk costs, and thus increases LCC. The benefit of accelerating a project by one year is the one-year savings in life cycle cost. In a priority programming context where a limited budget must be allocated among projects each year, the best projects are those which would save the most in risk costs, relative to each dollar spent, if they are done this year rather than waiting another year.

10.1.2. Predicting risk on one bridge

In the Project Level Analysis Tool, the reason for predicting risk is to develop a reasonable and consistent basis for selecting and prioritizing projects. The available data are fairly limited, as the earlier chapters of this report have shown. In some cases it is necessary to apply judgmental parameters or rules of thumb to provide reasonableness and consistency in the absence of hard data. For many types of risk, it is necessary to estimate key parameters by a process of calibration, fitting the data in a manner that agrees with statewide damage and disruption estimates.

In this context, the risk analysis does not try to forecast specific events and responses on a given bridge, but instead looks for the most objective and consistent method of allocating risk that is sensitive to possible mitigation actions.

In any kind of forecasting model, it is important to separate long-term trends from short-term statistical noise. The fact that a tornado hit a specific bridge in the past, does not mean that particular bridge is more susceptible than the other bridges near it. Historical risk data are inherently “lumpy,” and therefore must be smoothed geographically, over time, and within bridge types, in order to make reasonable forecasts. Smoothing can be done by classifying bridges into families or geographic groups, or by fitting statistical models to bridge characteristics. The selection of appropriate smoothing methods may depend on the availability of data and models, and on a judgmental assessment of reasonable variation within the inventory.

Consistent with the philosophy discussed above, risk for one hazard on one bridge in one year is modeled as a social cost as follows:

$$C^{risk} = \sum_s \left(P_s^L \sum_c \left(P_{sc}^C D_{sc} (C^A + C^U) \right) \right) \tag{10-1}$$

Where P_s^L = Hazard likelihood probability of severity level *s*
P_{sc}^C = Consequence probability of class *c*, given severity level *s*
D_{sc} = Degree of damage in class *c*, given severity level *s*
C^A = Agency cost of recovery, per unit of D_{sc}, for class *c* and severity level *s*
C^U = User and non-user costs, per unit of D_{sc}, from class *c* and severity level *s*

This equation reflects the idea that a hazard may occur at various levels of severity, and each level of severity can produce a spectrum of consequences. The specifics of each event determine a quantity of damage to the bridge. This damage, in turn, produces agency and user/non-user costs in proportion to the quantity of damage.

To set up a suitable risk allocation process, it is useful to divide up the degree-of-damage expression into two parts:

$$D_{sc} = D \times d_{sc} \tag{10-2}$$

Where D = Maximum level of potential damage the hazard can do
d_{sc} = Fraction of potential damage that is realized in class c and severity s

So *D* is an indication of the strength of the hazard, while d_{sc} is an indication of the vulnerability (lack of resilience) of the bridge.

Now it is useful to divide up the social cost equation into three parts that can be analyzed separately. First, a unitless risk index that quantifies the combined effect of the probability distributions of severity and consequence, and their relative damage levels:

$$X = \sum_s \left(P_s^L \sum_c P_{sc}^C d_{sc} \right) \tag{10-3}$$

Next is the agency cost consequence of one unit of the risk index, which reflects the cost of repairing or replacing the bridge and recovering from an extreme event. This can be expressed as a function of bridge replacement cost as follows:

$$DC^A = Dc^A C^R \tag{10-4}$$

Where Dc^A = Recovery cost, as a multiple of replacement cost, for one unit of damage
C^R = Replacement cost of the bridge, in dollars

Here Dc^A is a scaling constant that is the same for every bridge. As would be expected, larger bridges are assigned more risk, because they cost more to repair or replace. Replacement cost is computed by multiplying bridge deck area by the average cost per square foot for bridge replacement.

Finally comes the user cost of one unit of the risk index, which reflects the inconvenience and harm to users and non-users of an extreme event. This can be expressed as a function of the maximum daily detour cost as follows:

$$DC^U = Dc^U C^D \tag{10-5}$$

Where Dc^U = User/non-user cost, as a multiple of one days' detour cost, for one unit of damage
C^D = User cost of detouring all traffic around the bridge for a day, in dollars

The daily detour cost is computed using the same methods PLAT already uses for its user cost model (Thompson et al 1999), based on traffic volume, detour distance, and detour speed. Bridges with higher volumes or longer detours are assigned more risk because the total amount of user inconvenience is greater, in the event of a service disruption. The formula for this cost is:

$$C^D = ADT(c^{VOC}BL + c^{TT}BL/BS) \tag{10-6}$$

Where ADT = Average daily traffic forecast for the year to be analyzed
c^{VOC} = Unit vehicle operating cost per mile ($0.60 in 2013 dollars, or $0.37/km)
c^{TT} = Unit travel time cost per hour ($36.55 in 2013 dollars)
BL = Bypass length in miles (NBI item 19, or roadway.bypasslen in Pontis)
BS = Bypass speed in mph (roadway.det_speed in Pontis)

With these substitutions, the risk cost in equation 10-1 can be computed as:

$$C^{risk} = XDC^A + XDC^U \tag{10-7}$$

Which is simply the expected value of agency cost plus the expected value of user cost.

10.1.3. Significance of the risk index

The risk index can be understood simply as the likelihood of a hazard times the consequence of the hazard. It is expressed in a way that does not depend on the size or utilization of the bridge. Hazard likelihood can very often be developed from historical data, as earlier chapters have shown. Sometimes (as for hurricanes) it can even be developed separately by severity level.

The consequence assessment for each hazard should depend on bridge characteristics that give it more or less vulnerability to the hazard. Any available data about vulnerability or resilience should be taken into account, at the bridge, roadway, or element levels. For example, wildfire consequences may depend on structure type, materials, and age.

For a given hazard on a given bridge, the value of the risk index *X* can be computed from the probabilities and damage levels shown in the equation above, if such data are available. However, any of these variables can be estimated or even provided from expert judgment if appropriate data are absent. In fact, the risk index itself can be assigned using judgment if there is no better basis for computing it. Fortunately, the analysis described in this report does provide more objective methods for computing *X* for each hazard. Later sections of this chapter describe how the analysis has been adapted for each hazard.

10.1.4. Unknown parameters and model calibration

Since the risk index *X* is a unitless quantity, it is up to the parameters DC^A and DC^U to act as scaling factors to relate the expected degree of consequences to actual cost of physical damage or user inconvenience. In fact, the risk index *X* can be developed on any scale as long as the parameter *D* is adjusted correctly for the same scale. For example, *X* could indicate risk on a scale of 1 to 5, 0 to 100, or even an unlimited scale (if it is a physical quantity such as truck traffic volume). It is necessary to calibrate DC^A and DC^U to make the total statewide agency and user costs add up to historically defensible totals of annual hazard-related losses.

Another way of looking at this is that the historical statewide average annual hazard loss is a control total. The parameters DC^A and DC^U should be set so that the sum of all risk costs over the entire bridge inventory, equal the control totals. The risk index *X* is a criterion to allocate the control total among the bridges in the inventory, which is why the calibration process is called risk allocation. This not much different from the very common method of cost allocation used in financial planning.

From equations 10-4 and 10-7, if agency risk costs are summed over the entire inventory, then the statewide annual agency cost of the hazard is computed by summing over all bridges *b* as follows:

$$SAC = Dc^A \sum_b \left(X_b C_b^R \right) \tag{10-8}$$

And therefore, if *SAC* can be estimated from historical records, we have

$$Dc^A = \frac{SAC}{\sum_b (X_b C_b^R)} \tag{10-9}$$

Similarly, if we can obtain an estimate of statewide annual user cost of the hazard, *SUC*, we can estimate:

$$Dc^U = \frac{SUC}{\sum_b (X_b C_b^D)} \tag{10-10}$$

An estimate of *SUC* can be computed by estimating the average annual number of bridge-days of closure, and then multiplying by the statewide average of C^D.

As a part of this study, a spreadsheet model was created in order to compute the Dc^A and Dc^U parameters by means of risk allocation. It works with a complete inventory of bridges, defined as all structures in Pontis with servtypon<=”9” and district<>”09”. From the historical research presented earlier in this report, the following inputs are entered:

- Estimates of hazard likelihood by bridge (for natural hazards), or computations of hazard probability (for the hazards where this is possible). In some cases (e.g. floods) the bridge-level estimates are used, while in other cases (e.g. tornadoes) the averages are smoothed by district. The rationale is explained in later sections of this chapter.
- Estimates of relative consequences by bridge and by element, typically on a scale of 1 to 5. In some cases the consequences are computed from available bridge data, while in other cases the scale is assigned, using judgment informed by the historical research, for categories of structures.
- Unit replacement cost per square foot of existing bridge deck. The analysis does not consider deck expansion since it is measuring existing risk, not measuring the benefits of higher capacity.
- Statewide annual average agency cost for damage recovery, based on historical records for the subject hazard, in millions of dollars.
- Statewide annual average number of bridge-days of closure due to the subject hazard, estimated from historical records.

The spreadsheet then allocates the total hazard costs among the bridges according to their risk index and magnitude, to estimate the Dc^A and Dc^U parameters. These parameters are then used in the PLAT to compute the life cycle costs and benefits of risk mitigation or replacement, using the equations given above. Table 10.1 summarizes the inputs and results of the analysis, and Figure 10.4 shows portions of the risk parameters tables.

The following sections describe the specific methods for computing the risk index and risk cost for the specific hazards addressed in the study.

Table 10.1. Risk allocation results

Hazard	Annual historical agency cost ($M)	Annual bridge days of disruption	Annual historical user cost ($k)	Total annual hazard cost ($M)	Agency cost factor $1/Dc^A$	User cost factor $1/Dc^U$
Hurricane	1,185	100	11,325	12,510	30,699.6	107.0
Tornado	200	5	566	766	1,981.7	18.2
Wildfire	500	20	2,265	2,765	20,659.0	120.3
Flood	427	112	12,684	13,111	685.2	0.8
Vessel collision	100	1	113	213	324.4	2.8
Over-height collision	500	10	1,132	1,632	76.6	0.5
Overload	300	3	340	640	2,690.3	25.4
Truck crashes	300	20	2,265	2,565	1,156.4	3.4
Advanced deterioration	283,000	3,000	339,741	622,741	3.1	0.1
Fatigue	997	52	5,891	6,888	*	*

* The risk allocation process was not used for fatigue

General risk analysis parameters

Parameter	Hurricane	Tornado	Wildfire	Flood	Vessel	Height	Weight	Crash	Decay	Fatigue
Typical year	1990	1990	1990	1990			1990	1990		
Typical factor	1	1	1	1			1	1		
Reference year	1955	1955	1955	1955			1955	1955		
Reference factor	2	2	2	2			1.5	1.2		
Total of risk index	7237.11	67.71	2808.26	110.95	1.02	3.25	35.02	32.36	162.56	46.56
Annual historical damage cost ($k)	1185	200	500	427	100	500	300	300	283000	
Total of risk index × replacement cost	36379038	396333	10329483	292561	32444	38294	807076	346931	870742	1057474
Checksum - total agency risk cost ($k)	1185	200	500	427	100	500	300	300	283000	997
Annual bridge-days of closure	100	5	20	112	1	10	3	20	3000	
Annual user cost of hazard ($k)	11325	566	2265	12684	113	1132	340	2265	339741	0
Total of risk index × user cost	1211376	10293	272470	9781	322	562	8643	7616	22207	7639
Checksum - total user risk cost ($k)	11325	566	2265	12684	113	1132	340	2265	339741	5891
Agency cost factor (1/(DcA))	30699.6	1981.7	20659.0	685.2	324.4	76.6	2690.3	1156.4	3.1	
User cost factor (1/(DcU))	107.0	18.2	120.3	0.8	2.8	0.5	25.4	3.4	0.1	

Tornado	Long bridge threshold (ft)	150
	High underclearance threshold (ft)	30
Flood	Avg span length cutoff (ft)	65.62
Vessel	Number of bridges over navigation	394
	Number of collisions per year	1.5
Fatigue	Default fatigue category	D
	Default fatigue stress range (ksi)	5
	Span multiplier - main unit	0.2
	Span multiplier - approach unit	0.2
	Fatigue crack repair, category D ($)	2000
	Non-trusses, E or E'	4400
	Trusses, E or E'	5200
	Indirect cost of fatigue repairs ($)	1500
	Duration of fatigue repair (days)	8

Bridge general vulnerability		Coastal hurricane vulnerability by Superstructure material - main unit (NBI 43A)									
		0	1	2	3	4	5	6	7	8	9
Superstructure design - main unit		Other	RC	RCC	St	StC	PC	PCC	Wood	Mas	Alum
0	Other										
1	Slab		5	3	1	1	5	3	5	1	1
2	Stringer/MultiBeam/Girder		5	3	3	2	5	3	5	1	1
3	Girder & Floorbeam		5	3	3	2	5	3	5	1	1
4	Tee Beam		5	3	3	2	5	3	5	1	1
5	Box Beam or Girders - Multiple		5	3	3	2	5	3	5	1	1
6	Box Beam/Girders - Single or Spread		5	3	3	2	5	3	5	1	1
7	Frame (except frame culverts)		1	1	1	1	1	1	1	1	1
8	Orthotropic		1	1	1	1	1	1	1	1	1
9	Truss - Deck		1	1	1	1	1	1	1	1	1
10	Truss - Thru		1	1	1	1	1	1	1	1	1
11	Arch - Deck		1	1	1	1	1	1	1	2	1
12	Arch - Thru		1	1	1	1	1	1	1	2	1
13	Suspension		1	1	1	2	1	1	1	1	1
14	Stayed Girder		1	1	1	2	1	1	1	1	1
15	Movable - Lift		1	1	4	3	1	1	1	1	1
16	Movable - Bascule		1	1	4	3	1	1	1	1	1
17	Movable - Swing		1	1	4	3	1	1	1	1	1
18	Tunnel		1	1	1	1	1	1	1	1	1
19	Culvert		4	1	4	1	4	1	4	4	4
20	Mixed Types		1	1	1	1	1	1	1	1	1
21	Segmental Box Girder		4	1	1	1	1	1	1	1	1
22	Channel Beam		4	1	1	1	1	1	1	1	1

Figure 10.4. Example of a few portions of the risk parameters tables

10.2. Hurricanes

The hurricane likelihood model is prepared from geographic data on historical hurricane occurrence in the five categories on the Saffir-Simpson scale. This data source is already geographically smoothed, and reflects the fact that wind speeds are typically much higher on the coast than inland. One of the primary hurricane damage processes, storm surge, is also a coastal phenomenon.

In each of the five wind categories, consequence estimates were prepared for each element on a 0-5 scale, with 5 being the most severe. Structure design types (NBI items 43A and 43B) were also given relative consequence scores. In addition, consequences were scaled according to bridge age and scour criticality. Together, these explanatory variables reflect several different damage processes and causes of vulnerability to winds and storm surge. The relative values were assigned judgmentally after extensive review of historical records, as discussed earlier in this report.
Following equation 10-3 but taking the availability of data into account, the risk index is computed as follows:

$$X = \sum_{s} (P_s^L V_s^E) V^G V^S V^A \tag{10-11}$$

Where Ps^L = Likelihood probability for hurricane category *s*
V_s^E = Element vulnerability score, averaged over all elements on the bridge, for category *s*
V^G = General vulnerability score based on structure design and material (NBI item 43)
V^S = Vulnerability score based on scour criticality (NBI item 113)
V^A = Vulnerability score based on age of the bridge (computed from NBI item 27)

If Ps^L is missing for a given bridge, it is provided from district or statewide averages. Specific probabilities were developed for 8413 of the 12139 bridges in the inventory.

All of the vulnerability scores are statewide parameters that are assigned to each category or element using expert judgment, informed by a study of historical damage reports. In most cases they are initially assessed on a scale of 0 to 5, then scaled so they have a value of 1.0 for a "typical" bridge in the inventory, by adding 1 and dividing by 3. The correct score is then automatically assigned to each bridge or element based on the characteristics found in the inventory.

Age-based vulnerability is a linear function of bridge age, computed as follows:

$$V^A = (y - Y) * m + b \tag{10-12}$$

Where *y* = Base year, when the age-based vulnerability score is set to 1
Y = Year the bridge was built
m = Slope of the line
b = Vulnerability factor in the base year, which is 1.0

The parameters of this line are set using expert judgment to reflect the improvement in risk-based design methods over time.

Detailed data on the likelihood and consequence parameters, and results for every bridge in the inventory, can be found in the spreadsheets delivered with this report.

	Hurricane and flood likelihood					Hurricane								
Bridge key	Cat 1	Cat 2	Cat 3	Cat 4	Cat 5	Scour vuln	Age vuln	Coastal bridge	Design vuln	Element vuln and likelihd	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	hurrp 1	hurrp 2	hurrp 3	hurrp 4	hurrp 5	Scour vulnH	Age vulnH	Coastal H	Des VulnH	Elem VulnH	Haz probH	Risk indexH	Agcy costH	User costH
150107	0.0488	0.0198	0.0050	0.0020	0.0010	0.400	1.886	TRUE	2.000	0.410	7.656	3.0563	25.36	1.58
100300	0.0488	0.0198	0.0100	0.0020	0.0010	1.200	1.429	TRUE	2.000	0.422	8.152	4.6332	24.38	0.56
150189	0.0488	0.0198	0.0100	0.0020	0.0010	0.400	1.114	TRUE	0.667	0.422	8.152	0.6110	17.41	15.18
720518	0.0488	0.0100	0.0050	0.0010	0.0010	1.200	1.029	TRUE	0.667	0.399	6.571	1.0967	16.88	10.63
720343	0.0488	0.0100	0.0050	0.0010	0.0010	0.400	1.571	TRUE	1.333	0.394	6.571	1.6631	16.20	1.32
720249	0.0488	0.0100	0.0050	0.0010	0.0010	0.400	1.571	TRUE	1.333	0.394	6.571	1.6630	16.20	1.32
870592	0.0952	0.0488	0.0198	0.0100	0.0020	0.400	1.029	TRUE	1.667	0.575	17.568	1.6712	13.62	0.54
480035	0.0952	0.0050	0.0198	0.0050	0.0010	0.400	1.857	TRUE	1.000	0.486	12.594	1.6456	13.41	32.47
720076	0.0488	0.0100	0.0050	0.0010	0.0010	1.200	2.057	TRUE	0.667	0.400	6.571	2.1945	12.86	12.52
150210	0.0488	0.0198	0.0050	0.0020	0.0010	0.400	0.971	TRUE	1.333	0.416	7.656	1.0568	10.17	0.55
900101	0.0952	0.0488	0.0198	0.0100	0.0050	0.400	1.229	TRUE	0.667	0.547	17.867	0.7758	8.96	16.61
490032	0.0488	0.0100	0.0050	0.0010	0.0010	0.400	1.057	TRUE	2.000	0.398	6.571	1.6862	8.57	13.80
900020	0.0952	0.0488	0.0198	0.0100	0.0050	0.400	3.229	TRUE	1.333	0.549	17.867	4.0848	8.56	0.09
010106	0.0488	0.0198	0.0100	0.0050	0.0010	0.400	0.629	TRUE	2.000	0.427	8.451	1.0400	8.22	3.62
880077	0.0952	0.0050	0.0198	0.0050	0.0010	0.400	1.314	TRUE	2.000	0.479	12.594	2.3098	7.77	2.09
154259	0.0488	0.0198	0.0050	0.0020	0.0010	0.400	0.914	TRUE	2.000	0.416	7.656	1.4917	7.47	0.62
154260	0.0488	0.0198	0.0050	0.0020	0.0010	0.400	0.914	TRUE	2.000	0.415	7.656	1.4897	7.46	0.65
900016	0.0952	0.0488	0.0198	0.0100	0.0050	0.400	1.514	TRUE	2.000	0.553	17.867	2.8865	6.90	0.04
900045	0.0952	0.0488	0.0198	0.0100	0.0050	0.400	1.514	TRUE	2.000	0.549	17.867	2.8756	6.87	0.03
139003	0.0000	0.0000	0.0000	0.0000	0.0000	0.400	1.686	TRUE	2.000	0.397	6.123	2.6876	6.73	3.13

Figure 10.5. Top 20 bridges vulnerable to hurricanes (sorted by agency risk cost)

10.3. Tornadoes

Tornado likelihood was developed from historical records of tornado paths. A bridge is associated with a particular tornado if the hazard passed within one mile of it. Even though tornadoes are common in Florida and can happen anywhere in the state, they are not common enough to produce a smooth probability distribution. Therefore, the historical incidence of tornadoes was accumulated by district in order to produce a more reasonable forecast of future tornado probability for any particular bridge.

Consequences of tornadoes were graded on a 0-5 scale by structure design type (NBI 43B) and by range of underclearance. Bridges with higher underclearances have been found to have an increased incidence of severe tornado damage in the historical record. Since design methods for wind loads have improved over the past few decades, age of each bridge was also used as a consideration in the risk allocation.

10.4. Wildfires

Wildfires were modeled in a manner quite similar to tornadoes. While they are very common across Florida, it was still necessary to smooth the historical data in order to have consistent predictions of future event likelihood.

Bridge key	Tornado Element vuln	Age vuln	Under clear (m)	Design vuln	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Elem VulnT	Age vulnT	UClrT	Des vulnT	Haz probT	Risk indexT	Agcy costT	User costT
150189	0.994	1.114	53.340	1.667	1.045	0.0193	8.52	2.82
870592	1.174	1.029	9.388	1.667	1.488	0.0299	3.78	0.06
900101	0.990	1.229	19.812	1.000	1.488	0.0181	3.24	2.28
720518	1.032	1.029	53.340	1.667	0.762	0.0135	3.22	0.77
720076	1.117	2.057	46.330	2.000	0.762	0.0350	3.18	1.18
150107	0.970	1.886	13.076	1.000	1.045	0.0191	2.46	0.06
720343	1.024	1.571	19.812	1.000	0.762	0.0123	1.85	0.06
720249	1.012	1.571	19.812	1.000	0.762	0.0121	1.83	0.06
480035	1.013	1.857	15.240	1.000	0.654	0.0123	1.55	1.43
150210	1.018	0.971	14.874	1.000	1.045	0.0103	1.54	0.03
720107	1.131	1.657	42.885	2.000	0.762	0.0286	1.52	0.43
870453	0.973	1.657	22.860	1.000	1.488	0.0240	1.24	1.29
100300	1.017	1.429	13.381	1.000	1.045	0.0152	1.24	0.01
870356	0.930	1.657	22.860	1.000	1.488	0.0229	1.04	0.29
470029	1.101	2.486	15.850	2.000	0.654	0.0358	1.04	0.02
870301	1.000	1.886	16.673	1.000	1.488	0.0281	0.93	2.67
860478	0.909	1.086	5.060	0.667	1.310	0.0086	0.92	0.22
880077	0.984	1.314	19.812	1.000	1.310	0.0169	0.88	0.09
900020	0.824	3.229	6.096	0.667	1.488	0.0264	0.86	0.00
870077	1.000	1.943	10.668	1.000	1.488	0.0289	0.82	0.93

Figure 10.6. Top 20 bridges vulnerable to tornadoes

Bridge key	Wildfire Element vuln	Age vuln	Design vuln	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Elem VulnW	Age vulnW	Des vulnW	Haz probW	Risk indexW	Agcy costW	User costW
720343	1.095	1.571	1.667	20.263	0.5813	8.41	0.41
720249	1.062	1.571	1.667	20.263	0.5635	8.15	0.40
480035	1.067	1.857	1.667	17.625	0.5819	7.05	10.21
720076	1.126	2.057	1.667	20.263	0.7824	6.81	3.97
720114	1.035	1.657	1.667	20.263	0.5793	3.26	0.07
720107	1.095	1.657	1.667	20.263	0.6130	3.13	1.38
150107	0.970	1.886	1.333	9.243	0.2254	2.78	0.10
760043	1.139	1.371	1.667	20.263	0.5275	2.46	17.51
570091	1.039	0.914	1.333	17.625	0.2233	2.29	1.12
870592	1.188	1.029	1.667	9.268	0.1887	2.29	0.05
900101	0.843	1.229	1.333	9.268	0.1280	2.20	2.44
180940	0.972	1.686	1.333	15.700	0.3431	2.11	0.81
470029	1.014	2.486	1.667	17.625	0.7408	2.06	0.05
120002	1.107	1.800	1.667	15.308	0.5082	1.95	0.84
780056	1.107	1.829	1.667	20.263	0.6834	1.94	2.65
720153	1.098	1.886	1.667	20.263	0.6993	1.84	2.28
490100	0.967	0.629	1.333	17.625	0.1428	1.81	0.73
490032	0.961	1.057	1.333	17.625	0.2387	1.80	1.74
580174	1.020	0.743	1.333	17.625	0.1780	1.76	0.14
720518	1.097	1.029	0.333	20.263	0.0762	1.74	0.66

Figure 10.7. Top 20 bridges vulnerable to wildfire

10.5. Floods and scour

Unlike many other states, most occurrences of scour in Florida are associated with hurricanes or flooding, and rarely occur independently of those extreme events. As a result, both categories of risk were combined and modeled as flooding events in this analysis. Floods are localized phenomena that occur in areas of low elevation, so the location of a bridge relative to a flood plain was the most significant factor in assessing hazard likelihood.

For a flood to damage a bridge by scour, it is necessary for a substructure unit to be in the water, at least during flood events. A bridge may also be damaged by changes in the river channel, which may erode the river bank and wash out the approach or expose the abutment foundation. Bridges may also be damaged by hydraulic pressure on the structure, or by impact of water-borne debris with the structure. Several available data items therefore contribute to the consequence assessment: superstructure design type, average span length, channel condition (NBI item 61), scour criticality (NBI item 113), and bridge age.

	Flood										
Bridge key	Scour vuln	Element vuln	Age vuln	Bridge over water	Avg span (m)	Design vuln	Channel vuln	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Scour vulnF	Elem VulnF	Age vulnF	Over waterF	Avg spanF	Des vulnF	Chan vulnF	Haz probF	Risk indexF	Agcy costF	User costF
150189	1.200	1.172	1.114	TRUE	37.673	1.000	0.400	2.000	0.0125	16.01	43.22
480045	2.000	1.263	1.943	TRUE	13.337	1.667	1.600	2.995	0.3920	15.39	66.66
720076	2.000	1.216	2.057	TRUE	34.585	1.000	0.800	1.139	0.0456	11.97	36.09
100332	1.200	1.236	1.429	TRUE	23.585	1.000	0.800	2.995	0.0508	8.36	5.41
720518	2.000	1.269	1.029	TRUE	33.453	1.000	0.400	1.139	0.0119	8.20	15.98
150107	1.200	1.273	1.886	TRUE	15.071	1.667	0.400	0.995	0.0191	7.10	1.37
100300	2.000	1.333	1.429	TRUE	15.302	1.667	0.400	0.995	0.0253	5.96	0.43
010050	1.200	1.211	1.400	TRUE	21.388	1.000	0.800	2.995	0.0487	5.64	1.38
870592	1.200	1.396	1.029	TRUE	269.786	1.000	0.800	0.995	0.0137	5.01	0.62
720343	1.200	1.310	1.571	TRUE	23.108	1.000	0.400	1.139	0.0112	4.91	1.24
720249	1.200	1.309	1.571	TRUE	23.108	1.000	0.400	1.139	0.0112	4.91	1.24
120028	2.000	1.298	1.714	TRUE	16.561	1.667	0.800	2.995	0.1777	4.23	37.70
580951	2.000	1.247	1.857	TRUE	18.207	1.667	1.600	0.995	0.1229	4.16	55.11
570091	1.200	1.373	0.914	TRUE	41.645	1.000	0.400	1.995	0.0120	3.72	9.40
900101	1.200	1.078	1.229	TRUE	41.100	1.000	0.400	0.995	0.0063	3.28	18.79
180940	1.200	1.250	1.686	TRUE	12.773	1.667	0.400	1.007	0.0170	3.15	6.28
150210	1.200	1.351	0.971	TRUE	43.659	1.000	0.400	0.995	0.0063	2.70	0.45
100333	1.200	1.205	1.429	TRUE	22.714	1.000	0.800	0.995	0.0164	2.67	8.12
700184	1.200	1.200	1.057	TRUE	44.227	1.000	0.800	2.000	0.0244	2.41	15.00
480035	0.400	1.307	1.857	TRUE	18.549	1.667	0.400	0.995	0.0064	2.35	17.62

Figure 10.8. Top 20 bridges vulnerable to floods and scour

10.6. Vessel collisions

Although vessel collision is a very real concern in Florida, there is minimal data available about vessel traffic under bridges. As a result, it was necessary to assign an equal risk likelihood to all bridges having navigable waterways and requiring pier protection. The pier protection data item (NBI item 111) was used to distinguish consequences. Newer bridges are designed for vessel impact, while older bridges are not, especially before 1985.

Vessel collision							
Bridge key	Element vuln	Age vuln	Design vuln	Hazard prob (%)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Elem VulnV	Age vulnV	Des vulnV	Haz probV	Risk indexV	Agcy costV	User costV
100100	0.684	3.400	2.000	0.381	0.0177	0.32	0.71
874161	0.730	2.900	1.667	0.381	0.0134	0.23	0.33
470029	0.681	2.567	2.000	0.381	0.0133	2.13	0.03
150036	0.667	2.133	2.000	0.381	0.0108	0.14	2.49
864028	0.762	2.900	1.000	0.381	0.0084	0.03	0.09
874459	0.750	2.933	1.000	0.381	0.0084	0.50	0.35
100218	0.667	1.600	2.000	0.381	0.0081	0.09	0.14
105503	0.705	2.967	1.000	0.381	0.0080	0.13	0.03
860001	0.741	2.800	1.000	0.381	0.0079	0.04	0.41
874135	0.728	2.833	1.000	0.381	0.0079	0.02	0.08
890003	0.716	1.700	1.667	0.381	0.0077	0.25	0.01
930072	0.738	2.667	1.000	0.381	0.0075	0.04	3.23
900003	0.684	1.400	2.000	0.381	0.0073	0.52	0.02
900016	0.667	1.433	2.000	0.381	0.0073	1.48	0.00
900045	0.667	1.433	2.000	0.381	0.0073	1.48	0.00
874134	0.818	2.200	1.000	0.381	0.0069	0.01	0.03
120001	0.727	2.467	1.000	0.381	0.0068	0.02	0.09
930157	0.691	2.567	1.000	0.381	0.0068	0.61	0.17
480001	0.778	2.200	1.000	0.381	0.0065	0.10	0.29
900078	0.690	1.233	2.000	0.381	0.0065	0.06	0.03

Figure 10.9. Top 20 bridges vulnerable to vessel collisions

10.7. Overloads

The hazard probability for truck overloads was assumed to be proportional to the fraction of trucks detoured due to operating rating limits. This fraction can be computed using a truck weight model developed in an earlier study with Florida weigh-in-motion data (Sobanjo and Thompson, 2004). The percent of trucks detoured is:

For bridges carrying interstate highways:

OR<10000	100
OR<80000	102.24 - (8.982E-5)*OR - (1.4336E-8)*OR^2
OR<91000	18.976 - (2.083E-4)*OR
OR higher	0

For all other functional classes:

OR<3725	100
OR<85000	107.26 - (1.9743E-3)*OR + (6.5265E-9)*OR^2 + (2.2256E-14)*OR^3
OR higher	0

Where OR is the operating rating of the bridge, in pounds (NBI item 66).

This percentage is multiplied by average daily traffic and truck percent to yield an estimate of the number of trucks unable to use the bridge. It is assumed that some unknown fraction of these trucks will fail to detour, and an unknown fraction of those will overload the bridge, causing its damage or destruction. No effort is necessary to determine these fractions, since the calibration process will include an appropriate scale in the *D* parameter.

The consequence model for overloads considers the superstructure condition (NBI item 59) and bridge age as influential variables.

Overheight truck								
Bridge key	Fraction trucks detour	Truck fraction of ADT	ADT under bridge	Element vuln	Hazard prob (relatve)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Detr fracC	Truck fracC	ADT undC	Elem VulnC	Haz probC	Risk indexC	Agcy costC	User costC
874635	0.000	0.000	30488	1.133	36.166	0.4099	295.50	32.47
870099	0.039	0.090	36477	0.970	71.665	0.6949	29.68	49.82
870453	0.000	0.030	11513	0.973	1.552	0.0151	20.24	29.84
870237	0.000	0.020	8416	1.053	16.525	0.1739	19.95	49.98
930157	0.006	0.030	15308	0.776	4.806	0.0373	15.89	6.89
870472	0.031	0.050	3045	1.156	4.760	0.0550	13.15	211.10
870455	0.000	0.010	1781	1.133	3.068	0.0348	11.17	104.75
870332	0.000	0.050	10856	1.167	4.712	0.0550	7.18	78.14
870434	0.000	0.050	12446	1.091	4.116	0.0449	4.41	21.06
870366	0.000	0.110	3124	1.156	1.301	0.0150	4.22	2.75
870352	0.001	0.100	14935	1.019	1.760	0.0179	4.13	10.97
860237	0.000	0.060	245393	1.028	3.344	0.0344	3.28	44.72
550941	0.020	0.060	12004	1.056	12.250	0.1293	2.96	20.07
860341	0.000	0.040	160279	1.111	1.456	0.0162	2.89	6.40
870456	0.000	0.400	6873	1.137	1.330	0.0151	2.86	3.24
870370	0.000	0.060	99565	1.137	3.844	0.0437	2.62	34.15
109907	0.013	0.050	12684	1.216	8.348	0.1015	2.59	15.99
130103	0.000	0.010	107	0.979	0.296	0.0029	2.18	0.27
109908	0.010	0.050	12684	1.216	6.637	0.0807	2.09	12.72
870474	0.000	0.010	1060	1.233	0.607	0.0075	2.07	6.65

Figure 10.10. Top 20 bridges vulnerable to overloads

10.8. Over-height collisions

The hazard probability for over-height truck collisions was assumed to be proportional to the fraction of trucks detoured due to vertical clearance restrictions. This fraction can be computed using a truck height model developed in an earlier Florida study using laser truck counting equipment (Sobanjo and Thompson, 2004). The percent of trucks detoured is:

For bridges carrying interstate highways:

VC<9.65	100
VC<13	855.91 - 223.43*VC + 22.199*VC^2 - 0.74236*VC^3
VC<14	(1.0956E+56)*VC^(-48.683)
VC<16.1	14.567 - 0.9046*VC
VC higher	0

For all other functional classes:

VC<7.3	100
VC<13.5	-26.275 + 34.692*VC - 2.3894*VC^2
VC<14	138.86 - 9.886*VC
VC higher	0

Where VC is the vertical clearance over the roadway, in feet (NBI item 10). This can apply to roadways under a bridge or on it, if vertical clearance is restricted.

This percentage is multiplied by average daily traffic and truck percent to yield an estimate of the number of trucks unable to use the bridge. It is assumed that some unknown fraction of these trucks will fail to detour, and an unknown fraction of those will strike the bridge, causing its damage or destruction. No effort is necessary to determine these fractions, since the calibration process will include an appropriate scale in the *D* parameter.

The consequence model for over-height truck collisions considers only the types of elements present on the bridge.

Overweight truck								
Bridge key	Fraction trucks detour	Design vuln	Element vuln	Age vuln	Hazard prob (relatve)	Risk index	Agency cost ($k)	User cost ($k)
brkey	Detr fracL	Des vulnL	Elem VulnL	Age vulnL	Haz probL	Risk indexL	Agcy costL	User costL
150189	0.021	0.667	1.394	1.057	59.570	0.5854	190.37	61.17
480163	0.058	0.667	1.400	1.143	54.219	0.5783	18.34	0.51
720163	0.009	0.667	1.463	1.500	49.171	0.7194	14.02	11.12
480164	0.026	0.667	1.396	1.143	24.376	0.2592	8.63	0.82
720249	0.001	0.667	1.432	1.286	4.617	0.0567	6.30	0.19
720343	0.001	0.667	1.417	1.286	4.617	0.0561	6.23	0.19
874459	0.023	1.000	1.274	1.900	32.001	0.7745	6.22	4.65
720076	0.001	1.000	1.423	1.529	2.642	0.0575	3.84	1.38
724214	0.022	1.333	1.410	1.471	33.863	0.9369	2.82	1.47
720107	0.002	1.000	1.429	1.329	2.994	0.0568	2.23	0.60
750244	0.009	0.667	1.389	1.243	56.811	0.6538	1.84	4.41
870975	0.002	0.667	1.394	0.771	28.753	0.2061	1.56	0.84
460019	0.004	1.000	1.333	1.357	4.278	0.0774	1.36	6.33
930094	0.018	1.000	1.306	1.571	20.551	0.4216	1.22	2.50
720164	0.013	0.667	1.436	1.443	33.472	0.4623	0.90	4.13
750242	0.004	0.667	1.389	1.243	28.069	0.3230	0.89	2.12
724312	0.061	1.333	1.481	1.414	10.525	0.2940	0.76	0.00
724304	0.024	1.333	1.333	1.414	4.085	0.1027	0.72	0.01
794003	0.005	1.333	1.295	1.514	5.341	0.1396	0.70	0.40
930428	0.005	0.667	1.394	0.971	20.502	0.1851	0.70	2.81

Figure 10.11. Top 20 bridges vulnerable to over-height collisions

10.9. Other truck collisions

A bridge can be damaged by other types of truck collisions on or under the bridge. This is especially a problem with fuel tanker trucks, which can ignite and damage the bridge by fire. The hazard probability for truck collisions is assumed to be proportional to the truck accident risk computed by the FDOT accident risk model (Thompson et al., 1999). This model, developed using a regression analysis of bridge characteristics and crash statistics, computes accident risk as follows:

Expected truck accidents per year = Truck percent * (Term1 + Term2 + Term3)/1000, where:

Term1 = 886.0098 for urban arterials (functional class 14 or 16), or -377.3701 otherwise
Term2 = 0.7323*lanes*length
Term3 = coef3*lanes/roadwidth*adt

Where “length” is the structure length in meters (NBI item 49) and “lanes” is the number of lanes (NBI item 28). For roadways under the bridge, “length” is the bridge deck width (NBI item 52) in meters. “Roadwidth” is the traveled way width in meters (NBI item 51) and “adt” is the average daily traffic forecast for the year being analyzed. The coefficient on term3 takes the following values based on approach alignment and deck condition:

If approach <= “6” and deck <=”6”	0.7899
If approach <= “6” and deck >”6”	0.5031
If approach > “6” and deck <=”6”	0.4531
If approach > “6” and deck >”6”	0.3904

Note that this model was developed for metric data.

For the consequence model, bridge age and structure type (NBI 43A and 43B) are considered significant, since newer bridges have more fire protection (such as standoff barriers), and certain structure types and materials are more resistant to fires.

	Truck crash																
Bridge key	Func class	Lanes	Road length (m)	Road width (m)	ADT	Truck fraction of ADT	Accidnt model term 1	Accidnt model term 2	Accidnt model term 3	Predict acci-dents	Design vuln	Element vuln	Age vuln	Hazard prob (relatve)	Risk index	Agency cost ($k)	User cost ($k)
brkey	FCX	LanesX	LenX	Road widthX	ADTX	Truck fracX	Term1 X	Term2 X	Term3 X	Acc cntX	Des vulnX	Elem VulnX	Age vulnX	Haz probX	Risk indexX	Agcy costX	User costX
860478	11	8	1772.1	41.5	108879	0.27	-377.4	10381.5	8197.4	4.914	1.333	1.152	1.017	4.915	0.0768	13.99	10.56
720249	11	4	4968.3	20.7	67151	0.11	-377.4	14553.1	5872.1	2.205	1.667	1.074	1.114	2.205	0.0440	11.37	1.11
720343	11	4	4968.3	20.7	67151	0.11	-377.4	14553.1	5872.1	2.205	1.667	1.071	1.114	2.205	0.0439	11.35	1.11
900101	2	2	10932.5	11.0	13928	0.09	-377.4	16011.7	991.3	1.496	1.333	1.078	1.046	1.496	0.0225	6.90	15.32
100806	12	3	5181.9	17.0	24318	0.08	-377.4	11384.1	1671.6	1.014	1.333	1.022	0.909	1.905	0.0236	5.36	1.47
010106	11	6	2451.6	34.1	58032	0.13	-377.4	10771.7	3981.9	1.869	1.333	0.967	0.926	1.869	0.0223	4.68	2.47
150189	11	4	6668.1	24.4	55821	0.07	-377.4	19532.2	3574.9	1.591	0.333	1.139	1.023	1.591	0.0062	4.67	4.88
150107	11	4	4837.8	17.8	76824	0.06	-377.4	14170.9	7822.2	1.297	1.333	1.030	1.177	1.297	0.0210	4.62	0.34
480213	11	3	4212.3	17.1	27634	0.15	-377.4	9254.1	1896.1	1.616	1.333	1.074	0.909	1.616	0.0210	3.83	0.18
480214	11	3	4186.8	17.1	27634	0.15	-377.4	9198.1	1896.1	1.608	1.333	1.071	0.903	1.608	0.0207	3.75	2.89
150210	11	4	4846.2	21.0	76824	0.06	-377.4	14195.5	5712.5	1.172	1.333	0.930	0.994	1.172	0.0144	3.69	0.24
480035	14	4	4767.1	17.4	55341	0.03	886.0	13963.7	5772.9	0.619	1.667	1.093	1.171	0.619	0.0132	2.86	8.29
100333	12	2	2998.3	12.1	18239	0.07	-377.4	4391.3	1369.3	0.377	1.667	1.099	1.086	1.379	0.0274	2.64	3.11
490100	7	2	6565.9	13.4	3758	0.10	-377.4	9616.5	218.8	0.946	1.333	1.000	0.926	0.946	0.0117	2.64	2.15
870373	11	2	1223.5	11.6	68810	0.03	-377.4	1791.9	5383.8	0.204	1.333	1.204	1.114	3.507	0.0627	2.34	2.70
100332	12	2	3042.5	11.6	18239	0.07	-377.4	4456.0	1427.0	0.385	1.667	1.122	1.086	1.113	0.0226	2.21	0.55
180940	1	4	1366.7	34.5	44871	0.20	-377.4	4003.3	2030.8	1.131	1.333	1.139	1.137	1.131	0.0195	2.15	1.66
750114	12	6	1028.9	16.2	60796	0.13	-377.4	4520.8	10193.0	1.864	1.333	1.120	1.097	3.033	0.0497	2.13	2.91
870575	12	2	1206.7	11.6	68810	0.03	-377.4	1767.3	5383.8	0.203	1.333	1.175	1.114	3.164	0.0553	2.01	1.94
750064	11	11	456.3	45.7	174649	0.06	-377.4	3675.6	19039.1	1.340	1.333	1.154	1.166	1.471	0.0264	1.87	2.11

Figure 10.12. Top 20 bridges vulnerable to other truck collisions

10.10. Advanced deterioration

The hazard likelihood for advanced deterioration is estimated from forecast condition, using the following formula:

$$P^L = \frac{a\varphi\left(\frac{ln(D)-\mu}{\sigma}\right)}{\sigma D\left(1-\Phi\left(\frac{\ln(D)-\mu}{\sigma}\right)\right)} + bD + c \qquad (10\text{-}21)$$

where:
- D = decay index as computed below
- $ln(D)$ = natural logarithm of the decay index
- μ = mean of $ln(D)$ (Table 10.2)
- σ = standard deviation of $ln(D)$ (Table 10.2)
- $\varphi((ln(D)-\mu)/\sigma)$ = probability density function of the normal distribution
  = NORMDIST(ln(D),μ,σ,FALSE) in Excel
- $\Phi((ln(D)-\mu)/\sigma)$ = cumulative distribution function of the normal distribution
  = NORMDIST(ln(D),μ,σ,TRUE) in Excel
- a,b,c = regression coefficients (Table 10.2)

Table 10.2. Coefficients and distribution parameters for the hazard model

Material	Coefficients				Distribution	
	a	b	c		μ	σ
Concrete – prestressed	3000	0.00199	0		15.461	3.982
Concrete – reinforced	3000	0.00047	0		15.385	3.928
Steel	3000	0.00196	0		15.545	3.998
Timber	3000	0.00539	0		15.077	3.902

The regression coefficients and statistical distribution parameters in the model vary by NBI item 43A, main unit structure material type.

The decay index is similar to the health index (Shepard and Johnson, 2001), but focuses on the worst two condition states of each element. The rationale and development process for it are described in the earlier chapter on advanced deterioration. It is computed from current or forecast element-level condition as follows:

$$D = 100 \sum_{c} \frac{W_c}{TEV_c} \left(\sum_{s} w_{cs} CEV_{cs} \right) \tag{10-22}$$

$$CEV_{cs} = \sum_{e \in c} (P_{es} Q_e C_e)$$

$$TEV_c = \sum_{e \in c} (Q_e C_e)$$

Where P_{es} = Fraction of element *e* observed or forecast to be in condition state *s*
Q_e = Quantity of element *e* on the bridge
C_e = Unit replacement cost of element *e*
W_c = Relative weight (importance) of component c (Table 10.3)
w_{cs} = Relative weight (importance) of condition state *s* of component *c*

This equation is organized into three components, deck, superstructure, and substructure. The three components are combined as a weighted average, using W_c as the weight. The equation considers two condition states for each element. The worst condition state is always given full weight, w_{c1}=1.0. The second-worst state is tabulated in Table 10.2. The coefficients vary by NBI main unit material (item 43A).

The consequence measure for advanced deterioration is structure replacement, rehabilitation, or posting. From the historical record, approximately 55 bridges per year are replaced due to advanced deterioration (statewide, including local bridges); 39 are rehabilitated, and 13 are posted. Using average agency and user costs as described earlier, this represents a total agency cost of $283 million and a total user cost of $331 million. These results are used in the risk allocation process to determine the appropriate scaling constants for the model.

Table 10.3. Coefficients for the decay index

Material	Component weights W_c				2[nd]-worst state w_{cs}		
	Deck	Super	Substr		Deck	Super	Substr
Concrete – prestressed	20%	40%	40%		50%	50%	50%
Concrete – reinforced	20%	40%	40%		50%	50%	50%
Steel	20%	40%	40%		50%	50%	50%
Timber	40%	40%	20%		10%	50%	50%

Bridge key	Advanced deterioration: Coef B	Mean of distrib	StDev of distrib	Decay index	Hazard prob	Risk index	Agency cost ($k)	User cost ($k)
brkey	CoefBD	MuD	SigmaD	Decay indexD	Haz probD	Risk indexD	Agcy costD	User costD
150189	0.002	15.461	3.982	0.424	4.091	0.0409	11631.23	1663.49
720518	0.002	15.461	3.982	0.188	3.868	0.0387	5939.20	613.51
900101	0.002	15.461	3.982	3.704	4.421	0.0442	5095.33	1549.07
720249	0.002	15.545	3.998	0.861	4.080	0.0408	3965.06	52.98
720343	0.002	15.545	3.998	0.595	4.037	0.0404	3923.11	52.42
150107	0.002	15.461	3.982	4.453	4.495	0.0449	3721.28	37.93
480035	0.002	15.545	3.998	0.490	4.008	0.0401	3258.30	1294.09
150210	0.002	15.461	3.982	0.063	3.387	0.0339	3250.72	28.72
720076	0.002	15.545	3.998	12.270	5.400	0.0540	3157.21	504.28
010106	0.002	15.461	3.982	0.136	3.747	0.0375	2955.16	213.18
870592	0.002	15.545	3.998	0.047	3.169	0.0317	2578.09	16.91
900020	0.002	15.545	3.998	49.689	11.928	0.1193	2493.99	4.44
570091	0.002	15.461	3.982	0.100	3.617	0.0362	2493.85	333.62
100300	0.002	15.461	3.982	0.945	4.208	0.0421	2209.58	8.38
154259	0.002	15.461	3.982	0.859	4.198	0.0420	2097.75	28.38
100585	0.002	15.461	3.982	0.825	4.193	0.0419	2088.61	8.35
900094	0.002	15.461	3.982	2.064	4.290	0.0429	1682.78	1431.63
460113	0.002	15.461	3.982	0.129	3.726	0.0373	1647.10	25.93
154260	0.002	15.461	3.982	0.050	3.265	0.0327	1631.61	23.32
124096	0.002	15.461	3.982	0.058	3.345	0.0334	1626.92	373.93

Figure 10.13. Top 20 bridges vulnerable to advanced deterioration

10.11. Fatigue

The fatigue model, like the advanced deterioration model, directly estimates the probability of damage to a bridge. As a result, it is not necessary to use a risk allocation process. The earlier chapter on fatigue provides a method for computing likelihood, consequence, and impact. This is included in the risk analysis if any of the following are true:

- Main unit material (NBI 43A) is 3 or 4 (steel superstructure) and design type (NBI 43b) is between 2 and 17 inclusive;
- Approach unit material (NBI 44A) is 3 or 4 (steel superstructure) and design type (NBI 44b) is between 2 and 17 inclusive;

- Fracture critical inspections are required on the bridge (NBI 92AA)

The bridge is excluded from the analysis if it was built during or after the year 1980, or if key traffic and truck data are missing or zero.

The social cost of the risk is computed from:

$$C^{risk} = P^L C^{FR} + P^L C^{FD} \tag{10-23}$$

Where P^L = Probability of fatigue cracking
C^{FR} = Repair cost when a crack is found
C^{FD} = User cost when a crack is found

The probability of cracking is estimated using an adaptation of the AASHTO fatigue life model, using a lognormal distribution with respect to age of the bridge. It is computed as:

$$P^L = LOGNORMDIST\left(\frac{A}{A_M}, 0, ln\left(\frac{A_M}{A_E}\right)\right) \tag{10-24}$$

$$P^L = 0 \; if \; \frac{R_\gamma^M K_\gamma (g-1)}{T_1 C S^3} + 1 \leq 0$$

$$A_M = \frac{R_\gamma^M K_\gamma}{C T_1 S^3} \; if \; g = 1$$

$$A_M = \frac{\log\left(\frac{R_\gamma^M K_\gamma (g-1)}{T_1 C S^3} + 1\right)}{\log(g)} \; for\; any\; other\; g$$

$$A_E = \frac{R_\gamma^E K_\gamma}{C T_1 S^3} \; if \; g = 1$$

$$A_E = \frac{\log\left(\frac{R_\gamma^E K_\gamma (g-1)}{T_1 C S^3} + 1\right)}{\log(g)} \; for\; any\; other\; g$$

Where A = Age of the bridge in the year being analyzed
A_M = Mean age at the onset of cracking (years)
A_E = Evaluation fatigue life (one standard deviation sooner than A_M)
R_γ^M = Resistance factor (unitless) for fatigue category γ and mean life from Table 10.4
R_γ^E = Resistance factor (unitless) for fatigue category γ and evaluation life from Table 10.4

K_γ = Empirical fatigue constant (ksi^3) for fatigue category γ from Table 10.4
C = Number of stress range cycles per truck passage
1.0 if maximum span length (bridge.maxspan)>12m
2.0 otherwise
S = Effective stress range (ksi) at the critical detail
T_1 = Single-lane truck volume in the first year of the bridge's life (explained below)
g = Traffic growth factor (explained below)

For this analysis, the appropriate fatigue detail category and stress range for the critical detail can be specified individually for each bridge. By default, the model assumes category D with a conservative stress range of 5 ksi.

Table 10.4. Fatigue model parameters

Fatigue Category	R_γ^M	R_γ^E	K_γ (ksi^3)
A	2.8	1.7	2.50 E+10
B	2.0	1.4	1.20 E+10
B'	2.4	1.5	6.10 E+09
C	1.3	1.2	4.39 E+09
C'	1.3	1.2	4.39 E+09
D	1.6	1.3	2.20 E+09
E	1.6	1.3	1.10 E+09
E'	2.5	1.6	3.90 E+08

The growth factor *g* can be determined from Pontis data, as follows:

$$g = \left(\frac{adtfuture}{adttotal}\right)^{\frac{1}{adtfutyear-adtyear}} \tag{10-25}$$

where all of the symbols are fields in the Pontis roadway table. If adttotal is zero or missing, the bridge should not be included in the fatigue analysis. If any of the other variables are zero or missing, a growth factor of *g*=1.0 can be assumed.

The first-year truck volume is determined from the latest traffic count and the growth factor, and adjusted to estimate the maximum single-lane volume:

$$T_1 = 365 \times \frac{adttotal}{g^{(adtyear-yearbuilt)}} \times \frac{truckpct}{100} \times SLF \tag{10-26}$$

where adttotal, adtyear, and truckpct are in the Pontis roadway table, and yearbuilt is in the Pontis bridge table. If truckpct is missing, the AASHTO LRFD spec provides the following default values:

Functional class 01 (rural interstate)	20
Functional classes 02-11 (all other rural, and urban interstate)	15
Functional classes 12-19 (all other urban)	10

If any other fields are missing, the bridge should be excluded from the analysis. SLF is the single-lane factor, as follows:

If trafficdir=3 or lanes=1	1.000
If trafficdir=1 and lanes=2	0.850
If trafficdir=1 and lanes>2	0.800
If trafficdir=2 and lanes=2 or 3	0.500
If trafficdir=2 and lanes=4 or 5	0.425
If trafficdir=2 and lanes>5	0.400

where trafficdir and lanes are in the Pontis roadway table. If either of these database fields are missing or zero, the bridge should be excluded from the analysis.

Agency consequences of fatigue are estimated from typical crack repair costs. Total agency cost (dollars) of repair of all fatigue cracks on a bridge in a year, given that at least one crack has been observed, is computed from this formula:

$$C^{FR} = \sum_u M_u N_u \left(F_{\gamma u} U_{\gamma u} + I \right) \tag{10-27}$$

where: u = Bridge unit (main or approach)
M_u = Span multiplier, cracks per span in a typical fatigue repair, default value = 0.2
N_u = Number of spans in unit u
$F_{\gamma u}$ = Cost impact factor (unitless) for category γ (see below)
$U_{\gamma u}$ = Agency cost, in dollars, of repairing one typical crack (see below)
I = Indirect cost, dollars per crack (traffic control, access, and mobilization) = \$1500

The research described earlier in this report found an appropriate costing methodology in NCHRP Report 495 (Fu et al., 2003) and updated the metrics. The recommended cost impact factors are as follows:

For trusses with fatigue detail category D:

Up to 120 feet in span	0.75
Up to 200 feet in span	1.00
Greater spans	1.20

For non-trusses with fatigue detail category D:

Up to 65 feet in span	0.51
Up to 100 feet in span	1.00
Up to 120 feet in span	1.33
Up to 140 feet in span	1.70
Up to 160 feet in span	2.16
Greater spans	2.70

For trusses or non-trusses with fatigue detail category E or E’:

Up to 100 feet in span	0.75
Up to 120 feet in span	1.00
Up to 140 feet in span	1.28
Up to 160 feet in span	1.63
Greater spans	2.03

The unit costs recommended for 2013 dollars are as follows:

Trusses or non-trusses, category D	\$2000
Non-trusses, category E or E’	4400
Trusses, category E or E’	5200

The user impact of fatigue cracking is estimated to entail the detour of all trucks for an average of 8 days, based on FDOT’s experience. Thus the formula is:

$$C^{FD} = C^{D} T \times 8\,days \tag{10-28}$$

where: C^{D} = Maximum daily detour cost, from equation 10-6
T = Truck percent in the traffic stream

	Fatigue																
Bridge key	Has fatigue risk	Cycles per truck	Growth factor	Single lane factor	First year ADT	Mean life	Eval life	Main unit is truss	Cost impact factor	Agcy cost main	Appr unit is truss	Cost impact factor	Agcy cost appr	Hazard prob (%)	Risk index	Agency cost (\$k)	User cost (\$k)
brkey	Has fatigue riskS	CycS	Growth factorS	SLFS	T1S	Mean lifeS	Eval lifeS	Truss mainS	CIF mainS	AC mainS	Truss apprS	CIF apprS	AC apprS	Haz probS	Risk indexS	Agcy costS	User costS
720249	TRUE	1	1.025	0.800	716529	27.6	23.7	FALSE	2.700	4140	FALSE	2.700	292560	99.808	0.9981	296.13	74.54
720343	TRUE	1	1.025	0.800	716529	27.6	23.7	FALSE	2.700	4140	FALSE	2.700	292560	99.808	0.9981	296.13	74.54
124044	TRUE	1	1.002	0.850	1055174	25.9	21.2	FALSE	1.700	2940	FALSE	1.700	48020	99.945	0.9994	50.93	27.03
120084	TRUE	1	1.025	0.850	491193	35.8	31.1	FALSE	2.700	4140	FALSE	2.700	81420	58.937	0.5894	50.43	13.46
120083	TRUE	1	1.025	0.850	491193	35.8	31.1	FALSE	2.700	4140	FALSE	2.700	81420	58.937	0.5894	50.43	13.46
720163	TRUE	1	1.025	0.400	388837	41.6	36.5	FALSE	2.700	1380	FALSE	2.700	22080	99.383	0.9938	23.32	343.82
720153	TRUE	1	1.025	0.400	344744	44.8	39.4	FALSE	1.700	980	FALSE	1.700	19600	92.699	0.9270	19.08	203.21
870077	TRUE	1	1.025	0.400	229956	56.4	50.3	FALSE	1.330	2496	FALSE	1.330	28288	47.570	0.4757	14.64	200.01
740089	TRUE	1	1.025	0.800	1118244	19.7	16.7	FALSE	2.160	3492	FALSE	2.160	10476	100.000	1.0000	13.97	108.10
870591	TRUE	1	1.021	1.000	550431	35.2	30.3	FALSE	1.000	4200	FALSE	1.000	4200	94.797	0.9480	7.96	117.41
105500	TRUE	1	1.003	0.800	1578746	17.5	14.3	FALSE	1.330	832	FALSE	1.330	6656	100.000	1.0000	7.49	165.47
105606	TRUE	2	1.004	0.800	535442	25.1	20.6	FALSE	0.510	4032	FALSE	0.510	3024	100.000	1.0000	7.06	80.35
720348	TRUE	1	1.025	0.800	647012	29.7	25.6	FALSE	1.330	832	FALSE	1.330	5824	98.961	0.9896	6.59	24.10
720154	TRUE	1	1.025	0.400	336217	45.5	40.0	FALSE	2.160	1164	FALSE	2.160	5820	93.187	0.9319	6.51	206.83
720259	TRUE	1	1.025	0.800	647012	29.7	25.6	FALSE	1.330	832	FALSE	1.330	4160	98.961	0.9896	4.94	24.10
860198	TRUE	1	1.025	0.800	960035	22.2	18.9	FALSE	2.160	2328	FALSE	2.160	2328	99.974	0.9997	4.65	57.17
860128	TRUE	1	1.025	0.800	960035	22.2	18.9	FALSE	2.160	2328	FALSE	2.160	2328	99.974	0.9997	4.65	70.61
870237	TRUE	1	1.025	0.850	246884	54.3	48.3	FALSE	2.160	8148	FALSE	2.160	4656	35.770	0.3577	4.58	24.47
870407	TRUE	1	1.025	0.800	565237	32.6	28.2	FALSE	1.700	4900	FALSE	1.700	0	91.972	0.9197	4.51	57.19
720200	TRUE	1	1.025	0.850	408906	40.3	35.3	FALSE	1.330	4160	FALSE	1.330	0	97.066	0.9707	4.04	46.39

Figure 10.14. Top 20 bridges vulnerable to fatigue

10.12. References

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Appendix A1: Estimates of hurricanes' likelihood

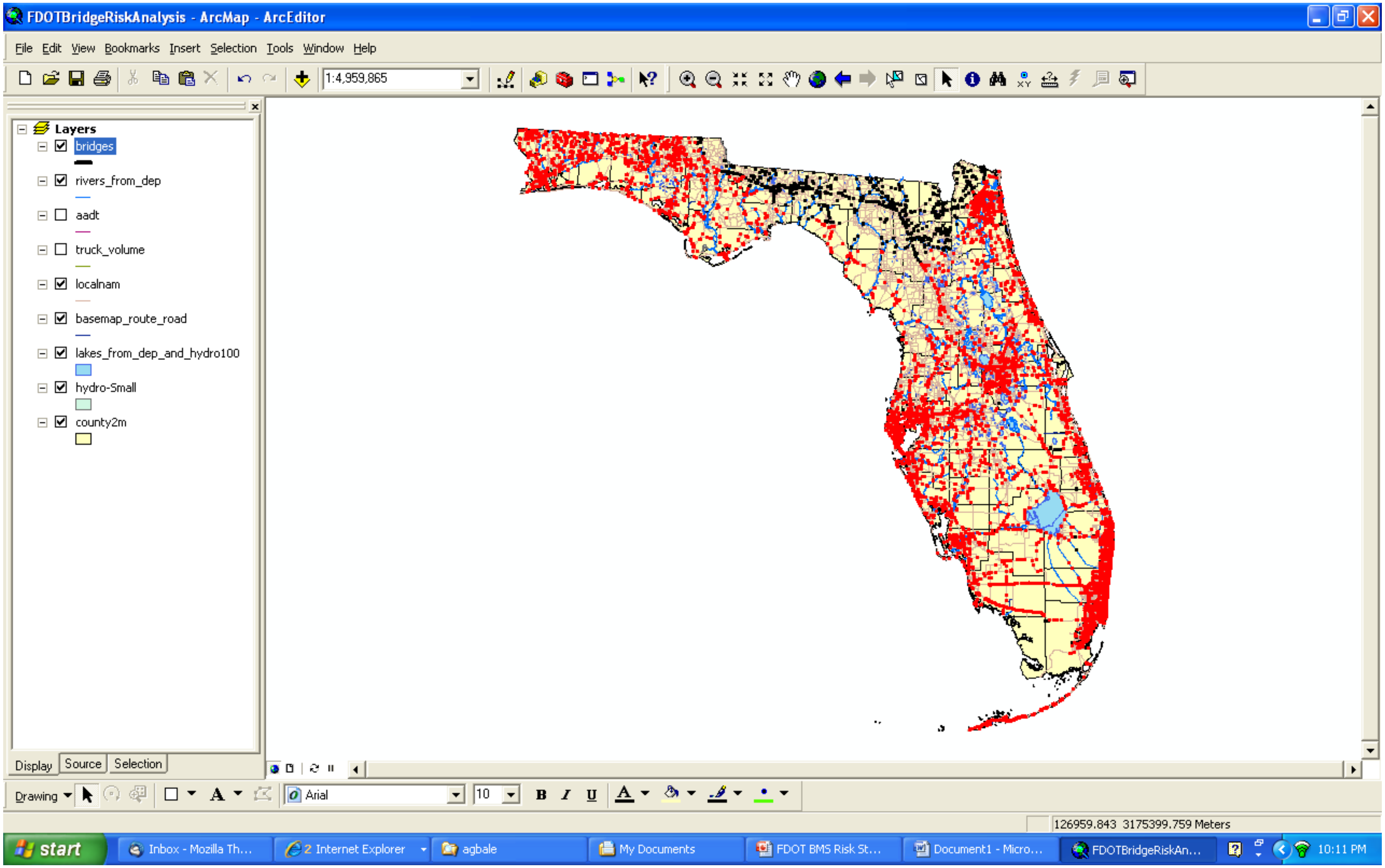

Figure A1.1. Bridges with hurricane category 1 probability (1 year) >= 0.04 (max. prob. = 0.095)

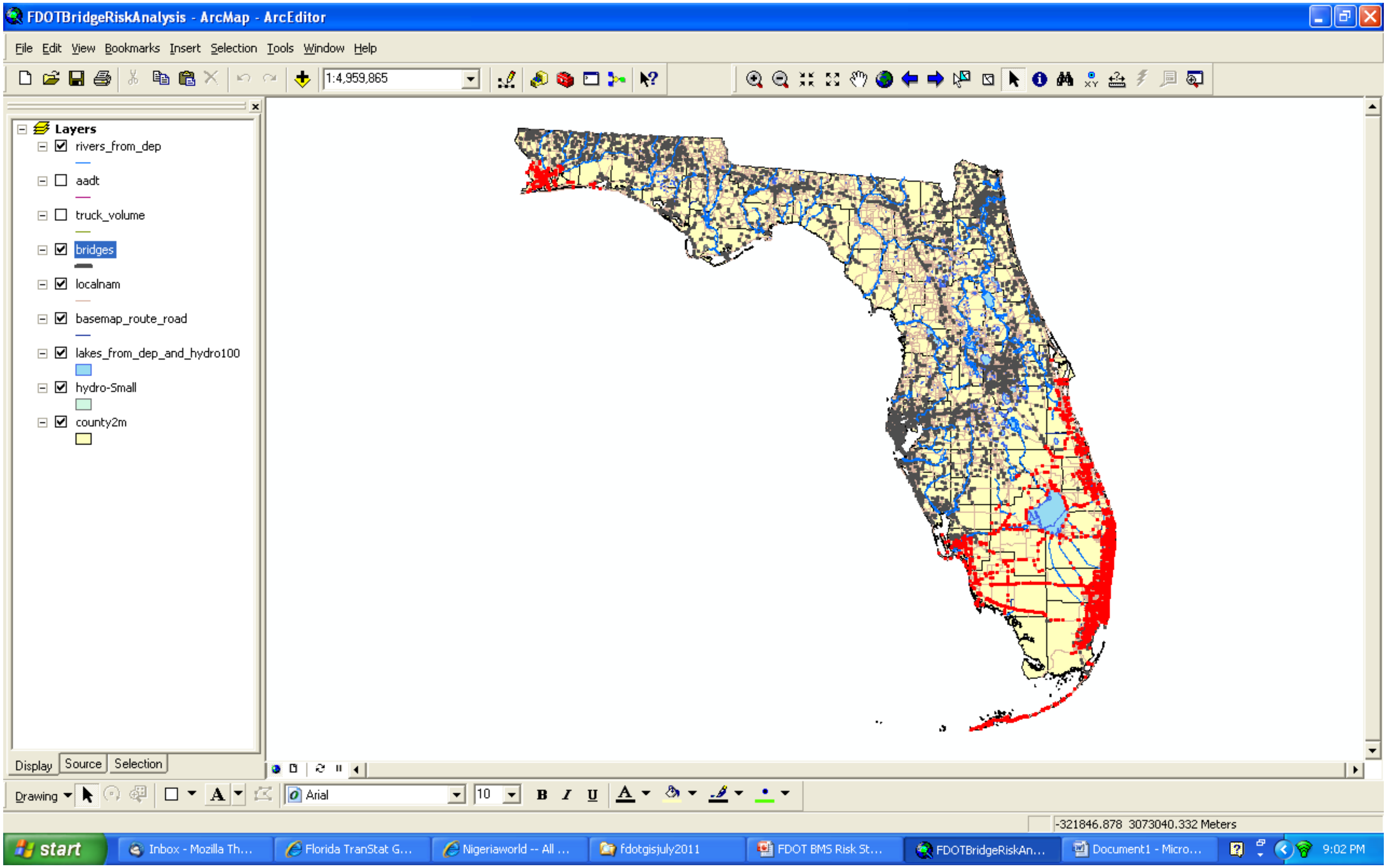

Figure A1.2. Bridges with hurricane category 1 probability (1 year) = 0.095

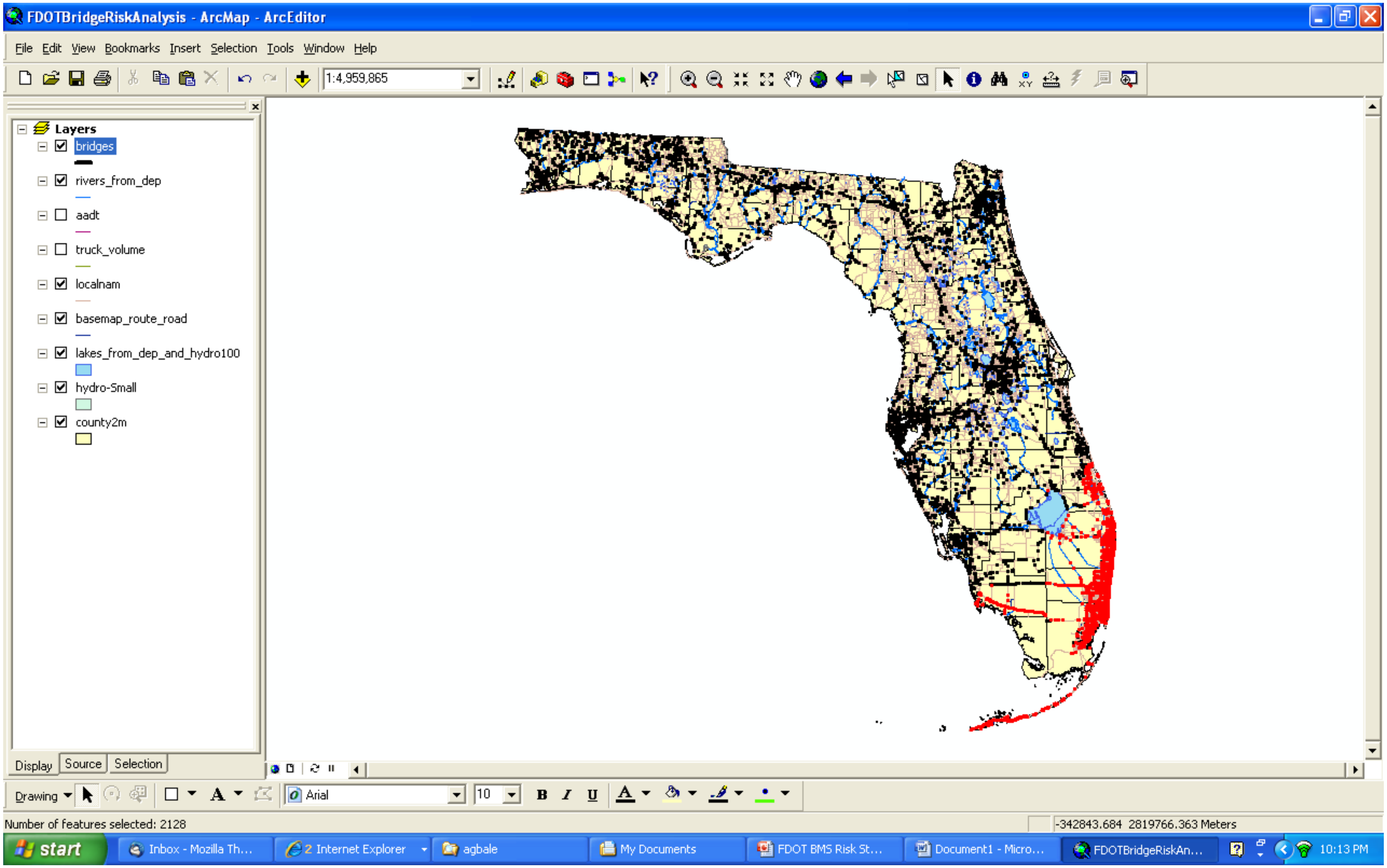

Figure A1.3. Bridges with hurricane category 2 probability (1 year) = 0.049

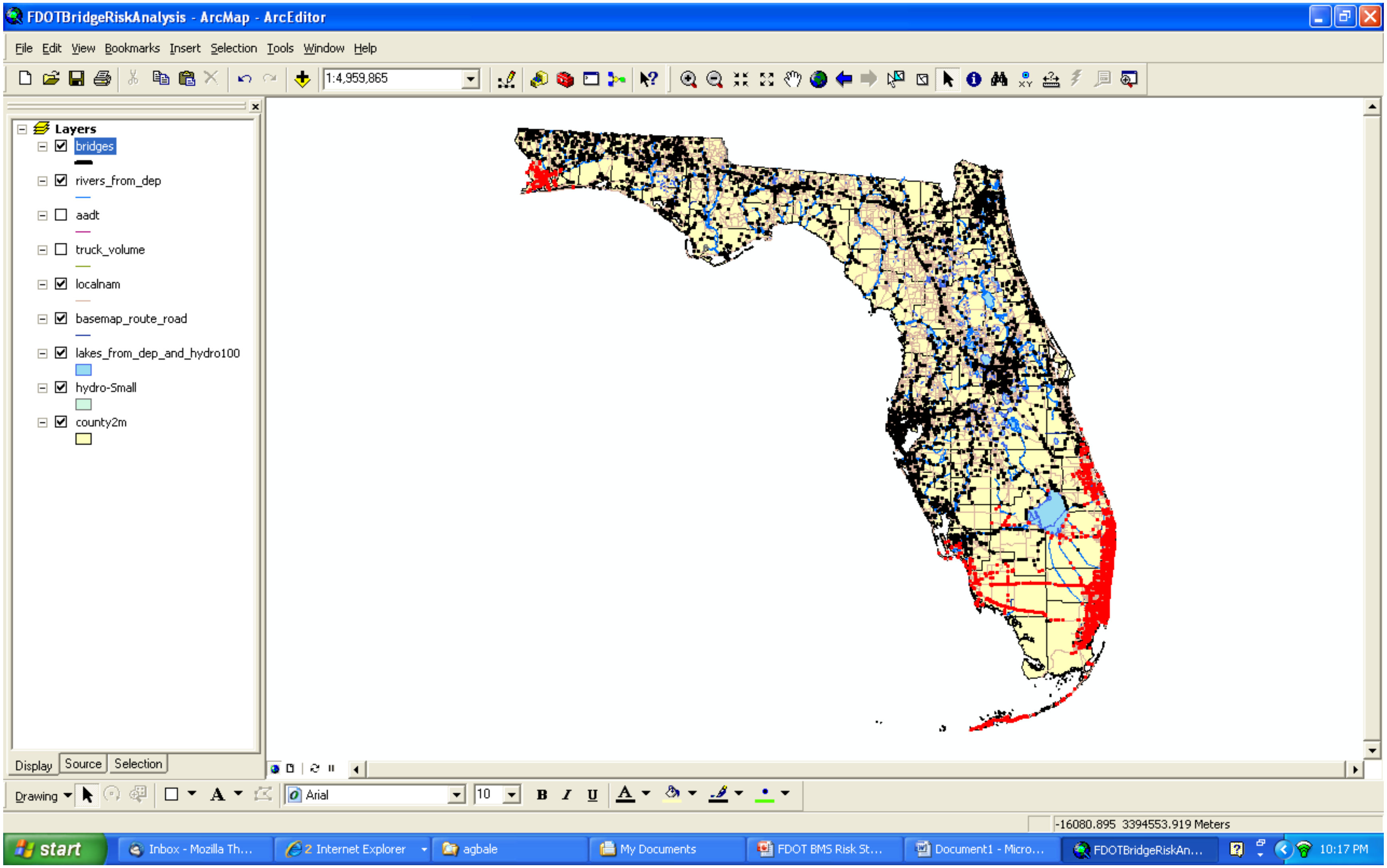

Figure A1.4. Bridges with hurricane category 3 probability (1 year) = 0.0198

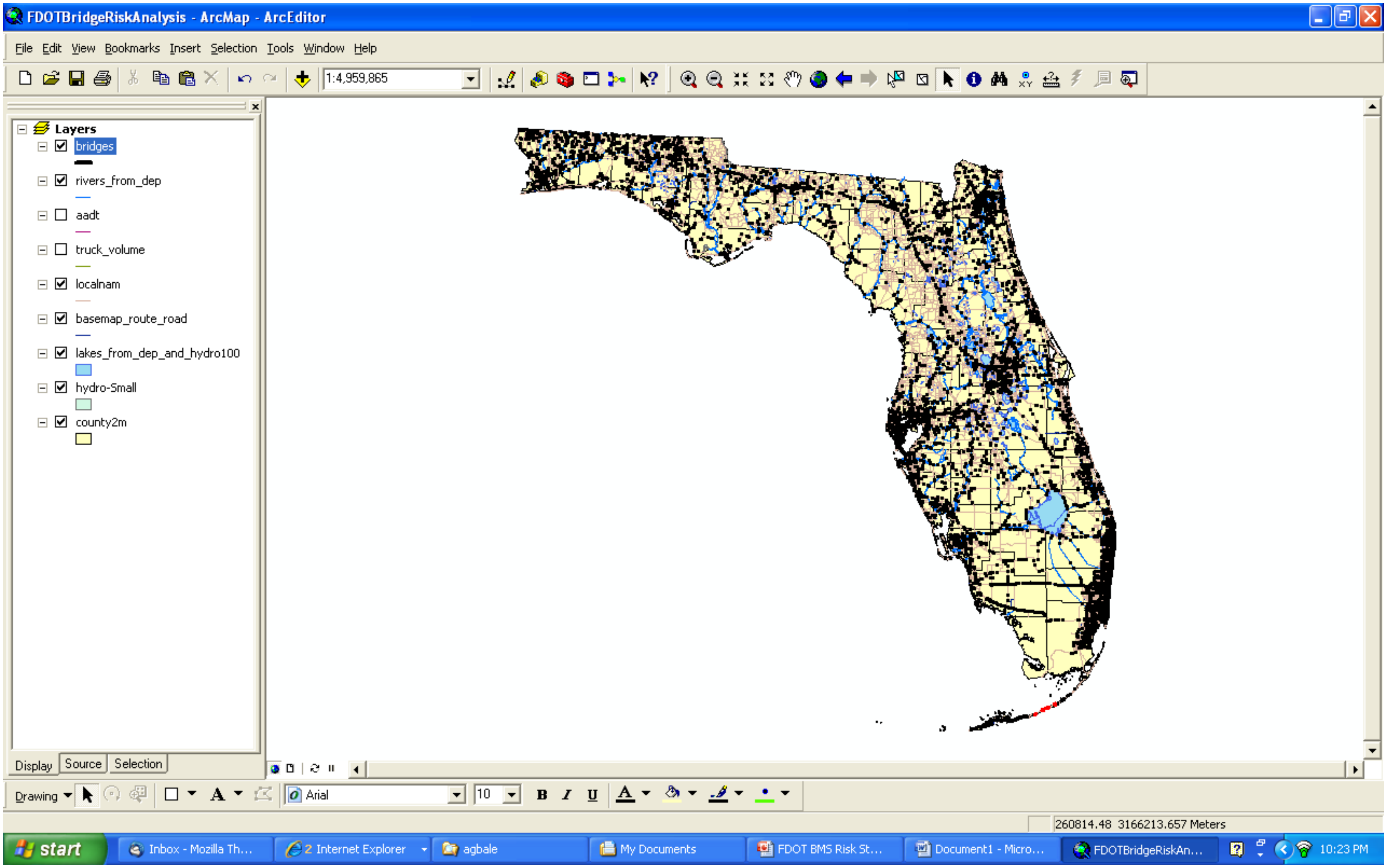

Figure A1.5. Bridges with hurricane category 4 probability (1 year) = 0.0198

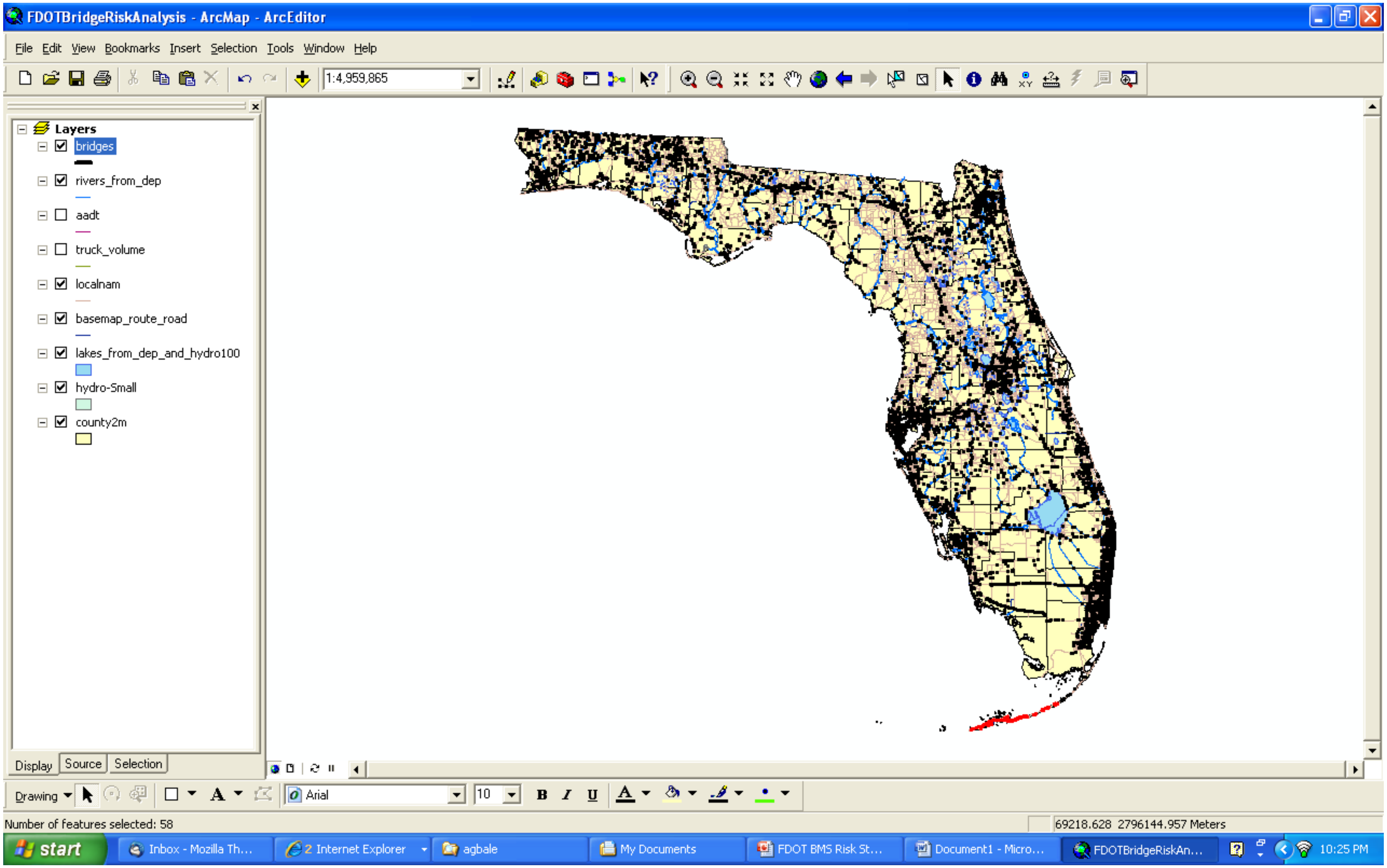

Figure A1.6. Bridges with hurricane category 5 probability (1 year) = 0.00499

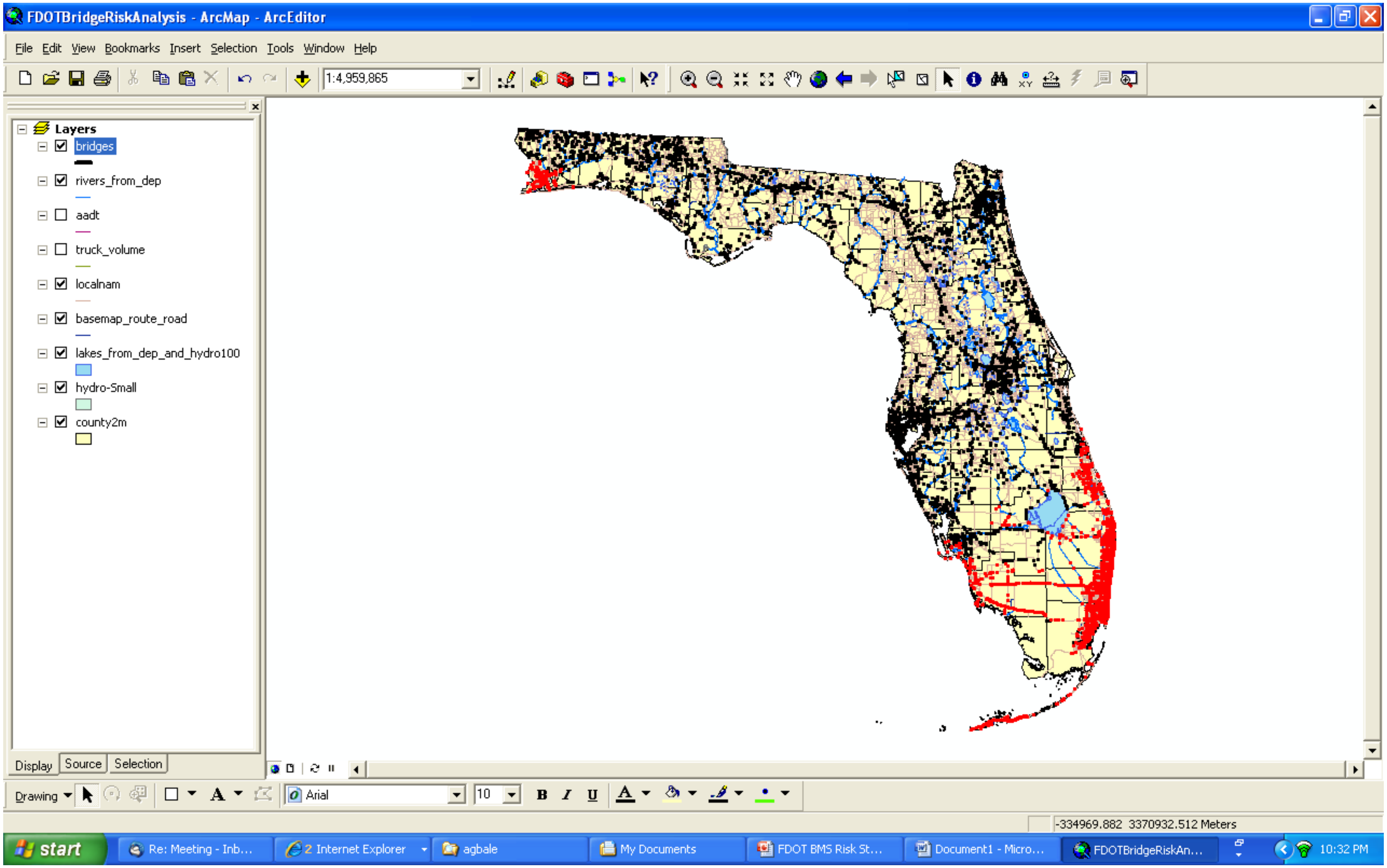

Figure A1.7. Bridges with hurricane category 3 probability (at least 1 event in10 years) >= 0.15 (max. prob. = 0.18)

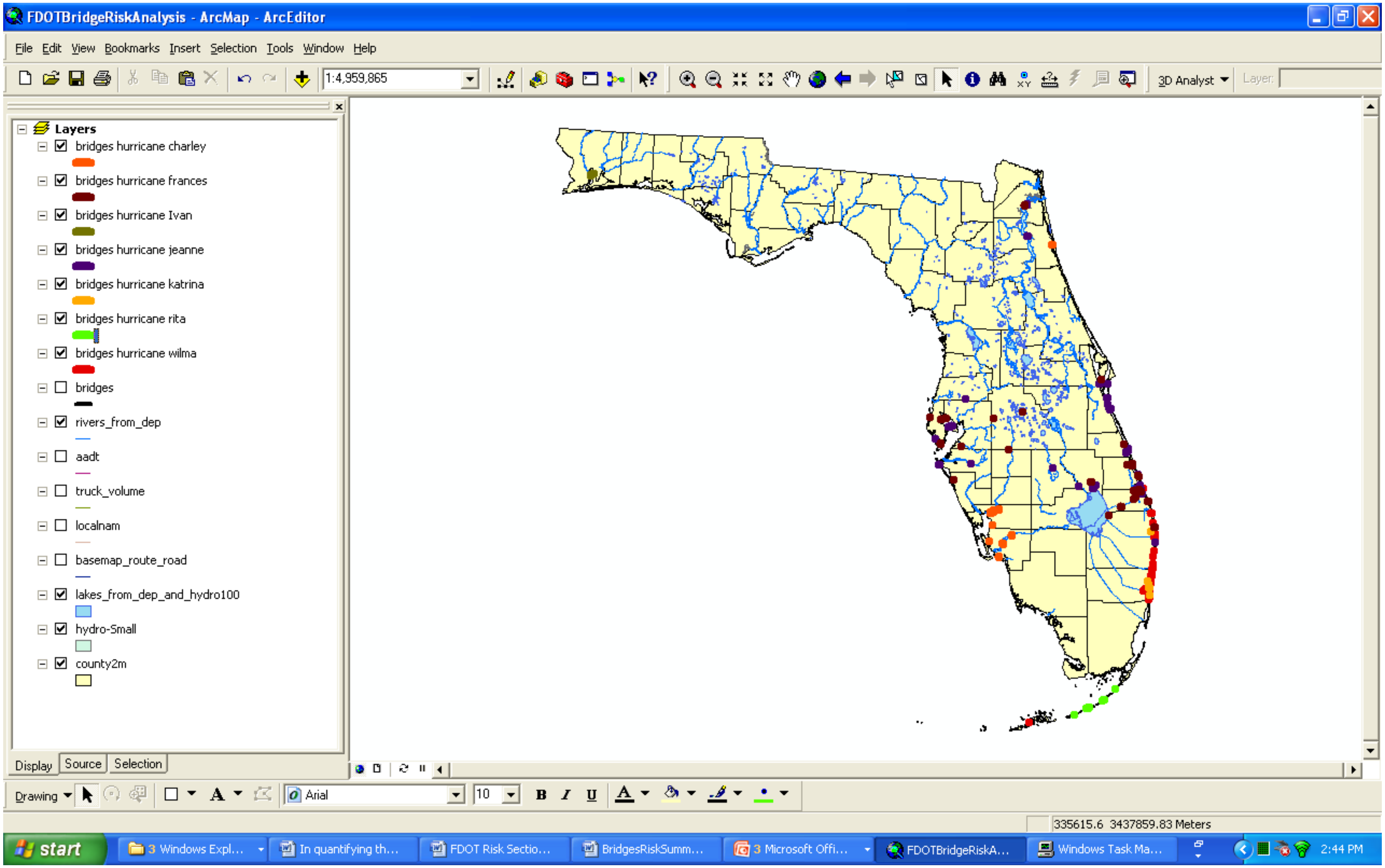

Figure A1.8. Bridges damaged during hurricanes in Florida.

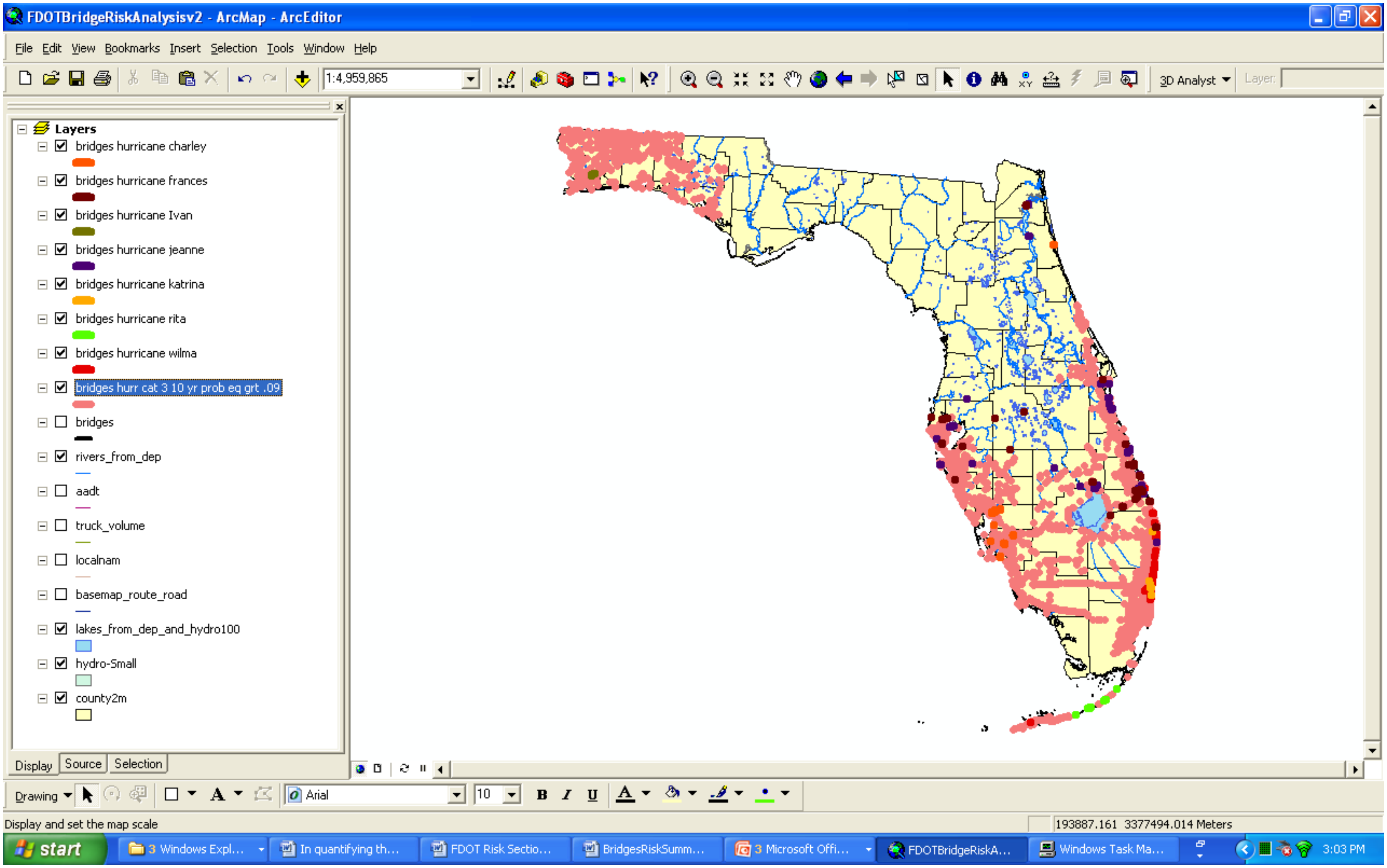

Figure A1.9. Bridges damaged during hurricanes in Florida relative to category 3 probability (at least 1 event within 10 years) >= 9%.

Appendix A2: Estimates of tornadoes ‘likelihood

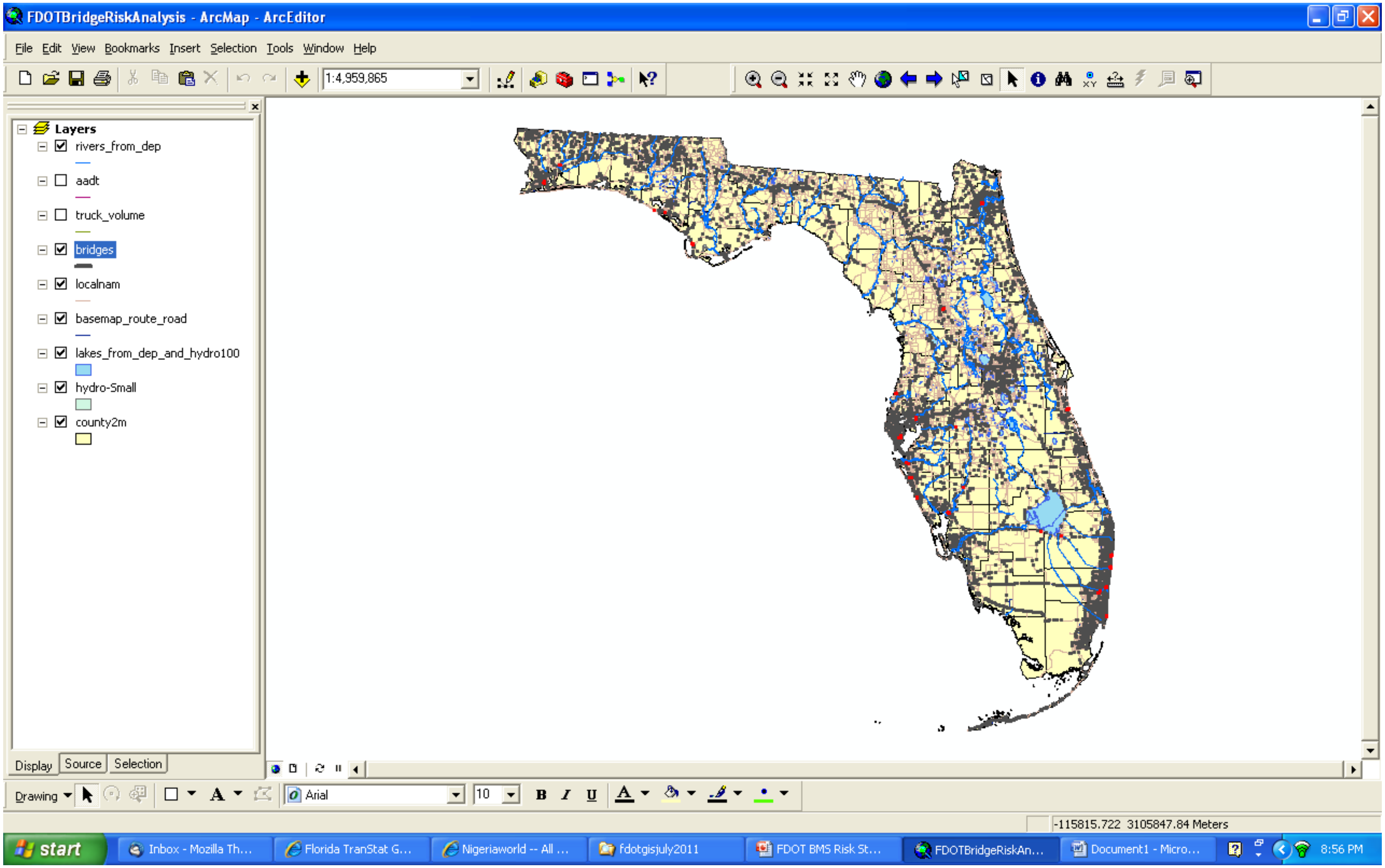

Figure A2.1. Bridges with tornado probability (1 year) >= 0.1 (max. prob. = 0.25)

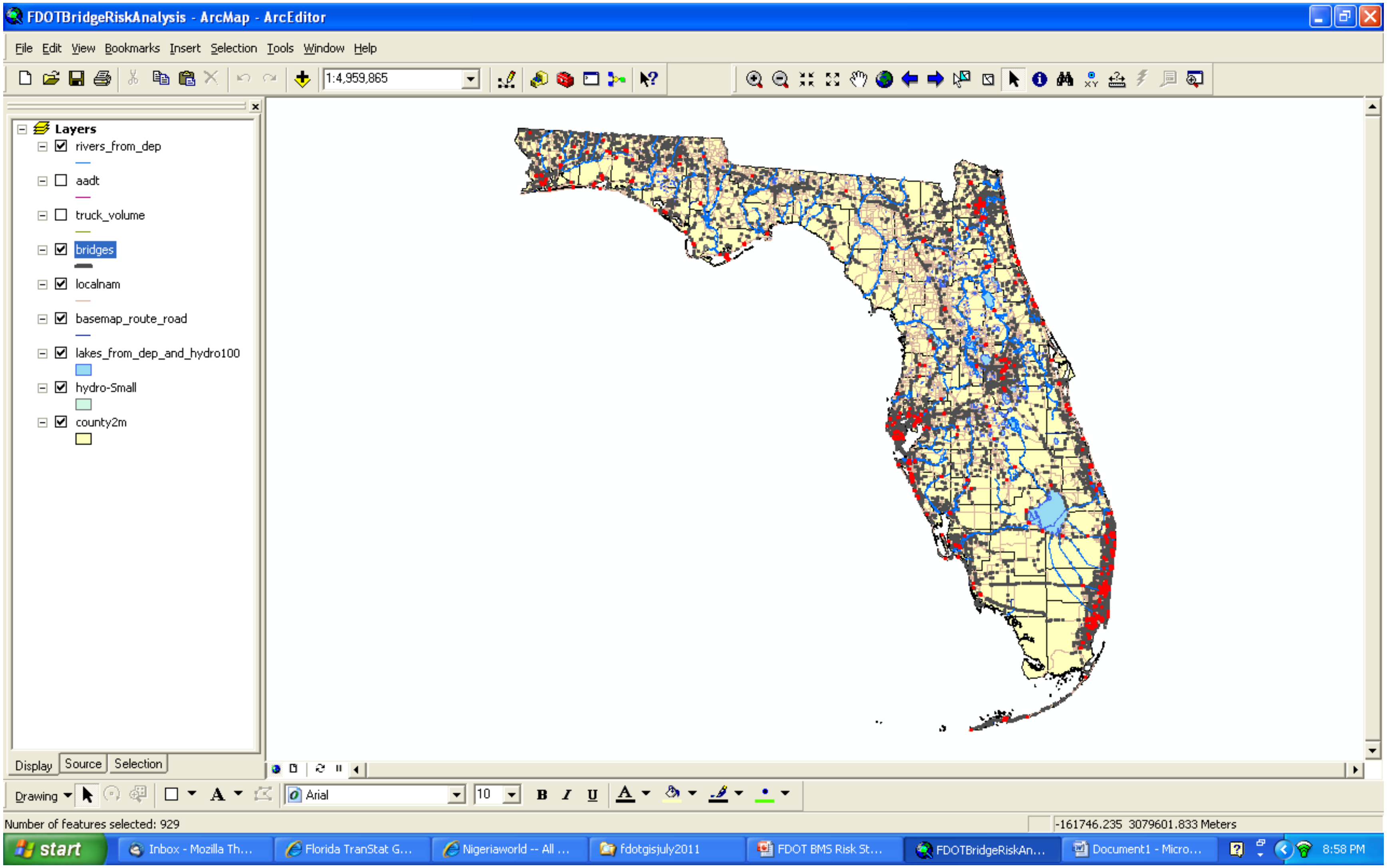

Figure A2.2. Bridges with tornado probability (at least 1 event in10 years) >= 0.25 (max. prob. = 0.95)

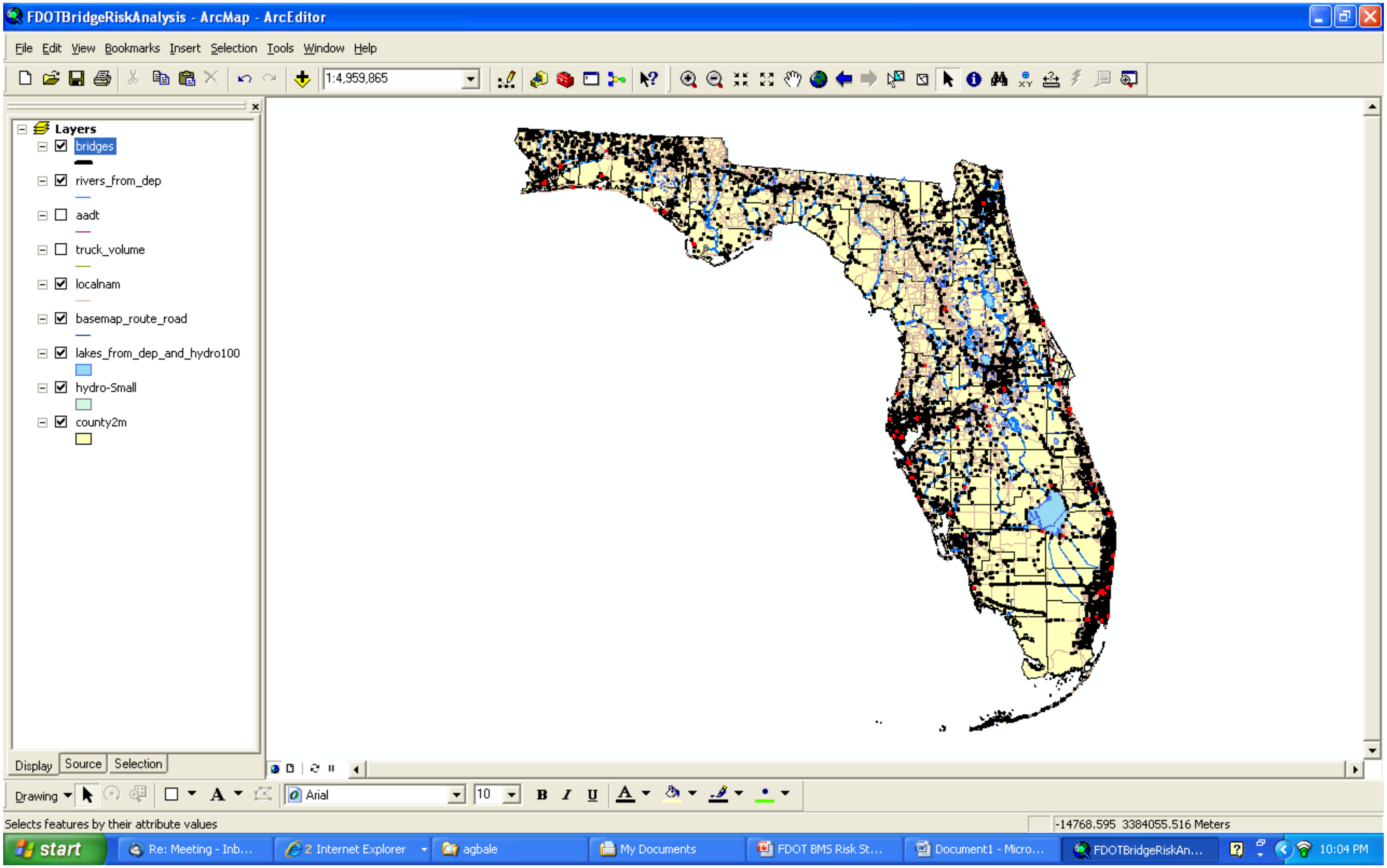

Figure A2.3. Bridges with tornado probability (at least 1 event in10 years) >= 0.5 (max. prob. = 0.95)

Table A2.1. County-based likelihood (annual rates) of tornado events in Florida (14-yr. data)

	F0 Tornado		F1 Tornado		F2 Tornado		F3 Tornado		F4 Tornado		F2 or Greater	
County	Count	Annual rates	Counts	Annual rates	Count	Annual rates	Counts	Annual rates	Counts	Annual rates	Counts	Annual rates
Alachua	20	0.333	13	0.217	8	0.133	0	0.000	0	0.000	8	0.133
Baker	8	0.133	1	0.017	2	0.033	0	0.000	0	0.000	2	0.033
Bay	27	0.450	27	0.450	12	0.200	1	0.017	0	0.000	13	0.217
Bradford	7	0.117	8	0.133	1	0.017	0	0.000	0	0.000	1	0.017
Brevard	57	0.950	37	0.617	15	0.250	3	0.050	0	0.000	18	0.300
Broward	74	1.233	28	0.467	7	0.117	3	0.050	0	0.000	10	0.167
Calhoun	4	0.067	6	0.100	7	0.117	0	0.000	0	0.000	7	0.117
Charlotte	40	0.667	6	0.100	4	0.067	0	0.000	0	0.000	4	0.067
Citrus	33	0.550	11	0.183	2	0.033	1	0.017	0	0.000	3	0.050
Clay	6	0.100	10	0.167	6	0.100	0	0.000	0	0.000	6	0.100
Collier	41	0.683	12	0.200	3	0.050	0	0.000	0	0.000	3	0.050
Columbia	10	0.167	5	0.083	4	0.067	1	0.017	0	0.000	5	0.083
De Soto	25	0.417	5	0.083	3	0.050	0	0.000	0	0.000	3	0.050
Dixie	2	0.033	1	0.017	0	0.000	0	0.000	0	0.000	0	0.000
Duval	38	0.633	17	0.283	8	0.133	0	0.000	0	0.000	8	0.133
Escambia	47	0.783	27	0.450	9	0.150	3	0.050	0	0.000	12	0.200
Flagler	17	0.283	2	0.033	2	0.033	0	0.000	0	0.000	2	0.033
Franklin	20	0.333	13	0.217	4	0.067	0	0.000	0	0.000	4	0.067
Gadsden	5	0.083	13	0.217	10	0.167	0	0.000	0	0.000	10	0.167
Gilchrist	1	0.017	0	0.000	2	0.033	0	0.000	0	0.000	2	0.033
Glades	11	0.183	3	0.050	1	0.017	0	0.000	0	0.000	1	0.017
Gulf	19	0.317	5	0.083	3	0.050	0	0.000	0	0.000	3	0.050
Hamilton	4	0.067	2	0.033	2	0.033	0	0.000	0	0.000	2	0.033
Hardee	14	0.233	4	0.067	0	0.000	0	0.000	0	0.000	0	0.000
Hendry	13	0.217	8	0.133	3	0.050	0	0.000	0	0.000	3	0.050
Hernando	26	0.433	7	0.117	0	0.000	0	0.000	0	0.000	0	0.000
Highlands	24	0.400	13	0.217	3	0.050	0	0.000	0	0.000	3	0.050
Hillsborough	74	1.233	43	0.717	14	0.233	0	0.000	0	0.000	14	0.233
Holmes	7	0.117	5	0.083	3	0.050	0	0.000	0	0.000	3	0.050
Indian River	13	0.217	6	0.100	2	0.033	0	0.000	0	0.000	2	0.033
Jackson	13	0.217	16	0.267	5	0.083	0	0.000	0	0.000	5	0.083
Jefferson	7	0.117	5	0.083	0	0.000	0	0.000	0	0.000	0	0.000
Lafayette	4	0.067	3	0.050	2	0.033	1	0.017	0	0.000	3	0.050
Lake	22	0.367	19	0.317	7	0.117	3	0.050	0	0.000	10	0.167
Lee	74	1.233	21	0.350	11	0.183	0	0.000	0	0.000	11	0.183
Leon	13	0.217	3	0.050	5	0.083	0	0.000	0	0.000	5	0.083
Levy	13	0.217	7	0.117	2	0.033	0	0.000	0	0.000	2	0.033
Liberty	4	0.067	5	0.083	0	0.000	0	0.000	0	0.000	0	0.000
Madison	6	0.100	8	0.133	2	0.033	1	0.017	0	0.000	3	0.050
Manatee	54	0.900	16	0.267	6	0.100	1	0.017	0	0.000	7	0.117
Marion	23	0.383	22	0.367	9	0.150	1	0.017	0	0.000	10	0.167
Martin	19	0.317	4	0.067	3	0.050	1	0.017	0	0.000	4	0.067
Miami Dade	101	1.683	24	0.400	6	0.100	1	0.017	0	0.000	7	0.117
Monroe	41	0.683	15	0.250	6	0.100	0	0.000	0	0.000	6	0.100
Nassau	21	0.350	5	0.083	0	0.000	0	0.000	0	0.000	0	0.000
Okaloosa	57	0.950	19	0.317	14	0.233	1	0.017	0	0.000	15	0.250
Okeechobee	12	0.200	4	0.067	3	0.050	0	0.000	0	0.000	3	0.050
Orange	26	0.433	19	0.317	9	0.150	3	0.050	0	0.000	12	0.200
Osceola	16	0.267	12	0.200	3	0.050	1	0.017	1	0.017	5	0.083
Palm Beach	105	1.750	37	0.617	9	0.150	1	0.017	0	0.000	10	0.167
Pasco	56	0.933	19	0.317	6	0.100	0	0.000	0	0.000	6	0.100
Pinellas	71	1.183	32	0.533	12	0.200	2	0.033	1	0.017	15	0.250
Polk	97	1.617	39	0.650	12	0.200	0	0.000	2	0.033	14	0.233
Putnam	33	0.550	8	0.133	3	0.050	0	0.000	0	0.000	3	0.050
Santa Rosa	38	0.633	19	0.317	5	0.083	1	0.017	0	0.000	6	0.100
Sarasota	42	0.700	23	0.383	8	0.133	1	0.017	0	0.000	9	0.150
Seminole	12	0.200	9	0.150	5	0.083	1	0.017	0	0.000	6	0.100
St. Johns	38	0.633	5	0.083	5	0.083	2	0.033	0	0.000	7	0.117
St. Lucie	24	0.400	7	0.117	2	0.033	2	0.033	0	0.000	4	0.067
Sumter	8	0.133	1	0.017	3	0.050	1	0.017	0	0.000	4	0.067
Suwannee	20	0.333	14	0.233	6	0.100	0	0.000	0	0.000	6	0.100
Taylor	5	0.083	5	0.083	4	0.067	0	0.000	0	0.000	4	0.067
Union	3	0.050	3	0.050	0	0.000	0	0.000	0	0.000	0	0.000
Volusia	59	0.983	21	0.350	12	0.200	4	0.067	0	0.000	16	0.267
Wakulla	8	0.133	6	0.100	1	0.017	0	0.000	0	0.000	1	0.017
Walton	25	0.417	14	0.233	4	0.067	1	0.017	0	0.000	5	0.083
Washington	8	0.133	14	0.233	1	0.017	0	0.000	0	0.000	1	0.017
Totals	1865		841		331		42		4		377	

Table A2.2. County-based likelihood (probability) of tornado events in Florida (14-yr. data)

	Probability of occurrence within 1 year						Probability of occurrence within 10 years					
County	F0	F1	F2	F3	F4	>= F2	F0	F1	F2	F3	F4	>= F2
Alachua	0.283	0.195	0.125	0.000	0.000	0.125	0.964	0.885	0.736	0.000	0.000	0.736
Baker	0.125	0.017	0.033	0.000	0.000	0.033	0.736	0.154	0.283	0.000	0.000	0.283
Bay	0.362	0.362	0.181	0.017	0.000	0.195	0.989	0.989	0.865	0.154	0.000	0.885
Bradford	0.110	0.125	0.017	0.000	0.000	0.017	0.689	0.736	0.154	0.000	0.000	0.154
Brevard	0.613	0.460	0.221	0.049	0.000	0.259	1.000	0.998	0.918	0.393	0.000	0.950
Broward	0.709	0.373	0.110	0.049	0.000	0.154	1.000	0.991	0.689	0.393	0.000	0.811
Calhoun	0.064	0.095	0.110	0.000	0.000	0.110	0.487	0.632	0.689	0.000	0.000	0.689
Charlotte	0.487	0.095	0.064	0.000	0.000	0.064	0.999	0.632	0.487	0.000	0.000	0.487
Citrus	0.423	0.168	0.033	0.017	0.000	0.049	0.996	0.840	0.283	0.154	0.000	0.393
Clay	0.095	0.154	0.095	0.000	0.000	0.095	0.632	0.811	0.632	0.000	0.000	0.632
Collier	0.495	0.181	0.049	0.000	0.000	0.049	0.999	0.865	0.393	0.000	0.000	0.393
Columbia	0.154	0.080	0.064	0.017	0.000	0.080	0.811	0.565	0.487	0.154	0.000	0.565
De Soto	0.341	0.080	0.049	0.000	0.000	0.049	0.984	0.565	0.393	0.000	0.000	0.393
Dixie	0.033	0.017	0.000	0.000	0.000	0.000	0.283	0.154	0.000	0.000	0.000	0.000
Duval	0.469	0.247	0.125	0.000	0.000	0.125	0.998	0.941	0.736	0.000	0.000	0.736
Escambia	0.543	0.362	0.139	0.049	0.000	0.181	1.000	0.989	0.777	0.393	0.000	0.865
Flagler	0.247	0.033	0.033	0.000	0.000	0.033	0.941	0.283	0.283	0.000	0.000	0.283
Franklin	0.283	0.195	0.064	0.000	0.000	0.064	0.964	0.885	0.487	0.000	0.000	0.487
Gadsden	0.080	0.195	0.154	0.000	0.000	0.154	0.565	0.885	0.811	0.000	0.000	0.811
Gilchrist	0.017	0.000	0.033	0.000	0.000	0.033	0.154	0.000	0.283	0.000	0.000	0.283
Glades	0.168	0.049	0.017	0.000	0.000	0.017	0.840	0.393	0.154	0.000	0.000	0.154
Gulf	0.271	0.080	0.049	0.000	0.000	0.049	0.958	0.565	0.393	0.000	0.000	0.393
Hamilton	0.064	0.033	0.033	0.000	0.000	0.033	0.487	0.283	0.283	0.000	0.000	0.283
Hardee	0.208	0.064	0.000	0.000	0.000	0.000	0.903	0.487	0.000	0.000	0.000	0.000
Hendry	0.195	0.125	0.049	0.000	0.000	0.049	0.885	0.736	0.393	0.000	0.000	0.393
Hernando	0.352	0.110	0.000	0.000	0.000	0.000	0.987	0.689	0.000	0.000	0.000	0.000
Highlands	0.330	0.195	0.049	0.000	0.000	0.049	0.982	0.885	0.393	0.000	0.000	0.393
Hillsborough	0.709	0.512	0.208	0.000	0.000	0.208	1.000	0.999	0.903	0.000	0.000	0.903
Holmes	0.110	0.080	0.049	0.000	0.000	0.049	0.689	0.565	0.393	0.000	0.000	0.393
Indian River	0.195	0.095	0.033	0.000	0.000	0.033	0.885	0.632	0.283	0.000	0.000	0.283
Jackson	0.195	0.234	0.080	0.000	0.000	0.080	0.885	0.931	0.565	0.000	0.000	0.565
Jefferson	0.110	0.080	0.000	0.000	0.000	0.000	0.689	0.565	0.000	0.000	0.000	0.000
Lafayette	0.064	0.049	0.033	0.017	0.000	0.049	0.487	0.393	0.283	0.154	0.000	0.393
Lake	0.307	0.271	0.110	0.049	0.000	0.154	0.974	0.958	0.689	0.393	0.000	0.811
Lee	0.709	0.295	0.168	0.000	0.000	0.168	1.000	0.970	0.840	0.000	0.000	0.840
Leon	0.195	0.049	0.080	0.000	0.000	0.080	0.885	0.393	0.565	0.000	0.000	0.565
Levy	0.195	0.110	0.033	0.000	0.000	0.033	0.885	0.689	0.283	0.000	0.000	0.283
Liberty	0.064	0.080	0.000	0.000	0.000	0.000	0.487	0.565	0.000	0.000	0.000	0.000
Madison	0.095	0.125	0.033	0.017	0.000	0.049	0.632	0.736	0.283	0.154	0.000	0.393
Manatee	0.593	0.234	0.095	0.017	0.000	0.110	1.000	0.931	0.632	0.154	0.000	0.689
Marion	0.318	0.307	0.139	0.017	0.000	0.154	0.978	0.974	0.777	0.154	0.000	0.811
Martin	0.271	0.064	0.049	0.017	0.000	0.064	0.958	0.487	0.393	0.154	0.000	0.487
Miami Dade	0.814	0.330	0.095	0.017	0.000	0.110	1.000	0.982	0.632	0.154	0.000	0.689
Monroe	0.495	0.221	0.095	0.000	0.000	0.095	0.999	0.918	0.632	0.000	0.000	0.632
Nassau	0.295	0.080	0.000	0.000	0.000	0.000	0.970	0.565	0.000	0.000	0.000	0.000
Okaloosa	0.613	0.271	0.208	0.017	0.000	0.221	1.000	0.958	0.903	0.154	0.000	0.918
Okeechobee	0.181	0.064	0.049	0.000	0.000	0.049	0.865	0.487	0.393	0.000	0.000	0.393
Orange	0.352	0.271	0.139	0.049	0.000	0.181	0.987	0.958	0.777	0.393	0.000	0.865
Osceola	0.234	0.181	0.049	0.017	0.017	0.080	0.931	0.865	0.393	0.154	0.154	0.565
Palm Beach	0.826	0.460	0.139	0.017	0.000	0.154	1.000	0.998	0.777	0.154	0.000	0.811
Pasco	0.607	0.271	0.095	0.000	0.000	0.095	1.000	0.958	0.632	0.000	0.000	0.632
Pinellas	0.694	0.413	0.181	0.033	0.017	0.221	1.000	0.995	0.865	0.283	0.154	0.918
Polk	0.801	0.478	0.181	0.000	0.033	0.208	1.000	0.998	0.865	0.000	0.283	0.903
Putnam	0.423	0.125	0.049	0.000	0.000	0.049	0.996	0.736	0.393	0.000	0.000	0.393
Santa Rosa	0.469	0.271	0.080	0.017	0.000	0.095	0.998	0.958	0.565	0.154	0.000	0.632
Sarasota	0.503	0.318	0.125	0.017	0.000	0.139	0.999	0.978	0.736	0.154	0.000	0.777
Seminole	0.181	0.139	0.080	0.017	0.000	0.095	0.865	0.777	0.565	0.154	0.000	0.632
St. Johns	0.469	0.080	0.080	0.033	0.000	0.110	0.998	0.565	0.565	0.283	0.000	0.689
St. Lucie	0.330	0.110	0.033	0.033	0.000	0.064	0.982	0.689	0.283	0.283	0.000	0.487
Sumter	0.125	0.017	0.049	0.017	0.000	0.064	0.736	0.154	0.393	0.154	0.000	0.487
Suwannee	0.283	0.208	0.095	0.000	0.000	0.095	0.964	0.903	0.632	0.000	0.000	0.632
Taylor	0.080	0.080	0.064	0.000	0.000	0.064	0.565	0.565	0.487	0.000	0.000	0.487
Union	0.049	0.049	0.000	0.000	0.000	0.000	0.393	0.393	0.000	0.000	0.000	0.000
Volusia	0.626	0.295	0.181	0.064	0.000	0.234	1.000	0.970	0.865	0.487	0.000	0.931
Wakulla	0.125	0.095	0.017	0.000	0.000	0.017	0.736	0.632	0.154	0.000	0.000	0.154
Walton	0.341	0.208	0.064	0.017	0.000	0.080	0.984	0.903	0.487	0.154	0.000	0.565
Washington	0.125	0.208	0.017	0.000	0.000	0.017	0.736	0.903	0.154	0.000	0.000	0.154

Appendix A3: Estimates of wildfires' likelihood

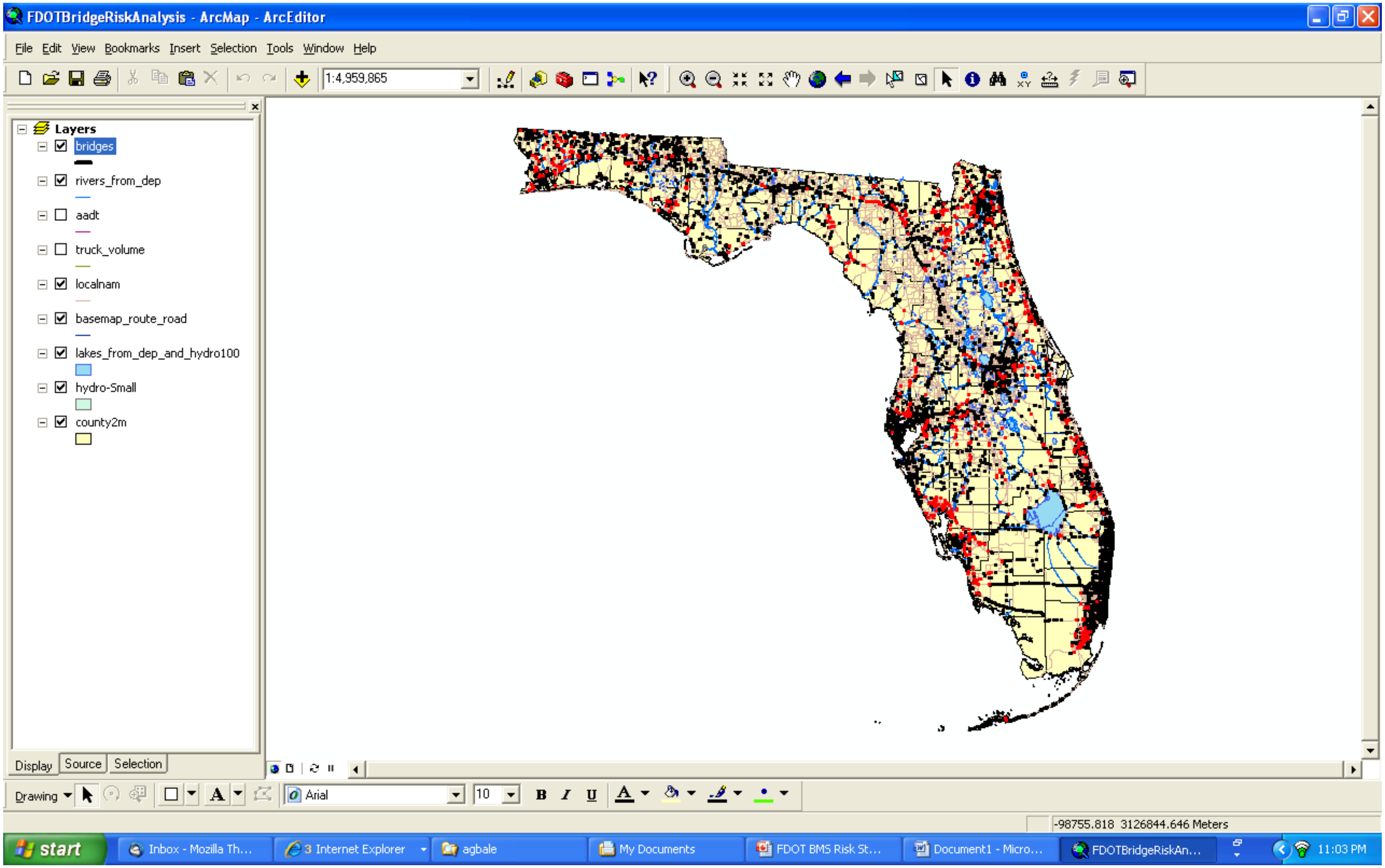

Figure A3.1. Bridges with wildfire probability (1 year) >= 0.5 (max. prob. = 0.99)

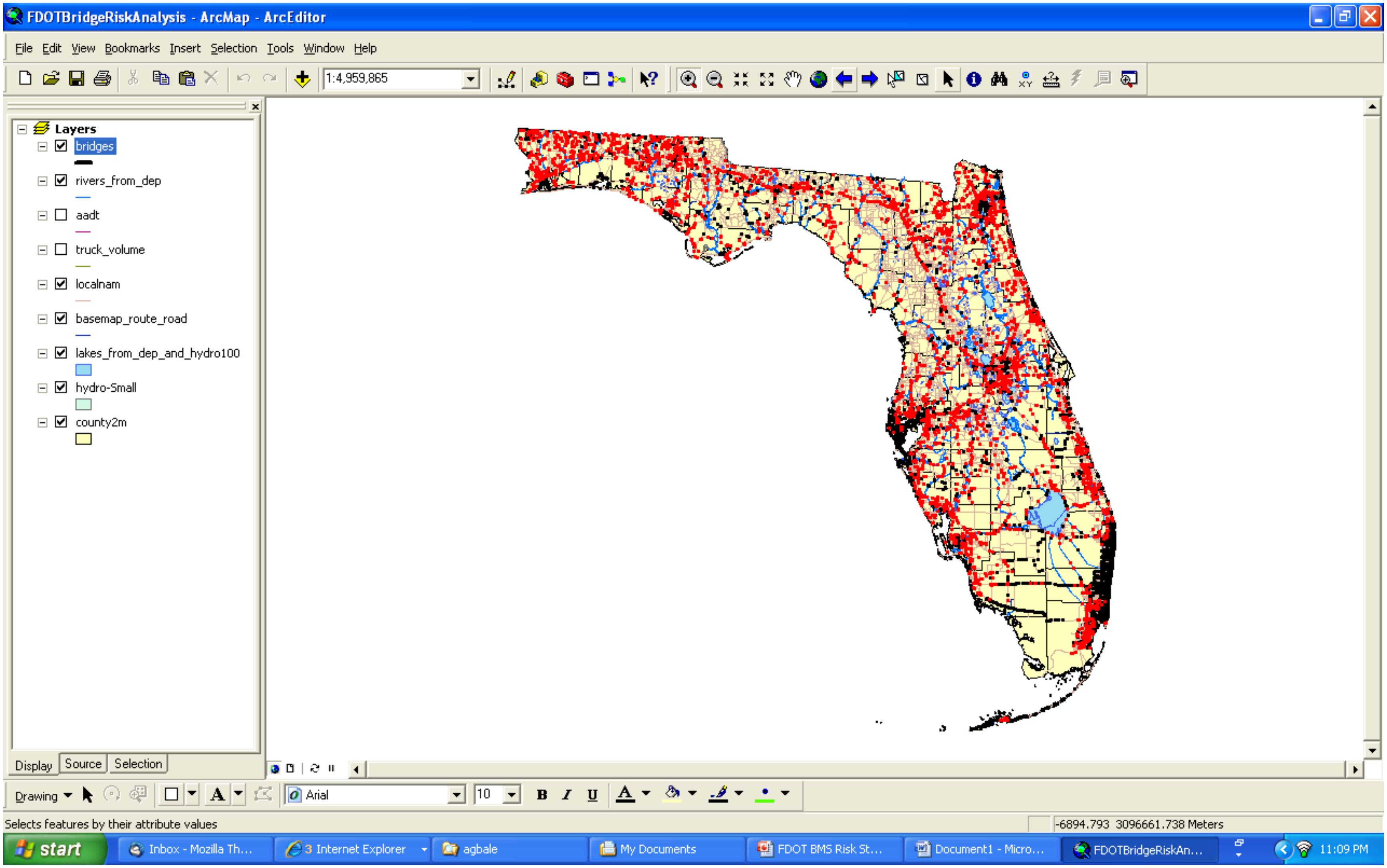

Figure A3.2. Bridges with wildfire probability (at least 1 event in10 years) >= 0.75 (max. prob. = 0.99)

Table A3.1. County-based likelihood of wildfire events in Florida (60-yr. data)

County	Counts	Annual rates	Probability of occurrence within 1 year	Probability of occurrence within 10 years
Alachua	14	1.000	0.632	1.000
Baker	7	0.500	0.393	0.993
Bay	8	0.571	0.435	0.997
Bradford	2	0.143	0.133	0.760
Brevard	2	0.143	0.133	0.760
Broward	9	0.643	0.474	0.998
Calhoun	1	0.071	0.069	0.510
Charlotte	5	0.357	0.300	0.972
Citrus	2	0.143	0.133	0.760
Clay	4	0.286	0.249	0.943
Collier	13	0.929	0.605	1.000
Columbia	8	0.571	0.435	0.997
De Soto	1	0.071	0.069	0.510
Dixie	1	0.071	0.069	0.510
Duval	4	0.286	0.249	0.943
Escambia	1	0.071	0.069	0.510
Flagler	8	0.571	0.435	0.997
Franklin	3	0.214	0.193	0.883
Gadsden	0	0.000	0.000	0.000
Gilchrist	3	0.214	0.193	0.883
Glades	7	0.500	0.393	0.993
Gulf	5	0.357	0.300	0.972
Hamilton	3	0.214	0.193	0.883
Hardee	0	0.000	0.000	0.000
Hendry	7	0.500	0.393	0.993
Hernando	1	0.071	0.069	0.510
Highlands	1	0.071	0.069	0.510
Hillsborough	3	0.214	0.193	0.883
Holmes	0	0.000	0.000	0.000
Indian River	0	0.000	0.000	0.000
Jackson	0	0.000	0.000	0.000
Jefferson	0	0.000	0.000	0.000
Lafayette	5	0.357	0.300	0.972
Lake	0	0.000	0.000	0.000
Lee	12	0.857	0.576	1.000
Leon	0	0.000	0.000	0.000
Levy	0	0.000	0.000	0.000
Liberty	6	0.429	0.349	0.986
Madison	1	0.071	0.069	0.510
Manatee	1	0.071	0.069	0.510
Marion	20	1.429	0.760	1.000
Martin	0	0.000	0.000	0.000
Miami Dade	15	1.071	0.657	1.000
Monroe	1	0.071	0.069	0.510
Nassau	11	0.786	0.544	1.000
Okaloosa	0	0.000	0.000	0.000
Okeechobee	0	0.000	0.000	0.000
Orange	0	0.000	0.000	0.000
Osceola	0	0.000	0.000	0.000
Palm Beach	8	0.571	0.435	0.997
Pasco	0	0.000	0.000	0.000
Pinellas	1	0.071	0.069	0.510
Polk	11	0.786	0.544	1.000
Putnam	19	1.357	0.743	1.000
Santa Rosa	0	0.000	0.000	0.000
Sarasota	6	0.429	0.349	0.986
Seminole	1	0.071	0.069	0.510
St. Johns	4	0.286	0.249	0.943
St. Lucie	1	0.071	0.069	0.510
Sumter	0	0.000	0.000	0.000
Suwannee	1	0.071	0.069	0.510
Taylor	3	0.214	0.193	0.883
Union	3	0.214	0.193	0.883
Volusia	1	0.071	0.069	0.510
Wakulla	5	0.357	0.300	0.972
Walton	2	0.143	0.133	0.760
Washington	0	0.000	0.000	0.000
Totals	261			

Appendix A4: Estimates of floods' likelihood

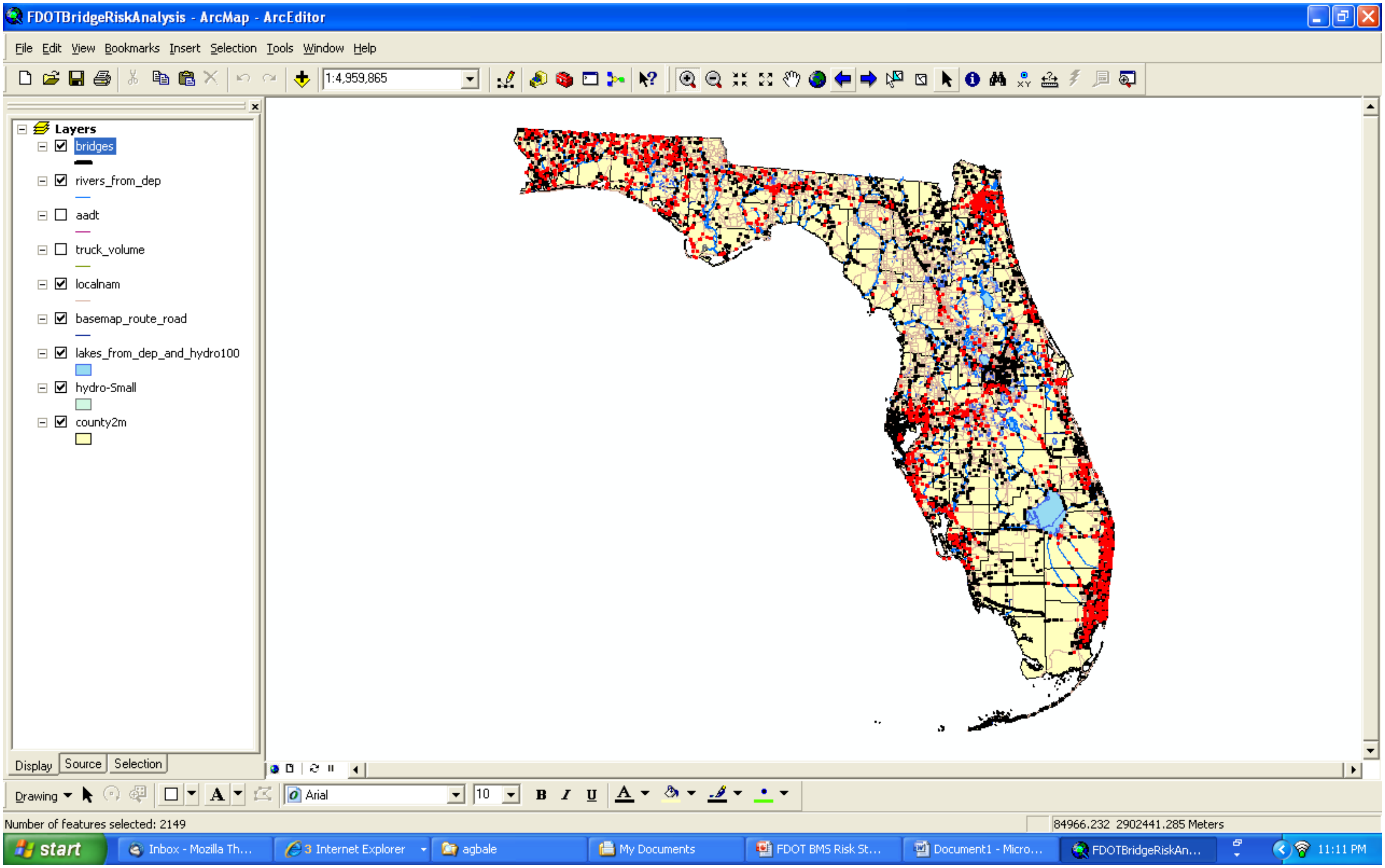

Figure A4.1. Bridges with flooding probability (1 year) = 0.01 (high risk)

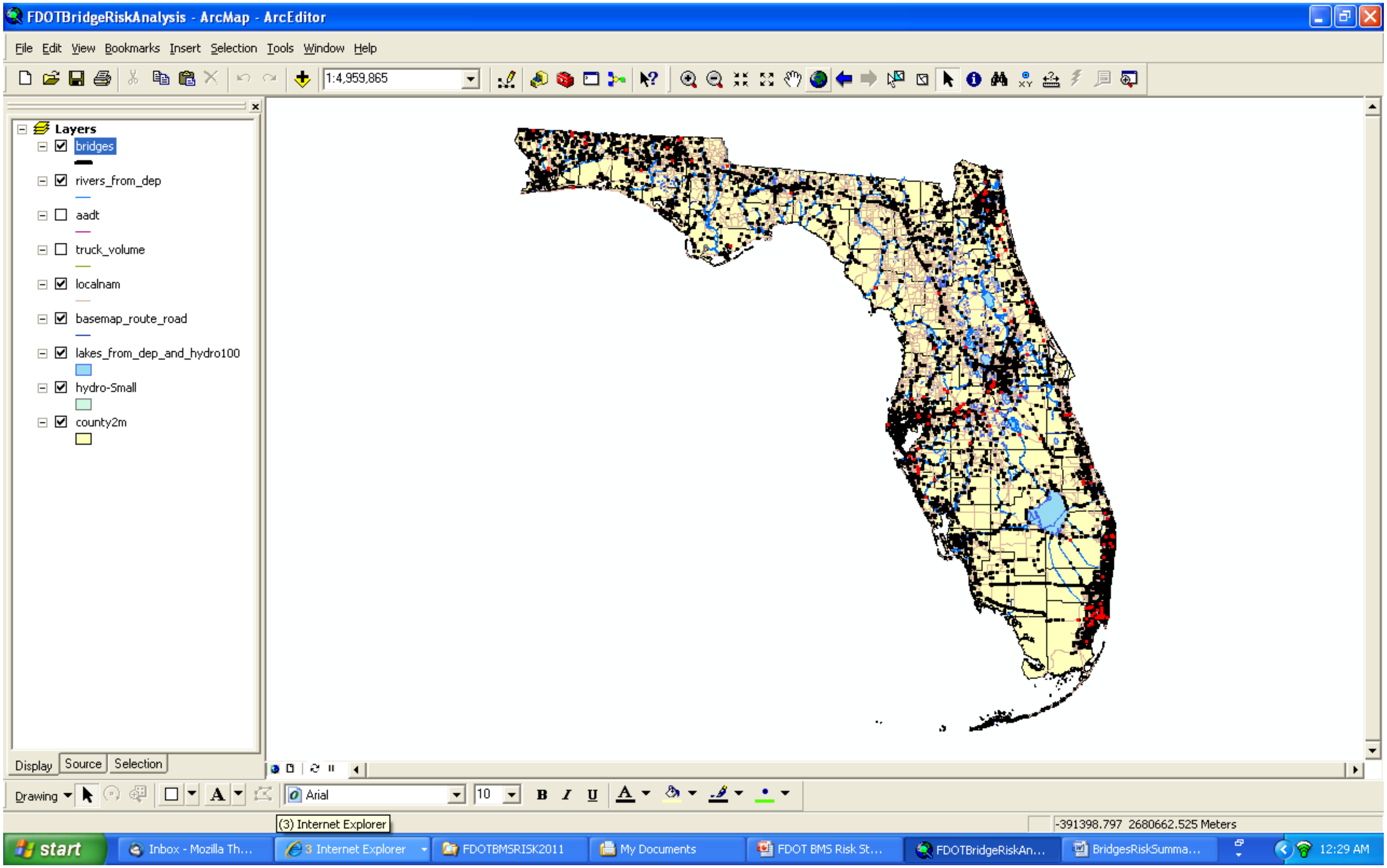

Figure A4.2. Bridges with flooding probability (1 year) = 0.002 (moderate risk)

Appendix A5: Consequences of hazards

Table A5.1. Summary of structures replaced or posted due to hazards ("Central Office" Bridges)

No.	Structure ID	Hazard Type	Hazard Description/Consequences	Year Built	Final Inspection Date
1	530006	Flood	Scour concerns. Bridge replaced.	1972	8/8/2005
2	530019	Flood	Scour concerns. Bridge replaced.	1972	4/19/2006
3	574073	Flood	Extreme flood damage. Bridge deck and superstructure not pile-supported.Bridge closed.	1984	2/9/2010
4	590022	Flood	Flood: Closure of the bridge is recommended during near overtopping events, or if any additional scour occurs or evidence of settlement is found. Bridge replaced.	1956	2/2/2010
5	710058	Flood	Scour concerns. Bridge replaced.	1986	11/19/2008
6	710059	Flood	Scour concerns. Bridge replaced.	1986	11/19/2008
7	720062	Flood	Scour concerns. Roadway overtopping.	1949	6/4/2009
8	039002	Hurricane	Hurricane damage and bad general condition: bridge closed and replaced.	1969	11/3/2005
9	489549	Hurricane	Hurricane-related deficiencies.		7/13/2005
10	589503	Hurricane	Hurricane-related deficiencies.		7/12/2005
11	48S544	Hurricane	Hurricane-related deficiencies.		7/13/2005
12	79S019	Hurricane Frances	Hurricane Frances damage. Removed.		9/8/2004
13	79S054	Hurricane Frances	Hurricane Frances damage. Removed.		9/8/2004
14	94S602	Hurricane Frances	Hurricane Frances damage. Removed.		9/16/2004
15	480213	Hurricane Ivan*	Hurricane Ivan caused severe damage to the eastbound I-10 bridge. Damage to deck, superstructure and substructure. Both parallel bridges replaced.	1968	9/16/2004[#]
16	480214	Hurricane Ivan*	Hurricane Ivan caused severe damage to the eastbound I-10 bridge. Damage to deck, superstructure and substructure. Both parallel bridges replaced.	1968	9/16/2004[#]
17	75S532	Hurricane Jeanne	Hurricane Jeanne damage. Total loss.	2001	9/29/2004
18	704049	Hurricane Wilma	Hurricane Wilma: Boat impact and pile/cap damages. Bridge Posted.	1949	3/25/2010
19	12S205	Hurricane Wilma	Hurricane Wilma damage. Sign replaced.	1996	9/9/2006
20	86S027	Hurricane Wilma	Hurricane Wilma damage. Sign replaced.	1995	9/13/2005
21	86S042	Hurricane Wilma	Hurricane Wilma damage. Removed.	1987	8/26/2005
22	86S060	Hurricane Wilma	Hurricane Wilma damage. Removed.	1993	8/26/2005
23	86S468	Hurricane Wilma	Hurricane Wilma damage. Removed.		7/13/2005
24	86S469	Hurricane Wilma	Hurricane Wilma damage. Removed.		7/13/2005
25	86S719	Hurricane Wilma	Hurricane Wilma damage. Sign replaced.		12/9/2005
26	93S210	Hurricane Wilma	Hurricane Wilma damage. Sign replaced.		11/9/2005
27	93S472	Hurricane Wilma	Hurricane Wilma damage. Sign replaced.		10/31/2007
28	720166	Vehicular Impact	Vehicle accident impact on bridge. Damaged girders and other non-impact related deficiencies. Bridge replaced.	1957	3/26/2009
29	72S081	Vehicular Impact	Vehicle accident impact damage. Removed.		2/25/2004
30	72S126	Vehicular Impact	Vehicle accident impact damage. Removed.		7/14/2004
31	72S391	Vehicular Impact	Vehicle accident impact damage. Removed.		4/22/2004

* Not shown in the Central Office Bridge list; Cost to replace the two parallel bridges (480213 and 480214) was listed as $243 million (http://www.roadtraffic-technology.com/projects/i-10/).

[#] Date of Hurricane; inspection date not available.

Survey Questionnaire: Florida Hazards and Effects on Bridges

Florida DOT is currently conducting a study on natural and manmade hazards that had occurred in Florida and their impacts on the bridges. Please complete this survey questionnaire for each bridge affected based on your experience with such hazards within the last 10 years. The completed survey can be "saved as" a text file, scanned as a pdf file and emailed back, or simply printed and mailed to the address shown below. You may use additional sheets or attach other pertinent documents. It would be greatly appreciated if the completed survey is returned by Friday December 16, 2011.

The Hazard Event Type (Check one): ☐ **Hurricane** ☐ **Tornado** ☒ **Flood** ☐ **Fire**

Brief Description of the Hazard: This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.

Dates of Hazard Event: Start Date: Oct 20, 2008 End Date: Oct 22, 2008

Structure ID: 914002

Description of Bridge Elements Damaged: Element 290 - Channel: The north end of the floor slab was undermined with maximum penetration of 7 ft. There was a 2'x2'x10" depression behind the retaining wall. There was also a 3'x3'x2' and a 8'x8'x4' washout at the NE wingwall and the Se wingwall respectively.

Costs of Repair: Repair construction cost was $462,825. Miscellaneous cost such as design and construction engineering inspection cost was $61,851.

Roadway Closures: Roadway Name: CR-724 (Over Gore Slough)

No. of Lanes Closed: 2 Duration of Closure (Hours): 960*

Other Comments: The exact closure time is undeterminable at this time. This road was closed for at least 40 days.

Thank you very much!

Contact: John Sobanjo, Research Principal Investigator
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2525 Pottsdamer Street, Rm. 129, Tallahassee, FL 32309
Phone: 850.410.6153; Email: jsobanjo@fsu.edu

Figure A5.1. Sample completed survey questionnaire for bridge damage

Table A5.2. Summary of hazard impacts (with costs or road closure) on bridges in Florida

District	Hazard Event Type	Hazard Date (or Inspection date)	Bridge ID	Description of the Hazard	Bridge Elements Damaged	Described Costs of Repair	Costs of Repair	No. of Lanes Closed	Duration of Closure (Hours)	Other Comments
Turnpike	Fire	12/14/2009	110070	Vehicle fire occurring underneath Turnpike Bridge Structure 110070 resulting in structural damage and subsequent repairs.	reinf. conc. deck, prestr. conc. Beams, reinf. conc. Substructure	Structural Repair Cost: $981.0 K Traffic Diversion Cost: $825.5 K Hazardous Waste Mitigation Cost: $28.5 K Misc. Support Services: $82.1 K	$1,917,100	2	244	Detour and roadway diversion established to accommodate repair work.
1&7	Fire	6/4/2008	130103	This bridge was damaged due to fire form a tanker truck accident on 06/04/08.	12, 109, 205, 234	Repair cost was about $4.5M.	$4,500,000	3	384	Southbound traffic on I-75 was directed to Northbound bridge during the repair.
1&7	Fire	3/28/2007	150124	A tanker truck crashed on the ramp and caught on fire.	12, 109, 205, 234, 301, 302, 303, 310, 331, 398		$2,300,000	2	576	The ramp was closed during the reconstruction. The cost show does not include DOT costs or by other parties. Minor superstructure damage occurred to Bridge 150158 for Pontis elements 12, 301, 331, 398, 310, 205, and 234. The cost shown above includes the repairs to that bridge as well.
6	Fire	10/3/2002	870363	Fuel tanker truck lost control while exiting I-95 to SR-836 in Miami.	underdeck, expansion joints , drainage gutters, girders, bearings, columns, caps, substructure	Two bridges cost $955,588	$477,794	3	14	
6	Fire	10/3/2002	870364	Fuel tanker truck lost control while exiting I-95 to SR-836 in Miami.	underdeck, expansion joints , drainage gutters, girders, bearings, columns, caps, substructure	Two bridges cost $955,588	$477,794	3	14	
6	Fire	11/6/2006	870439	Fuel tanker truck lost control while driving on I-95 South Bound over NW 132 Street in Miami-Dade County .	underdeck, girders, columns, caps, bearings, expansion joints, drainage gutters		$375,000	N/A	N/A	
6	Fire	10/2/2005	900101	Fuel Tanker Truck Accident on the Seven Mile Bridge.	12, 331		$89,852	2	15	
4	Fire		940133	smoke and fire damage to bridge	deck, beams		$410,000			
6	Hurricane Dennis	7/10/2005	900091	Hurricane Dennis	321, 396		$33,000	N/A	N/A	
6	Hurricane Dennis	7/10/2005	900098	Hurricane Dennis.	396		$57,750	N/A	N/A	
6	Hurricane Dennis	7/10/2005	900108	Hurricane Dennis.	396		$66,000	N/A	N/A	
6	Hurricane Dennis	7/10/2005	900115	Hurricane Dennis	321, 396		$35,200	N/A	N/A	
1&7	Hurricane Frances	9/5/2004	150138	Fender and Fender Lighting Damage from Hurricane Frances	390, 580		$11,212	0	0	The total shown includes the cost submitted by the asset maintenance contractor for repair of fender.
4	Hurricane Frances	9/8/2004	940005	failed north approach roadway; washed out area; wingwall undermining. Scour and damage to approach slab and bulkhead	321, 393, scour		$800,000			

Table A5.2. Summary of hazard impacts (with costs or road closure) on bridges in Florida (Cont'd)

District	Hazard Event Type	Hazard Date (or Inspection date)	Bridge ID	Description of the Hazard	Bridge Elements Damaged	Described Costs of Repair	Costs of Repair	No. of Lanes Closed	Duration of Closure (Hours)	Other Comments
2	Hurricane Jeanne/Flood	9/28/2004	780056	Extremely high water led to over-topping of the South bulkhead and subsequent washout.	321, 396		$26,070	1	16	Two-way, two lane traffic.
6	Hurricane Rita	9/20/2005	900078	Hurricane Rita	290, 396		$11,000	N/A	N/A	
6	Hurricane Rita	9/20/2005	900089	Hurricane Rita.	290, 396		$85,140	N/A	N/A	
6	Hurricane Rita	9/20/2005	900094	Hurricane Rita.	396		$57,400	N/A	N/A	
6	Hurricane Rita	9/20/2005	900095	Hurricane Rita.	321, 396		$187,440	N/A	N/A	
6	Hurricane Rita	9/20/2005	900096	Hurricane Rita.	396		$440,000	N/A	N/A	
6	Hurricane Rita	9/20/2005	900126	Hurricane Rita	396		$455,070	N/A	N/A	
4	Hurricane Wilma	12/14/2005	860008		small sign		$1,000			
4	Hurricane Wilma	12/14/2005	860011		580, 591, 592		$15,000			
4	Hurricane Wilma	12/14/2005	860018		581, street light		$77,300			
4	Hurricane Wilma	12/14/2005	860038		581, 591, street light		$9,000			
4	Hurricane Wilma	12/14/2005	860043		581, 591, 592		$15,000			
4	Hurricane Wilma	12/14/2005	860060		386, 580, 591, 592, street light		$15,000			
4	Hurricane Wilma	12/14/2005	860061		386		$60,000			
4	Hurricane Wilma	12/14/2005	860144		386, 580, 581, 592, street light		$10,000			
4	Hurricane Wilma	12/14/2005	860146		581, 592		$20,000			
4	Hurricane Wilma	12/14/2005	860157		581, 592, street light		$13,000			
4	Hurricane Wilma	12/14/2005	860230		small sign, 580, 581		$7,000			
4	Hurricane Wilma	12/14/2005	860466		581, 591, 592		$13,000			
4	Hurricane Wilma	12/14/2005	860618		581, 591		$19,500			
4	Hurricane Wilma	12/14/2005	860622		386, 581, 591, 592		$66,000			
4	Hurricane Wilma	12/14/2005	860623		581		$28,000			
4	Hurricane Wilma	12/14/2005	860920		581, 592		$6,000			
4	Hurricane Wilma	12/14/2005	860941		592		$2,000			
4	Hurricane Wilma	12/14/2005	890003		581, 592, small sign, street light		$15,000			
4	Hurricane Wilma	12/14/2005	890150		386, 580		$35,000			
4	Hurricane Wilma	12/14/2005	890151		580		$2,000			
4	Hurricane Wilma	12/14/2005	890158		386, 580		$21,500			
6	Hurricane Wilma	10/24/2005	900107	Hurricane Wilma	scour/piles		$16,500	N/A	N/A	
4	Hurricane Wilma	12/15/2005	930004		street light, 591, 592		$6,000			
4	Hurricane Wilma	12/15/2005	930060		386, 591, street light		$42,000			
4	Hurricane Wilma	12/15/2005	930064		581, 591		$17,000			
4	Hurricane Wilma	12/15/2005	930097		386, 581, 591, street light		$19,500			
4	Hurricane Wilma	12/15/2005	930104		small sign, 581, 591, 592		$15,000			
4	Hurricane Wilma	12/15/2005	930154		small sign, 581, 591, street light		$20,000			
4	Hurricane Wilma	12/15/2005	930157		386, 580, 581, 591, street light		$24,800			
4	Hurricane Wilma	12/15/2005	930370		581, 591, street light		$28,500			
4	Hurricane Wilma	12/15/2005	930453		581, 591, street light		$25,200			
4	Hurricane Wilma	12/15/2005	930454		591, 592, street light		$7,000			
4	Hurricane Wilma	12/15/2005	930506		small sign, 581, 591, 592		$24,000			
1&7	Hurricane/Tropical Storm Fay	10/20/2008	910003	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	241	Repair construction cost was $276,990. Miscellaneous cost such as design and construction engineering inspection cost was $26,250.	$303,240			

Table A5.2. Summary of hazard impacts (with costs or road closure) on bridges in Florida (Cont'd)

District	Hazard Event Type	Hazard Date (or Inspection date)	Bridge ID	Description of the Hazard	Bridge Elements Damaged	Described Costs of Repair	Costs of Repair	No. of Lanes Closed	Duration of Closure (Hours)	Other Comments
1&7	Hurricane/Tropical Storm Fay	10/20/2008	910006	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	396	Repair construction cost was $117,160. Miscellaneous cost such as design and construction engineering inspection cost was $22,715.	$139,875			There is no closure information at this time.
1&7	Hurricane/Tropical Storm Fay	10/20/2008	910065	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	396	Repair construction cost was $125,482. Miscellaneous cost such as design and construction engineering inspection cost was $21,482.	$146,964			There is no closure information at this time.
1&7	Hurricane/Tropical Storm Fay	10/20/2008	910066	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	396	Repair construction cost was $26,483. Miscellaneous cost such as design and construction engineering inspection cost was $14,519.	$41,002			
1&7	Hurricane/Tropical Storm Fay	10/20/2008	910081	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	396	Repair construction cost was $16,007. Miscellaneous cost such as design and construction engineering inspection cost was $10,410.	$26,417			There is no closure information at this time.
1&7	Tropical Storm Fay (Flood)	10/20/2008	054003	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	290	Repair construction cost was $391,295. Misc. cost (design and construction engineering inspection) was $60,616.	$451,911	2	24	This road was periodically closed from 9/11/08 through 10/07/08 for few hours at a time to accommodate repair construction. The 24 hours lane closure noted above is an estimate of the total closure.
1&7	Tropical Storm Fay (Flood)	10/20/2008	914001	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	290, 321	Repair construction cost was $62,839. Miscellaneous cost such as design and construction engineering inspection cost was $18,275.	$81,114	0	0	
1&7	Tropical Storm Fay (Flood)	10/20/2008	914002	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	290	Repair construction cost was $462,825. Miscellaneous cost such as design and construction engineering inspection cost was $61,851.	$524,676	2	960	The exact closure time is undeterminable at this time. This road was closed for at least 40 days.
1&7	Tropical Storm Fay (Flood)	10/20/2008	914007	This bridge was damaged due to tropical storm Fay which affected the area with heavy rains.	290	Repair construction cost was $306,503. Miscellaneous cost such as design and construction engineering inspection cost was $43,860.	$350,363	2	792	This road was closed for at least 33 days. The exact closure time is not available at his time.

**Note: There are more records but shown here are the ones with costs or durations of closure.

Table A5.3. Summary of damage to Florida bridges and repair costs during Hurricane Wilma

District	Hazard Date (or Inspection	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
4	12/14/2005	860008	Repair signs	Slight	$1,000
4	12/14/2005	860623	Repair lights.	Slight	$28,000
4	12/14/2005	860941	Repair traffic signal.	Slight	$2,000
4	12/14/2005	860011	Repair traffic signals, barrier gate, navigation lights, and traffic gate.	Moderate	$15,000
4	12/14/2005	860018	Repair/replace street light poles, repair street light, roof, speaker in tenderhouse and A/C.	Moderate	$77,300
4	12/14/2005	860038	Replace bridgehouse mirros, repair street light, tenderhouse roof, traffic gate, and repair/replace street light feed.	Moderate	$9,000
4	12/14/2005	860043	Repair computer, signs, traffic light, gate, PA amps/horns, fender gauge light, emergency light in tenderhouse.	Moderate	$15,000
4	12/14/2005	860060	Repair traffic signals, gate, handrail and sidewalk (boat impact), fender gauge light, street light, navigation light, signs and fender handrail (boat impact)	Moderate	$15,000
4	12/14/2005	860061	Repair fender.	Moderate	$60,000
4	12/14/2005	860144	Repair traffic signal, street lights, pedestrian signs, navigation lights, A/C compressor, and repalce fender guage light.	Moderate	$10,000
4	12/14/2005	860146	Repair traffic signals and twisted light poles of the tenderhouse.	Moderate	$20,000
4	12/14/2005	860157	Repair traffic signal, replace the following: (pedestrian) sign, broken fuel gauge, tender house light, street light arm and head, and repair damaged A/C.	Moderate	$13,000
4	12/14/2005	860230	Repair tenderhouse windows, emergency lights, marine roadio, memergency lights, telephone, PA speaker, navigation lights, and "bridge ahead" sign.	Moderate	$7,000
4	12/14/2005	860466	Repair traffic signals, replace gate and sidewalk, repair generator room door, transformer, windows, fender handrials, and control power units.	Moderate	$13,000
4	12/14/2005	860618	Repair gate, bridge house, electrical panel, PA bracket, sump pump and soffet flashing.	Moderate	$19,500
4	12/14/2005	860622	Repair tenderhouse, repalce marine radio antenna, repqair insulation, roof, display, blinds, speaker, PA amps/horn, rpeiar light fxtures, navigation light, gates, traffic	Moderate	$66,000
4	12/14/2005	860920	Repair traffic signal, tenderhouse mirror, and diesel generator.	Moderate	$6,000
4	12/14/2005	890003	Repair "Drawbridge Ahead" sign, traffic signlas, bridgehouse roof, window, and street lights.	Moderate	$15,000
4	12/14/2005	890150	Replace missing navigation lights, vertical clearance gauges, and fender system (incl. conduit).	Moderate	$35,000
4	12/14/2005	890151	Replace span and fender navigation lights.	Moderate	$2,000
4	12/14/2005	890158	Repair vertical clearance gauge, repalce nagivation lights, and repair fender.	Moderate	$21,500
6	10/24/2005	900107	Scour holes around seven of the bridge piles. Holes approximately 5 ft x 5 ft x 4 ft.	Moderate	$16,500
4	12/15/2005	930004	Repair street light, traffic light, and traffic gates.	Moderate	$6,000
4	12/15/2005	930060	Repair traffic gate, barrier gate, street lights, fender (planks missing, gauge light missing), CCTV system (camera missing), PA system horns/amp, waether station, traffic lights, and "Bridge Ahead" sign.	Moderate	$42,000
4	12/15/2005	930064	Repair CCTV system on traffic pole, traffic gate, ATS Voltmeter, and water in fule fill basin.	Moderate	$17,000
4	12/15/2005	930097	Repair traffic gate (electrical damage), street light (pole), fender access ladder, fuel tank platform, and marine radio antenna.	Moderate	$19,500
4	12/15/2005	930104	Repair "Bridge Ahead" sign, traffic signal, barrier gates, span swing light, tenderhouse roof, trafic light, ceiling tiles (tenderhouse), marine radio antenna, and fender (electrical conduit, planks, and clearance gauage).	Moderate	$15,000
4	12/15/2005	930154	Repair "Bridge Ahead" sign, bridgehouse window, ceiling fan, window blinds, traffic gate, elcetrical service (feed) to street lights.	Moderate	$20,000
4	12/15/2005	930157	Repair street light poles, traffic light, traffic gate, fender, bridgedhouse roof, navigation light, and CCTV system.	Moderate	$24,800
4	12/15/2005	930370	Repair traffic signals, street lights, traffic gates, traffic light, PLC, door (wind), A/C in tenderhouse, radiator leaks, and bridgehouse street light.	Moderate	$28,500
4	12/15/2005	930453	Repair/replace street light poles, repair traffic and barrier gates, traffic light sign, gate, and roof.	Moderate	$25,200
4	12/15/2005	930454	Repair traffic signal light, traffic gate, and street light.	Moderate	$7,000
4	12/15/2005	930506	Repair traffic gate, bridge house mirror, traffic signals, street lights, signs, tenderhouse entry gate, and center median grating.	Moderate	$24,000

Table A5.4. Summary of bridges (non-Florida) damaged during hurricane Katrina (Padgett et al., 2008)

Bridge Name	Roadway Carried	Bridge Type	Damage State	Damage Source	Cost Estimate	Surge Elevation (m)	Comments
Alabama							
BayouLaBatre	Hwy 188	Fixed	Moderate	SC	$10,000	—	
Dauphin Island Parkway	193	Fixed	Moderate	I,SC	$6,000,000	—	
Cochrane Africatown USA	US-90	Fixed	Extensive	I	$1,000,000	—	Cost reported as $1.7 million in Hitchcock et al. (2008)
Mobile Delta Causeway	I-10	Fixed	Extensive	D	$1,140,000	—	
Louisiana							
Bayou DesAllemands	LA-631	Movable	Slight	W	$3,000	0.98	
Bayou Dulac	LA-57	Movable	Slight	W	$1,000	1.16	
Country Club	LA-3127	Movable	Slight	W	$1,000	0.91	
Galliano	LA-308	Movable	Slight	W	$5,000	1.95	
Golden Meadow	LA-308	Movable	Slight	W	$9,000	1.95	
Harvey Canal	LA-18	Movable	Slight	W	$2,000	3.54	
Houma Navigation Canal	LA-661	Movable	Slight	W	$1,000	0.91	
Lockport Company Canal	LA-1	Movable	Slight	W	$2,000	0.91	
PresqueIsle @Bayou Petite Caillou	LA-24	Movable	Slight	W	$1,000	—	
Belle Chase	LA-23	Movable	Moderate	W	$200,000	4.08	
Claiborne	LA-39	Movable	Moderate	W	$40,000	—	
Intracoastal Waterway @Larose	LA-1	Movable	Moderate	W	$170,000	1.34	
Perez	LA-23	Movable	Moderate	W	$200,000	—	
Seabrook	LocalRoad	Movable	Moderate	W	$25,000	3.11	
St.Bernard Canal	LA-46	Movable	Moderate	W	$40,000	—	
West Pearl River	US-90	Movable	Moderate	EM,SC,W	$350,000	4.6	
Bayou Barataria	LA-302	Movable	Extensive	EM	$50,000	1.16	
Bayou Lafourche @Leeville	LA-1	Movable	Extensive	SC,W	$1,600,000	2.13	
Bayou Liberty	LA-433	Movable	Extensive	EM,W	$1,500,000	3.47	
Bonfouca	LA-433	Movable	Extensive	EM,W	$200,000	3.57	
Caminada Bay	LA-1	Fixed	Extensive	D,SC	$500,000	2.44	
Chef Menteur	US-90	Movable	Extensive	EM,SC	$3,600,000	3.96	
Doullut Canal	LA-11	Movable	Extensive	EM,W	$700,000	3.44	
East Pearl River	US-90	Movable	Extensive	EM,SC,W	$400,000	4.6	
Inner Harbor Navigation Canal	Florida Ave.	Movable	Extensive	EM,I,W	$500,000	1.01	
NorthDraw—LakePontchartrain	US-11	Movable	Extensive	EM	$50,000	4.02	
Rigolets Pass	US-90	Movable	Extensive	EM,SC,W	$2,000,000	4.6	
Rigolets Pass–Under Construction	US-90	Movable	Extensive	I,W	$1,700,000	4.6	
Tchefuncte River Madisonville	LA-22	Movable	Extensive	EM,SC,W	$25,000	2.32	
US11 @Lake Pontchartrain	US-11	Movable	Extensive	EM,SC,W	$6,000,000	4.02	
Yscloskey	LA-46	Movable	Extensive	EM,SC,W	$900,000	5.12	
Lake Pontchartrain	I-10	Fixed	Complete	D,SC	$30,000,000	4.02	
Pontchartrain Causeway	LA-Causeway	Fixed	Complete	D,SC	$1,500,000	2.77	
Mississippi							
David V.LaRosa	W.WittmanRd.	Fixed	Moderate	D	$60,000	7.5	
Biloxi Back Bay	I-110	Movable	Extensive	I	$2,500,000	6.22	
I-10 Pascagoula River	I-10	Fixed	Extensive	D,I	$5,800,000	4.57	
Popps Ferry	PoppsFerryRd.	Movable	Extensive	D,EM	$7,700,000	5.82	
Biloxi-Ocean Springs	US-90	Movable	Complete	D,EM	$275,000,000	6.58	
US-90 Bay St.Louis	US-90	Movable	Complete	D,EM	$276,000,000	5.58	
US-90 Henderson Point	US-90	Fixed	Complete	D	$1,900,000	7.01	

Note: — indicates no available surge estimate. Damage key: D = Deck Movement, EM = Electrical Mechanical, I = Impact, SC = Scour, W = Wind.

Table A5.5. Summary of fire hazard impacts on structures in Florida

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
Turnpike	12/14/2009	110070	Excessive heat exposure to various structural components of bridge including reinforced concrete substructure, prestresssed concrete beams and reinforced concrete bridge deck.	Extensive	$1,917,100
1&7	6/4/2008	130103	Element 109 - Prestressed concrete open girder, element 205 - reinforced concrete columns, element 234 - reinforced concrete cap and element 12 - concrete deck were damaged due to fire. Beams in spans 39 and 40 were severely damaged with major section loss and some complete strand loss. Spans 39 and 40 were severely damaged by fire and experienced widespread shallow spalling. Three columns in bent 40 had section loss and the bent cap had major fire damage.	Extensive	$4,500,000
1&7	3/28/2007	150124	Damage included substructure damage, replacement of one full span of the bridge and partial replacement of an adjacent span. Span 9 was replaced. The Pontis elements affected are: 12 - Bare Concrete Deck; 301 - Pourable Joint Seal; 302 - Compression Joint Seal; 303 - Assembly Joint Seal; 331 - Concrete Bridge Railing; 109 - Prestress Concrete Girder; 398 - Drain System; 310 - Elastomeric Bearing; 205 - Reinforced Concrete Column; and 234 - Reinforced Concrete Cap. A sign attached to the right barrier was impacted by the fire.	Extensive	$2,300,000
2	3/31/2009	260062	There is a 28 in. x 22 in. x 1/2 in. spall in the backwall of Abutment 1, adjacent to Beam 1-1. This appears to be due to a fire having been set on the cap and poses no problem at this time.	Slight	
2	4/1/2002	280012	There is a spalled area, 22 in. diameter x 1 in. deep, on top of the cap at Abutment 1, between Beams 1-3 and 1-4, due to heat from a fire that was built on top of the cap. Poses no problem at this time.	Slight	
2	10/18/2007	290076	CS 4 = There are areas of fire damage (charring) at the beam ends at abutment 1 due to fires on the caps (Refer to photo 7 on Addendum page A-6)	Slight	
2	12/3/2003	340042	There is a spalled area 22" diameter x 1" deep on top of the cap at Abutment 1 between Beams 1-3 and 1-4 due to heat from a fire that was built on top of the cap.	Slight	
2	11/24/2009	340042	There is a spalled area, 22 in. diameter x 1 in. deep, on top of the cap at Abutment 1, between Beams 1-3 and 1-4, due to heat from a fire that was built on top of the cap. Poses no problem at this time.	Slight	
2	5/7/2008	340043	Stringer 4-5 has minor fire damage in an isolated area in the top face at the near end. Also, there are deep splits in the left and right faces.	Slight	
2	4/12/2011	370023	Previously Reported Deficiencies: CS2: There is evidence of a past fire on the east side of the structure. There is discoloration and corrosion on the columns, diagonals, and base plates. (Refer to Photo 4).	Slight	
2	8/11/2005	380030	Six planks and 2m of top wale at the west fender, near the center navigation light, are fire damaged.	Slight	

Table A5.5. Summary of fire hazard impacts on structures in Florida (Cont'd)

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
5	1/21/2011	700017	On 01/21/2011, a traffic accident involving a gasoline tanker truck occurred at the east approach of this structure and twin structure, Bridge No. 700017. Extreme heat and severe fire damage occurred to the superstructure and substructure elements in Span 2, Span 3, at Pier 3 and Abutment 4. Refer to photos 1 through 3. Immediate action is being taken to replace the damaged superstructure and substructure components. The structure is scheduled to be open to the traveling public in approximately 35 days.	Extensive	
2	10/18/2010	720022	INCIDENTAL: Two fires have taken place adjacent to the west face of the crash wall at pier 2 between columns 1 and 2 & 3 and 4. The crash wall is spalled at each location for an area of approximately 8'L x 6' H x 1"D. No exposed steel seen. The fire appears to have been started with debris collected by vagrants. NEW. See photo 7.	Slight	
2	6/12/2002	720053	There are three fender panels with slight fire damage adjacent to Pier 9 (north fender panels 5, 6, & 8). See accident report dated 6/3/91.	Slight	
2	10/29/2003	720055	Column 6-3 has a 26" x 22" x 1" spall on the west face next to the groundline. This area is due to fire and poses no problem. Column 7-3 and 7-4 has two minor incipient spall on the bottom SE corner.	Slight	
2	3/17/2004	720076	Due to an electrical fire, the 3rd and 4th catwalk panels of the west fender have been damaged. The 3rd panel has fifteen 2 in. x 10 in. x 36 in. deckboards and two 10 in. x 3 1/2 in. x 12 ft. 6 in. deck stringers heavily damaged. The 4th panel has six deckboards and a 60 in. area of damaged deck stringers. See Photos 1 and 2 attached for views. Due to fire damage, temporary fender lights have been installed on the west fender. The electrical conduit and wiring has been destroyed by fire in a 17 ft. 6 in. area on the northwest fender.	Slight	
2	2/19/2009	720076	Approximately a 10 ft. diameter area of the inside Westbound travel lane, mid-span of Span 30, exhibits charred areas and random shallow surface spalls due to a vehicle fire. See Photo 1 attached for view. Poses no problem at this time. The inside barrier was also charred adjacent to this area.	Slight	
2	4/28/2011	720089	Due to vehicular impact, the right aluminum barrier rail in Span 3 was damaged in an 18 ft. long area and the roadway, approximately 40 ft. North of the North approach slab, exhibits fire damage, due to vehicular fire.	Slight	
2	9/29/2003	720107	PREVIOUS CORRECTIVE ACTION: The previously reported missing navigation light and fire-damaged conduit on the north end of the Pier 2 fender have been replaced and are currently in good condition.	Slight	
2	6/30/2005	720107	At Pile Clusters 2 and 3, the wales along the east face are chard from past fire damage.	Slight	

Table A5.5. Summary of fire hazard impacts on structures in Florida (Cont'd)

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
2	5/11/2009	720137	The west face of Bent 2 cap has staining of black soot from the vehicle fire under Span 1. The outside face of the left and right barriers in Span 1 are covered with black soot from the vehicle fire under Span 1. End Bent 1 cap is covered with black soot from the vehicle fire under Span 1. The underside of all 11 slab units in Span 1 is covered with black soot from the vehicle fire that took place under Span 1. Refer to photo 2. The only slab units showing any damage as a result of the fire are Slab Units 1-9, 1-10 and 1-11.	Slight	
2	4/24/2008	720165	INCIDENTAL: Two fires have taken place adjacent to the west face of the crash wall at pier 2 between columns 1 and 2 & 3 and 4. The crash wall is spalled at each location for an area of approximately 8'L x 6' H x 1"D. No exposed steel seen. The fire appears to have been started with debris collected by vagrants. NEW. See photo 7.	Slight	
2	6/11/2008	720170	Beam 1-7, East face and Beam 1-8 West face, smoke stained due to fire at Abutment 1. Charred area (apparently due to vehicular fire), right emergency lane, Span 5 which has surface scaling and delamination in 10 ft. diameter area. Smoke stained (due to fire) between Beams 1-7 and 1-8 over Abutment 1, extending out to intermediate diaphragm. Beam 1-7, East face and Beam 1-8 West face, smoke stained due to fire at Abutment 1.	Slight	
2	6/9/2004	720172	The cap at Abutment 1 has minor fire damage between Beams 1-3 and 1-4. Beam 1-3 over Abutment 1 has minor spalling in the area of the diaphragm and poured beam end due to fires set on top of the abutment cap. Span 2 has several beams which have been impacted by overheight vehicles. The deck underside of Span 1 between Beams 1-3 and 1-4 is heavily sooted due to fires being built on top of Abutment 1. Beam 1-3 over Abutment 1 has minor spalling in the area of the diaphragm and poured beam end due to fires set on top of the abutment cap. Span 2 has several beams which have been impacted by overheight vehicles. The cap at Abutment 1 has minor pop-out spalls due to fire damage	Slight	

Table A5.5. Summary of fire hazard impacts on structures in Florida (Cont'd)

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
2	12/21/2009	720226	The base of the right barrier in Spans 2 and 3 exhibit up to moderate size pop-out spalls due to vehicular fire, up to 11 in. high x 78 ft. long area. See Photos 2 and 3 attached for view. Due to vehicular fire, deck top in Spans 2 and 3 exhibit minor pop-out spalls in a 22 ft. x 9 ft. area in the right emergency lane. These spalls are up to 18 in. long x up to 2 in. x 1/4 in. deep. See Photos 1 and 3 attached for typical/overall view. The base of the right barrier in Spans 2 and 3 exhibit up to moderate size spalls, up to 11 in. high along a 78 ft. long area. (See Photo 1 for view). These conditions are due to a car fire on the structure.	Slight	
2	11/24/2009	720239	Power resistors mounted to the top plate are heating and causing burning on insulation of the adjacent conductors and terminal block, creating a fire hazard.	Slight	
2	4/20/2007	720249	The south shoulder in Spans 103, 114 and 200 has up to 8ft. x 6ft. x 1in. fire damage spalls. Refer to photo 2 on page 20 in the Appendix. The previously noted similar spalls in Spans 102 and 104 were not observed during this inspection.	Slight	
2	4/29/2004	720263	CS3> The anchor bolt nuts of the handrail stanchions are missing in random locations. The aluminum handrails exhibit numerous areas of gunfire related damage.	Slight	
2	3/18/2003	720333	There is a 4' x 1 3/4" x 1/2" spall on the west side of Column 2-6 due to vehicular impact and fire. See Photo 1 for view of spall attached. There is a 39" x 12" x 4" spall with 2" of exposed reinforcing steel in left bottom corner of the cap at Pier 2 between Columns 2-5 and 2-6 (used ZRC). This spall appears to be due to vehicle fire below cap. See Photo 2 for view of spall attached. There is a 48 in. x 19 in. x 1/2 in. spall on the West side of Column 2-6 due to vehicular impact and fire.	Slight	
2	8/18/2008	720343	Several shallow surface spalls exist in deck top of Span 119 in a 5 ft. x 3 ft. area in the left emergency strip, the result of vehicular (motorcycle) fire damage. See Photos 1 and 2, attached. Concrete sounded with a hammer in this area and returned a solid report. This condition poses no problem at this time. The following other conditions were noted in the deck top: The north shoulder in Spans 7, 27, 105, 119 and 149 has up to 6ft. long x 4ft. wide x 1in. deep fire damage spalls. Refer to photo 2 on page 18 in the Appendix. There is minor damage to the left bridge rail in Span 41 due to a	Slight	
2	8/3/2005	720370	Light fire damage was noted in the deck top in the right emergency strip of Span 14 near Bent 14 where a minor shallow spall exists, but this and the surrounding area provided a solid report when sounded with a hammer and poses no problem at this time.	Slight	

Table A5.5. Summary of fire hazard impacts on structures in Florida (Cont'd)

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
2	6/9/2010	740129	CS3: The lower northeast edge of pier 5 cap has a 10ft x 10in x 4in delamination/spall with exposed steel due to fire damage which also damaged the column -NEW. Refer to Photo 10. P3 WO The north and east faces of column 5 have an 8ft x 6ft x 3in fire related spall with exposed steel. The cap is also damaged -NEW. Refer to Photo 8. P3 WO	Moderate	
2	12/7/2004	760002	Column 27-1, two spalls, averaging 10" x 10" x 3" deep on the south face. These spalls are located within 8 ft. of the groundline and ZRC was used. They appear to be due to firearm discharge (see Photos 12 and 13 for typical view).	Slight	
2	3/10/2011	780035	North face, bottom edge of Column 5-1, has a 28 in. x 20 1/2 in. x 2 in. spall due to fire damage. Poses no problem.	Slight	
2	6/15/2010	780097	Pile 15-4 has a 2 ft. 3 in. x 13 in. incipient spall, Northeast corner, with two minor spalls up to 1 ft. 9 in. x 8 in. x 5/8 in. deep due to heat from a fire.	Slight	
2	3/20/2006	780099	North face, bottom edge of Column 5-1 has a 28" x 20 1/2" x 2" spall due to fire damage. Poses no problem.	Slight	
6	10/3/2002	870363	The tanker rolled off the exit ramp and exploded upon impact in the parking lot of the Miami-Dade County Women's Detention Center, burning a least 10 cars. The incident caused extensive fire damage to the substructure, including girders, underdeck, columns, caps, bearings, expansion joints and drainage gutters.	Extensive	$477,794
6	10/3/2002	870364	The tanker rolled off the exit ramp and exploded upon impact in the parking lot of the Miami-Dade County Women's Detention Center, burning a least 10 cars. The incident caused extensive fire damage to the substructure, including girders, underdeck, columns, caps, bearings, expansion joints and drainage gutters.	Extensive	$477,794
6	11/6/2006	870439	The gasoline tanker struck the west barrier and fell onto the parking lot below causing a severe explosion. A large fire ensued which was extinguished in approximately 20 minutes. There were intermittent concrete spalls with no expose steel on the girders, underdeck, columns, caps, bearings, expansion joints and drainage gutters.	Extensive	$375,000
6	10/2/2005	900101	On 10/03/2005 an accident occurred on the Seven Mile Bridge with a fuel tank truck damaging section of the bridge deck at spans #14 and 15. The concrete deck surface along bridge segments 14-7 & 14-8 along with 15-1 presented intermitting delamination throughout the deck. Including concrete delamination to the bridge railing system along both sides of span 14-7, 14-8 & 15-1. All the concrete delamination was as a result of the intense heat along the interior faces of the concrete railing and the deck surface.	Extensive	$89,852
Turnpike	1/12/2010	920027	Excessive heat exposure to various structural components of bridge including reinforced concrete substructure, prestresssed concrete beams and reinforced concrete bridge deck.	Extensive	
4		940133	Repair 3 beams of Span 3 (beams 3-3, 3-4 and 3-5) with Carbon Fiber and restore the underside deck of 2 bays (possible 3rd bay if determined area is affected). MOT provided by Ranger Construction	Moderate	$410,000
2	1/19/2005	720061	Minor fender damage; impact & fire	Slight	
2	9/29/2003	720107	Fender damage & navigational light damage	Slight	

Table A5.5. Summary of fire hazard impacts on structures in Florida (Cont'd)

District	Hazard Date (or Inspection date)	Bridge ID	Description of Damage or Repair Needs	Level of Damage	Costs of Repair
2	2/22/2003	720395	Fire damage resulted in spalls to Beams 2-5 and 2-6. West face of Beam 2-5 exhibits 13" x 2" x 1/2" spall on upper flange and 58" x 6" x 1 1/2" spall on lower flange. See Photo 1 attached.	Slight	
2	1/15/2002	720400	There is a 3.0 m x 2.0 m area at N. end of Beam 4-3 over Abutment 5 that has heavy soot and paint peeling which was caused by a fire at top of slope. The diaphragm over Abutment 5 between Beams 4-2 & 4-3 has heavy soot & peeled paint due to fire.	Slight	
2	2/25/2002	720503	Box Girder 1, over Abutment 1 has approximately 37.1 square meters of paint which has blistered and peeled due to fire damage inside of box. There is no active corrosion within the blistered area. Several anchor bolts for the box girders are bent laterally and resting against	Slight	
2	4/10/2003	720506	Column 2-4 also has a spall at the groundline, north side, measuring 38" x 28" x 2 1/2" (no exposed steel) which appears to be due to fire, posing no problem at this time.	Slight	
2	3/7/2011	720518	Vehicular fire damage in Span 31, left side, right Southbound and emergency lanes adjacent to Bent 31, in a 20 ft. x 8 ft. 6 in. area. See Photo 1 attached for overall view. There are four areas up to 6 ft. x 3 ft. x up to 1 in. deep delaminated/pop-out spalls, no exposed steel. See Photo 2 attached for view.	Moderate	
2	3/19/2003	720535	There are several minor surface spalls in right lane of Spans 2, 3, and 4, possibly due to some type of object being dragged across the structure. There is a 12' x 6' x 3/4" pop-out spall in the deck in Span 3 due to fire damage. There are no cracks or exposed steel due to the fire, and poses no problem at this time. There is an 18 ft. x 15 in. x up to 1 in. spall in the right barrier wall of Span 3 due to fire damage, but poses no problem at this time.	Slight	
2	11/3/2003	720553	Both 1 1/2" fiber optic conduit mounted to the north abutment cap was burnt and destroyed in an 8" area. See Photo 1 for view. There was also minor surface spalls on north abutment cap. This was due to fire beneath structure. Repairs have been made by ITS, therefore, no repairs are being requested.	Slight	
2	3/20/2007	720670	Beams 10-1 thru 10-6 at the abutment has vulgar graffiti, located in the webs and the diaphragms. See Photo 1 attached for typical view. Vagrants are living under the structure and are utilizing fires in certain areas.	Slight	
2	9/4/2005	720692	Beam 17-2 exhibits 1 in. diameter minor pop-out spalls freckling the surface of the lower flange, 10 ft. North of Bent 17, due to a vehicular fire. This incident was inspected on 09/04/05. Posing no problem at this time. See Photo 4 attached for view.	Slight	
2	2/4/2008	729002	The beams in Spans 1, 2 and 11 are blackened from fires. No obvious structural damage was observed as a result of the fires. A stolen car which was set on fire at the north end of Pier 2 blackened the underside of Spans 1 and 2. Fires set by transients at the east end of the bridge have blackened the underside of Span 11. No obvious signs of structural damage were seen at either of the locations.	Slight	

Table A5.6. Summary of bridges (non-Florida) damaged in fire hazards

Bridge and Location	Date	Fire Event Type	Damage Description	Estimated Cost of Damage	Notes/References
Bridge over I-75 near Hazel Park, Michigan	7/15/2009	Gasoline tanker struck an overpass on I-75.	Complete collapse of bridge, which fell on freeway below.	N/A	Source: Kodur et al. (2010)
Big Four Bridge, Louisville, Kentucky	5/7/2008	Electrical problem in the lighting system; took 2 1/2 hours to control	Minor structural damage, resulting in large amount of debris on the bridge.	N/A	Source: Kodur et al. (2010)
I-80 - 880 interchange in Oakland, California	4/29/2007	Gasoline tanker crashed.	A 228-m section of the interchange collapsed.	N/A	Source: Kodur et al. (2010)
Bill Williams River Bridge, Arizona	7/28/2006	Gasoline tanker overturned.	Beam, concrete girders, and underside of deck were damaged.	N/A	Source: Kodur et al. (2010)
I-95 Howard Avenue Overpass in Bridgeport, Connecticut	3/26/2003	Car struck a truck carrying 8,000 gallons of heating oil.	Collapse of the south bound lanes and partial collapse of the northbound	N/A	Source: Kodur et al. (2010)
I-80W-I-580E ramp in Emeryville, California	2/5/1995	Gasoline tanker crashed.	Deck, guardrail, and some ancillary facilities were damaged	N/A	Source: Kodur et al. (2010)
Puyallup River Bridge, Washington	12/11/2002	A railroad tanker collision caused a fire under a prestressed girder bridge	The bridge experienced fire damage to the south face of both columns at Pier 9. Fire also damaged all 15 lines of girders in Span 8.	$870,000	Estimated cost of repair and replacement was both approximately $870,000. Source: Stoddard (2004)
East- and westbound bridges carrying I-465 traffic over a ramp for I-69.	10/22/2009	A truck hauling a trailer of liquefied propane lost control and crashed beneath the east- and westbound bridges.	The explosion and subsequent fire did not negatively impact the overall load distribution nor adversely alter the behavior of the bridge.	N/A	Source: Brandt et al. (2011)
I-65 North overpass bridge in Birmingham, Alabama	7/5/2002	A vehicle crashed into a tanker truck that was carrying 9,000 gallons of fuel		$3,396,421	Bridge Replacement: Contact Amount = $2,096,421 plus Bonus. Source: Hitchcock et al. (2008)
I-20/59 North bridge at the interchange of I-65 and I-20/59	10/21/2004	A 9,000 gallon fuel tanker truck crashed under the I-20/59 north bridge at the interchange of I-65 and I-20/59		$6,743,000	Affected approximately 245,000 vehicles per day. Estimated daily user cost of $200,000. Source: Hitchcock et al. (2008)

Table A5.7. Cost data from MMS on bridge accident repairs

Site No.	Date Reported	Bridge ID	ACT	Unit of Measure	Activity Description	Estimated Units	Total Cost ($)	Date Completed	Type of Bridge Element
8886001	3/30/1994	500052	888	MH	Repair spalls on beams with dry pack gun	7	8,530.13	4/18/1994	Beam
8886002	3/30/1994	500054	888	MH	Repair spalls on beams with dry pack gun	3	4,199.71	4/6/1994	Beam
8886001	1/11/1995	720068	888	MH	Repair the damage caused by barge hit at fender.	20	4,224.89	3/15/1995	Fender
8886001	4/11/1995	030078	888	MH	Replace damaged handrail post NO. 2 on the right side of span 4.	15	113.27	8/15/1996	Railing/Handrail
8886002	4/12/1995	290071	888	MH	BRIDGE ACCIDENT.	100	6,041.17	7/10/1995	Unknown
8886006	5/31/1995	260056	888	MH	BRIDGE ACCIDENT.	100	0.00		Unknown
8886003	5/31/1995	290071	888	MH	BRIDGE ACCIDENT.	100	0.00		Unknown
8886007	5/31/1995	290071	888	MH	BRIDGE ACCIDENT.	100	377.38	5/26/1995	Unknown
8886005	5/31/1995	320039	888	MH	BRIDGE ACCIDENT.	100	0.00	12/28/1995	Unknown
8886009	5/31/1995	320039	888	MH	BRIDGE ACCIDENT.	100	51.43	5/21/1995	Unknown
8886004	5/31/1995	320046	888	MH	BRIDGE ACCIDENT.	100	0.00		Unknown
8886008	5/31/1995	320046	888	MH	BRIDGE ACCIDENT.	100	93.77	5/21/1995	Unknown
8886003	9/11/1995	480061	888	MH	REPAIR BEAMS 3-1 3-2 3-3 WITH DRI PACKET	1	832.35	1/9/1996	Beam
8886001	3/1/1996	100138	888	MH	Repair the damaged area caused by vehicle inpact span 2 beam 1.	4	1,059.98	4/18/1996	Beam
8886010	4/24/1996	290082	888	MH	Splice the broken strands in the bottom flange of Beam VI Span 3.	6	426.50	6/20/1996	Beam
8886011	4/24/1996	290082	888	MH	Repair the spalled area in the bottom flange of Beam VI Span 3.	8	16.64	12/16/1996	Beam
8886017	5/30/1996	720060	888	MH	Repair the spalled areas on the handrail.	24	0.00	7/1/1996	Railing/Handrail
8886019	5/30/1996	720061	888	MH	Replace aluminum handrail beams 7.3 ft in Span 5 and 8.4 ft in Span 6.	4	299.54	7/24/1996	Railing/Handrail
8886015	5/30/1996	720071	888	MH	Patch Handrail Posts east side Spans 3 4 and 5.	15	382.66	7/2/1996	Railing/Handrail
8886016	5/30/1996	720153	888	MH	Remove fire soot on west barrier Spans 15 and 16.	8	750.00	10/29/1996	barrier
8886014	5/30/1996	720186	888	MH	Repair the spalled areas on Beam I and II in Span 2.	6	454.11	6/12/1996	Beam
8886013	5/30/1996	720206	888	MH	Repair the spalled areas of Beam I II and V in Span 3.	80	350.00	10/29/1996	Beam
8886018	5/30/1996	720307	888	MH	Repair the spalled areas on Beams IX and X in Span 2.	12	1,646.00	10/31/1996	Beam
8886012	5/30/1996	780070	888	MH	Repair 16.1LF damag.handrail R.side Span 5. Repair spalled/crack.area deck ovrhang und.2nd post Pr5.	160	16.73	2/6/1997	Handrail/deck
8886022	6/18/1996	720072	888	MH	Repair spalled concrete under Post 2 in Span 2 on left side.	6	271.36	7/24/1996	RailPost
8886023	6/18/1996	720072	888	MH	Replace concrete Posts 2/ 3/ and 4 in Span 2 and concrete Handrail Beams between Posts 2/ 3/ 4.	12	0.00	4/16/1996	Handrail
8886021	6/18/1996	720175	888	MH	Patch spalled area in Span 2 over Pier 3 in Lane 3 of northbound roadway.	6	101.00	8/25/1996	Unknown
8886020	6/18/1996	780007	888	MH	Repair Post 3 on the right side of Span 4. Also repair Post 2 on the right side of Span 5.	15	357.21	7/25/1996	Unknown
8886024	6/20/1996	380011	888	MH	Repair spalled area in the sidewalk on the left side over Abutment 4.	10	282.46	6/25/1996	Unknown
8886025	6/20/1996	380011	888	MH	Repour the guardrail anchor block at the northwest end of structure.	25	1,056.25	6/27/1996	Guardrail
8886026	6/20/1996	380011	888	MH	Seal the vertical crack in the northwest wing wall and back wall at Abutment 4.	2	189.71	6/25/1996	Wingwall/Backwall
8886002	6/27/1996	080021	888	LF	Repaired spalled area on beam as the result of vehicle accident.	40	0.00		Beam
8886028	7/15/1996	270057	888	MH	Repair spalled areas in Beam VII/VIII and IX/Span 2.	40	286.33	8/1/1996	Beam
8886029	7/15/1996	380053	888	MH	SEE INSTRUCTIONS.	40	14.55	11/4/1996	Unknown
8886030	7/15/1996	720103	888	MH	Repair the spalled beam Span 2.	12	145.02	12/10/1996	Beam
8886031	7/15/1996	740054	888	MH	Repair spalled areas on handrail post in Spans 11/12/13 and top rail in Span 12.	40	6,369.35	11/7/1996	Handrail
8886032	7/15/1996	740055	888	MH	Repair spalled area in deck overhang/replace 2 destroyed handrail posts and beams/repair spall.	40	0.00	6/26/1996	Deck/RailPost/Beam
8886033	7/15/1996	780056	888	MH	Replace 9 ft. of aluminum handrail/replace 2 sheared bolts/patch spall on right side/Span 13.	10	635.23	10/8/1996	Handrail
8886034	7/15/1996	780096	888	MH	Pressure wash Beams and Deck (underside) in Span 2.	20	1,276.80	10/29/1996	Beam/Deck
8886035	7/30/1996	380011	888	MH	Repair the gouged areas in asphalt surface of deck and south approaching roadway.	10	31.65	8/7/1996	Deck/Approach roadway
8886038	7/30/1996	720163	888	MH	Repair spalled area on handrail.	8	278.09	2/2/1997	Handrail
8886037	7/30/1996	720309	888	MH	Repair the spalled areas on the beams, Span 2.	10	550.00	2/27/1997	Beam
8886036	7/30/1996	720361	888	MH	Repair the spalled bottom flange Beam IV, Span 3.	8	814.64	10/21/1996	Beam

Table A5.7. Cost data from MMS on bridge accident repairs (cont'd)

Site No.	Date Reported	Bridge ID	ACT	Unit of Measure	Activity Description	Estimated Units	Total Cost ($)	Date Completed	Type of Bridge Element
8886042	8/1/1996	720242	888	MH	Patch spalled beams, Span 3.	6	292.10	11/19/1996	Beam
8886041	8/1/1996	720315	888	MH	Patch the spalled beams, Span 3.	6	469.42	11/18/1996	Beam
8886043	9/9/1996	390001	888	MH	Repair damaged curb and handrail on right side of Span 22.	80	174.25	10/28/1996	Handrail
8886045	9/9/1996	720156	888	MH	Patch spalls under posts 1-3/repair spalled post and handrail, west side, Span 22.	56	2,110.70	12/16/1996	Handrail
8886044	9/9/1996	720272	888	MH	Repair damaged handrail and deck overhang on left side of Spans 1 and 2.	100	0.00	8/1/1996	Handrail/deck
8886004	10/9/1996	100177	888	MH	Repair spalls on beams 2-5 and 2-6, midspan, and beam 1-1, south end.	16	1,117.22	10/25/1996	Beam
8886003	10/9/1996	100183	888	MH	Repair the damaged area caused by vehicle inpact span 2 beam 1.	4	1,405.78	6/16/1997	Beam
8886005	10/9/1996	100189	888	LF	Repair Spalls with exposed Reinforced strands on beams span 2	70	569.79	10/24/1996	Beam
8886047	12/3/1996	290039	888	MH	Repair spalled area on east side of Beam 3-7 and 3-9.	100	100.91	1/27/1997	Beam
8886048	12/3/1996	290059	888	MH	Repair spalled area on east side of Beam 3-1.	100	50.46	1/27/1997	Beam
8886046	12/3/1996	720069	888	MH	Repair cracked/damaged handrail left side Span 6/Spans 7/8 S. side.	120	1,848.30	3/12/1997	Handrail
8886052	12/16/1996	290061	888	MH	Repair as needed spalled beams in Span 3 of Bridge Numbers 290061 and 290064.	40	786.52	2/11/1997	Beam
8886053	12/16/1996	720022	888	MH	Repair damaged aluminum pedestrian handrail on west side of Span 16.	20	429.30	1/7/1997	Handrail
8886050	12/16/1996	720306	888	MH	Repair cracked/spalled handrail, east side, span 2.	20	88.69	4/15/1997	Handrail
8886051	12/16/1996	740008	888	MH	Repl. deter. nuts holding E/W guide to lower chord and pin holding wedge in guide NW corner	10	71.25	4/22/1997	Truss
8886049	12/16/1996	780007	888	MH	Repair/repour concrete handrail over Bent 4.	40	8,045.78	2/19/1997	Handrail
8886006	12/17/1996	100223	888	MH	Repair damaged handrail caused by vehicle impact. SB South end of Structure	1	1,470.73	2/24/1997	Handrail
8886055	2/10/1997	720177	888	MH	Splice severed strand, patch spalled areas and surface patch all cracks in Beams 1-1 and 1-2.	81	1,054.54	3/11/1997	Beam
8886056	2/10/1997	720216	888	MH	Repair the spalled areas on Beams 3-1, 3-2 and 3-3 with exposed cable.	40	3,500.00	5/13/1997	Beam
8886057	2/10/1997	780045	888	MH	Repair spalled areas on Beams 2-1, 2-2, 2-3	100	1,250.00	4/29/1997	Beam
8886058	2/10/1997	780090	888	MH	Re-secure vertical timber at south end of east fender.	4	735.67	2/24/1997	Fender
8886059	2/10/1997	780090	888	MH	Replace missing navigational clearance gauge and repair broken conduit at south end of east fender.	6	51.16	3/26/1997	Fender
8886060	3/31/1997	260079	888	MH	Repair spall on Beams 2-8 thru 2-12.	80	1,258.45	5/29/1997	Beam
8886061	3/31/1997	720022	888	MH	Reweld aluminum handrail posts (2) to base plates east side, Span 16 over pier 16.	10	33.21	7/28/1997	Handrail
8886002	4/29/1997	160021	888	3	Repair collision damage on posts 4 & 5 in span 1 left side.	3	240.44	11/30/1997	RailPost
8886062	4/29/1997	260079	888	MH	Repair the spalled areas on the beams with exposed cables.	40	193.49	9/8/1997	Beam
8886064	4/29/1997	720022	888	MH	Replace the damaged aluminum handrail/brackets Span 16, left side.	8	231.60	6/19/1997	Handrail
8886063	4/29/1997	720496	888	MH	Patch the spalls on the west side and underside of the flat slab, Span 6.	32	1,000.00	11/10/1997	Slab
8886065	7/3/1997	720022	888	MH	Reweld handrail posts to baseplates right side of Span 16 between L-3 and L-4.	10	169.96	2/26/1998	Handrail
8886068	7/3/1997	720022	888	MH	REPLACE THE DAMAGED HANDRAIL IN SPAN 16 BETWEEN L-6AND L-7 ON THE WEST SIDE.	10	228.49	9/9/1997	Handrail
8886066	7/3/1997	720069	888	MH	Repair concrete Posts 6 and 7 in Span 6. Patch spalls on concrete Post 8.	40	3,583.10	8/28/1997	Post
8886067	7/3/1997	720343	888	MH	Repair spalled barrier over Bent 39.	20	310.45	9/11/1997	barrier
8886069	7/30/1997	780056	888	MH	Repair parapet,replace sheared bracket and section of handrail,Span 17,replace one section handrail,	40	680.95	1/21/1998	Parapet/Handrail
8886070	9/30/1997	290061	888	MH	Repair the spalled area in the bottom flange of Beam 2-1 and Beams 2-3 through 2-10.	40	376.80	12/8/1997	Beam
8886071	9/30/1997	290064	888	MH	Repair the spalled area in the bottom flange of Beams 2-8, 2-9 and 2-10.	40	156.27	12/8/1997	Beam
8886074	10/21/1997	720071	888	MH	Repair cracked/spalled sidewalk west side/Repair/repour damaged bridge rail Spans 1 and 2.	20	1,288.16	11/6/1997	Sidewalk/Bridge rail
8886075	10/21/1997	720122	888	MH	Repair spalled areas on south bottom flange of Beams 2-1 through 2-3.	10	423.44	12/17/1997	Beam
8886076	10/21/1997	729001	888	MH	Shift north end of Span 3 back into proper position. Repair spall on east end of cap over Bent 3.	40	2,654.42	10/29/1997	Cap

Table A5.7. Cost data from MMS on bridge accident repairs (cont'd)

Site No.	Date Reported	Bridge ID	ACT	Unit of Measure	Activity Description	Estimated Units	Total Cost ($)	Date Completed	Type of Bridge Element
8886077	10/21/1997	780067	888	MH	Repair spalled areas on beams.	40	200.47	12/4/1997	Beam
8886078	12/2/1997	723450	888	MH	Install two bolts either side of damaged bolt/reattach loose ground wire/repl bottom strut.	80	538.83	8/11/1998	Unknown
8886079	12/8/1997	720069	888	MH	Repair damaged right handrail in Spans 5 and 6.	80	1,711.35	3/4/1998	Handrail
8886080	12/8/1997	720184	888	MH	Repair the damaged right handrail and spalled deck overhang in Span 2.	80	4,943.87	2/23/1998	Handrail/Deck
8886081	12/8/1997	780074	888	MH	Repair damaged handrail and parapet on left side of Span 7.	40	569.09	12/31/1997	Handrail/Parapet
8886082	1/27/1998	720156	888	MH	Repair or replace vertical clearance tide gauge northeast fender.	12	77.41	11/3/1998	Fender
8886083	4/16/1998	260057	888	MH	Repair the spalls in concrete Beams 2-1 through 2-5.	20	900.00	12/2/1998	Beam
8886084	4/16/1998	260082	888	MH	Repair the spalls in concrete Beams 2-6 through 2-8.	20	200.00	12/2/1998	Beam
8886087	4/16/1998	720069	888	MH	Repair 8th handrail post & deck overhang base of post Span4, right side.	80	222.95	9/14/1998	Handrail/deck
8886085	4/16/1998	740031	888	MH	Repair sidewalk beneath Handrail Post 7 Span 2, west side.	40	949.02	10/14/1998	Sidewalk
8886086	4/16/1998	740031	888	MH	Repair Handrail Posts 6, 7 and 8 and rail, Span 2 west side.	40	663.39	9/8/1998	Handrail/rail
8886088	7/13/1998	270047	888	MH	Repair spalled area on Beam 2-1, 2-2 and 2-3.	40	603.69	9/9/1998	Beam
8886089	7/13/1998	270057	888	MH	Repair spalled areas on Beams 2-6 through 2-9.	20	55.12	9/14/1998	Beam
8886090	7/30/1998	370009	888	MH	Repair spalled barrier wall NW guardrail attachment and destroyed slope pav ement NE radius section.	60	18.42	10/28/1998	Barrier Guardrail/Slope Protection
8886093	9/15/1998	720174	888	MH	Repair all the spalled beams in Span 2.	160	2,580.00	11/5/1998	Beam
8886092	9/15/1998	720177	888	MH	Repair the spalls on Beams 2-22, 2-23 and 2-24.	40	2,900.00	12/7/1998	Beam
8886091	9/15/1998	720206	888	MH	Repair spalled areas with exposed cable on Beams 2-1 and 2-5.	40	5,373.79	7/28/1999	Beam
8886096	10/23/1998	720011	888	MH	Repair the damaged barrier in median on the bridge in Span 25.	10	6,460.00	3/2/1999	Barrier
8886095	10/23/1998	720069	888	MH	Repair the damaged post on the south side of Span 6, (Post 7).	10	1,350.12	5/26/1999	Post
8886094	10/23/1998	720083	888	MH	Repair the spalled area on the beam with exposed cable (Beam 2-10).	40	272.44	6/15/1999	Beam
8886097	10/23/1998	760028	888	MH	Repair the spalled areas in the damaged barrier wall in Spans 1, 7, 8 and 9.	10	2,050.00	5/21/1999	Barrier
8886098	11/13/1998	720079	888	MH	Spot paint underside of all bottom flanges and north side of web on Beam 2- 1.	20	288.06	7/22/1999	Beam
8886099	11/13/1998	720509	888	MH	Repair the spalls in the left Barrier, Spans 64 and 65.	20	1,700.00	1/6/1999	Barrier
8886100	3/19/1999	720201	888	MH	Repair the aluminum and concrete handrail in Span 2, west side.	1	1,206.55	7/7/1999	Handrail
8886101	6/21/1999	780018	888	MH	Splice broken cables and patch spalls in Beams 3-2 & 3-5. P=2	1	0.00	7/7/1999	Beam
8886103	6/29/1999	720371	888	MH	Repair the spalled barrier on the right side of Spans 8, 9, 10 and left side of 13. p=2 (888 ADR)	1	4,200.00	9/29/1999	Barrier
8886104	6/29/1999	780075	888	MH	Repour the handrail, left side, Span 6. (888 ADR) P=2	1	6,091.67	9/27/1999	Handrail
8886105	7/20/1999	720177	888	MH	Repair spalled area 1.5 m x 0.3 m x 0.03 m with exposed steel, Beam 2-24. p=3 (888 ADR) El. 109	1	3,112.06	5/3/2000	Beam
8886001	3/29/2001	790102	888	MH	Clean and patch spalls with exposed prestress strands in span 2 BEAM V SPAN 3 BEAM I.	1	553.56	4/19/2001	Beam

A5.1. References

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Appendix A6: Vulnerability to storm surge and wave loading

While hurricanes are known for damage to infrastructures due to the accompanying strong winds, coastal structures are particularly vulnerable to damage from storm surges and wave loading. This section presents the effort on this research study to evaluate coastal bridges for vulnerability to storm surge and wave loading damages during hurricanes. The results can be regarded as preliminary as no validated models were developed.

In order to study the impact and risk of coastal bridges susceptible to hurricane storm surges, it was be necessary to review the current pertinent design and evaluation guidelines. Sheppard and others have developed methodologies (AASHTO Guide Specs) to assess coastal bridges for vulnerability to hurricane forces, especially for Florida bridges. Sheppard (2011) presented a useful model using 52 bridges in a pilot study conducted in the Tampa, Florida area (FDOT District 7). These are results from an ongoing project on AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms (done by Modjeski & Masters, Inc. and OEA). AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms requires that:

> *"the vertical clearance of highway bridges should be sufficient to provide at least 1 ft. of clearance over the 100-yr. design wave crest elevation, which includes the design storm elevation."*

Sheppard (2011) demonstrated that a current AASHTO specification on storm surges and waves has three levels of conformance (Levels I, II, and III).

- Level I: simplest and least accurate (conservative); based on wind speed, surge height, etc.
- Level II: uses improved data through simulation.
- Level III: Used of advanced numerical simulation.

Hayes (2008) in a master's thesis work applied the same model to three bridges in Delaware but uses the vertical clearance as the only criterion. A proprietary computer program was developed by OEA to evaluate equations, generate force and moment data and develop parametric equations (Figure A6.1). The following input is required to apply the force/moment equations: superstructure type, dimensions, elevation; and design water elevation, wave height and period. The output is obtained as follows: maximum vertical force and associated horizontal force and overturning moment; maximum horizontal force and associated vertical force and overturning moment; maximum overturning moment and associated; vertical and horizontal forces.

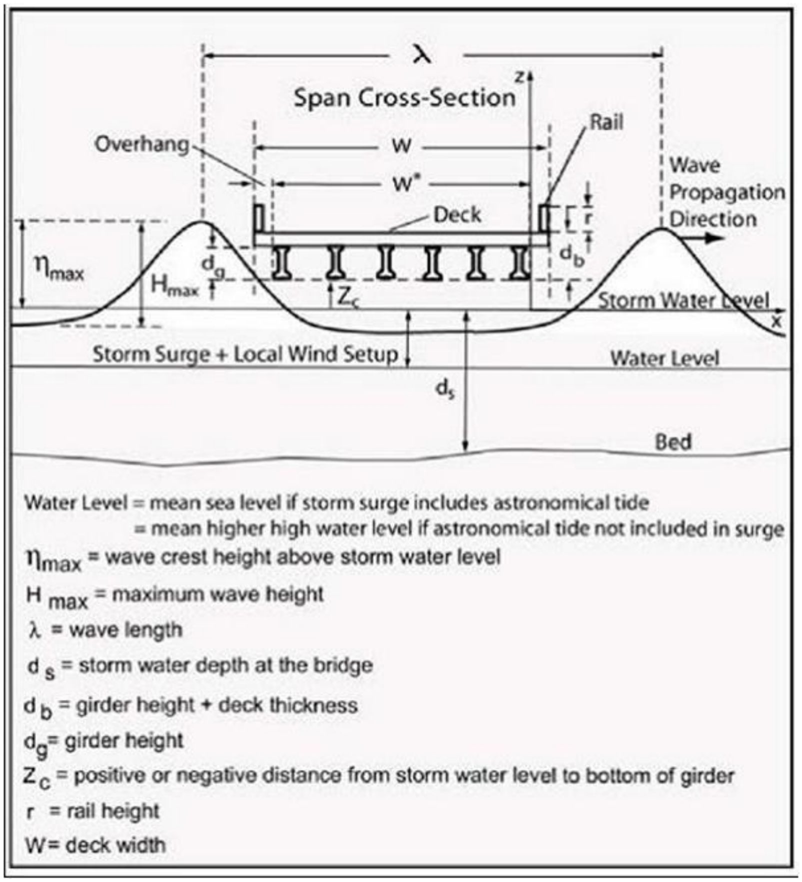


Figure A6.1. Force components during storm surge and wave loading a bridge superstructure.

According to Sheppard (2011), the current FDOT Criteria for Classifying Bridge Vulnerability to Coastal Storms are as follows:

- "Extremely Critical" – Design to withstand surge/wave forces to the strength limit state (1.75 load factor)
- "Critical" – Design to withstand surge/wave forces to the extreme event limit state (no load factor)
- Or "Non-Critical."

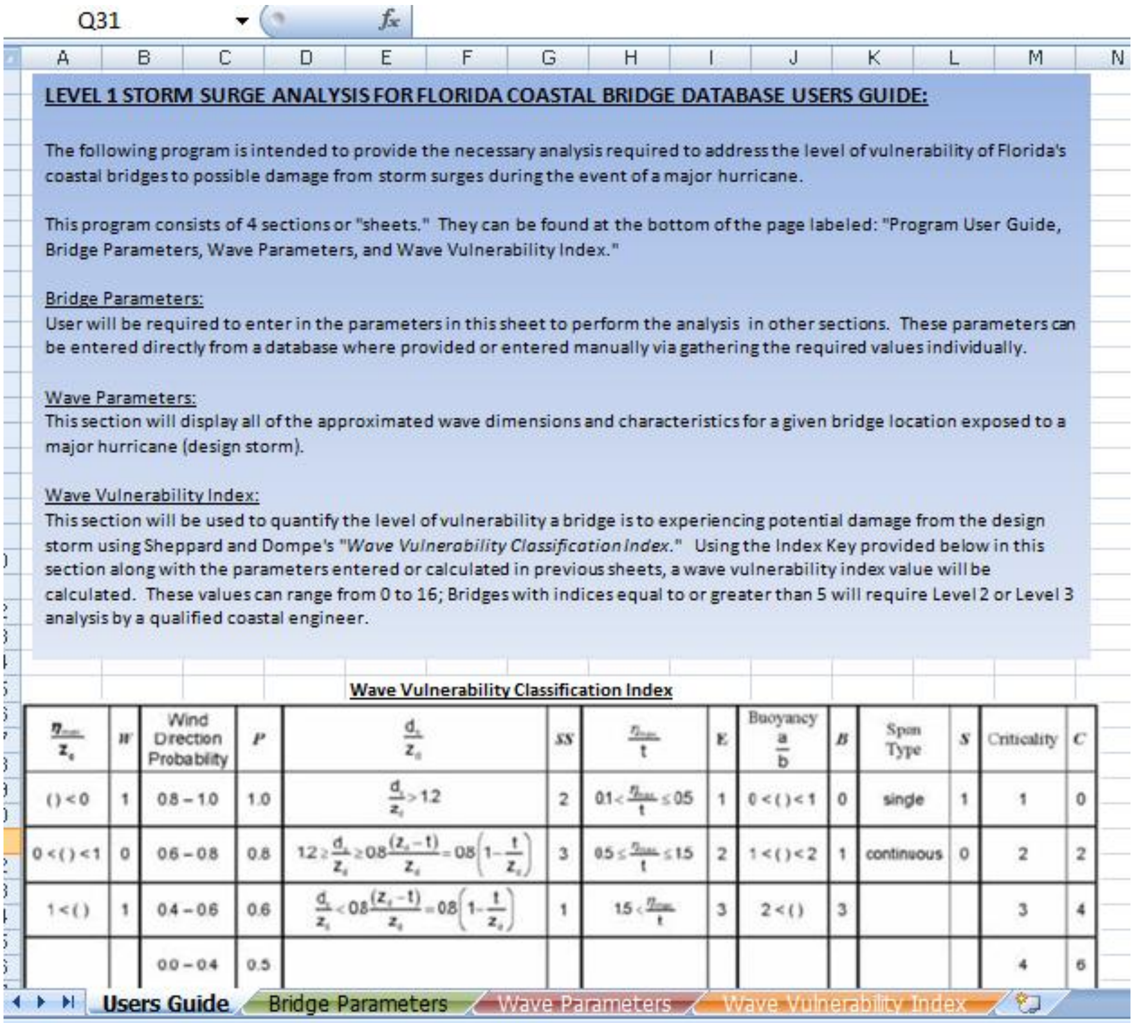
Sheppard and Dompe (2006) also discussed a study with the objective to develop a screening criterion and a procedure to identify all bridges in an area that might possibly be vulnerable; screen out those bridges that do not need further analysis; analyze potentially susceptible bridges; compute surge/wave and resistive forces; and compute structural response. Sheppard and Dompe (2006) narrated the procedure in details and presented the computation of a "Wave Vulnerability Classification Index" as W*P(SS+E+B+S+C) using the variable defined in the table below. The results vary from 0 to 16 and bridges with Indices ≥ 5 need further analysis.

Table A6.1. Computation of bridge vulnerability to storm surge and wave loading (Sheppard and Dompe, 2006)

$\frac{\eta_{max}}{z_e}$	W	Wind Direction Probability	P	$\frac{d_s}{z_d}$	SS	$\frac{\eta_{max}}{t}$	E	Buoyancy $\frac{a}{b}$	B	Span Type	S	Criticality	C
$( ) < 0$	1	0.8 – 1.0	1.0	$\frac{d_s}{z_d} > 1.2$	2	$0.1 < \frac{\eta_{max}}{t} \le 0.5$	1	$0 < ( ) < 1$	0	single	1	1	0
$0 < ( ) < 1$	0	0.6 – 0.8	0.8	$1.2 \ge \frac{d_s}{z_d} \ge 0.8\frac{(z_d - t)}{z_d} = 0.8\left(1 - \frac{t}{z_d}\right)$	3	$0.5 \le \frac{\eta_{max}}{t} \le 1.5$	2	$1 < ( ) < 2$	1	continuous	0	2	2
$1 < ( )$	1	0.4 – 0.6	0.6	$\frac{d_s}{z_d} < 0.8\frac{(z_d - t)}{z_d} = 0.8\left(1 - \frac{t}{z_d}\right)$	1	$1.5 < \frac{\eta_{max}}{t}$	3	$2 < ( )$	3			3	4
		0.0 – 0.4	0.5									4	6

One of the largest limitations in existence in the ongoing study of bridge vulnerability to hurricane-induced storm surges is the lack of ability to apply the analysis for each bridge case study to the entire network of coastal bridges in each state. It would be beneficial to identify on the network of Florida roadways, the coastal bridges in existence that are considered to be at high risk to being damaged from storm surge and wave loading during a hurricane. This will help develop a priority ranking for mitigation. The two major bridge database resources currently available are the National Bridge Inventory (NBI) database compiled by the Federal Highway Administration (FHWA) and the AASHTO's Pontis Database. However, both databases lack the required information necessary to perform a storm surge risk analysis, i.e., data such as the fetch length, the average water depth over the fetch length, vertical clearance of the lowest superstructure member above the water, the base design wind speed, and the 100-Year SWL given in ft-NAVD. Currently in the analyses, the superstructure elevations have to be obtained from engineering drawings and the fetch lengths must be manually determined either from Geographical Information Systems (GIS) plots from the US Army Corp or FEMA.

It is recommended that a database Florida Coastal Bridge Database (FCBD), be established for all coastal bridges that will contain all the pertinent data needed for the network evaluation for storm surge and wave loading effects during hurricanes. This database can be interfaced with the proposed Excel spreadsheet presented in the following paragraphs, for evaluation of the bridges. More details on the development of the spreadsheet and its application to the I-10 Escambia Bridge as a case study are presented in Stanford (2012), an unpublished master's degree thesis from Florida State University. Following the methodology presented by Sheppard and Dompe (2006), the Microsoft Excel spreadsheet was developed based on the AASHTO Level I analysis, to enhance a quick evaluation of coastal bridges. The rest of this section consists of screen shot images of the different sheets from the sample spreadsheet analyzer. There are a total of four sheets in the spreadsheet program. The first sheet is the User's Guide of the program; this sheet explains how to use the software and what the other sheets are within the program.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LEVEL 1 STORM SURGE ANALYSIS FOR FLORIDA COASTAL BRIDGE DATABASE USERS GUIDE:

The following program is intended to provide the necessary analysis required to address the level of vulnerability of Florida's coastal bridges to possible damage from storm surges during the event of a major hurricane.

This program consists of 4 sections or "sheets." They can be found at the bottom of the page labeled: "Program User Guide, Bridge Parameters, Wave Parameters, and Wave Vulnerability Index."

Bridge Parameters:
User will be required to enter in the parameters in this sheet to perform the analysis in other sections. These parameters can be entered directly from a database where provided or entered manually via gathering the required values individually.

Wave Parameters:
This section will display all of the approximated wave dimensions and characteristics for a given bridge location exposed to a major hurricane (design storm).

Wave Vulnerability Index:
This section will be used to quantify the level of vulnerability a bridge is to experiencing potential damage from the design storm using Sheppard and Dompe's "*Wave Vulnerability Classification Index.*" Using the Index Key provided below in this section along with the parameters entered or calculated in previous sheets, a wave vulnerability index value will be calculated. These values can range from 0 to 16; Bridges with indices equal to or greater than 5 will require Level 2 or Level 3 analysis by a qualified coastal engineer.

Wave Vulnerability Classification Index

$\frac{\eta_{max}}{z_c}$	W	Wind Direction Probability	P	$\frac{d_s}{z_c}$	SS	$\frac{\eta_{max}}{t}$	E	Buoyancy $\frac{a}{b}$	B	Span Type	S	Criticality	C
$() < 0$	1	0.8 – 1.0	1.0	$\frac{d_s}{z_c} > 1.2$	2	$0.1 < \frac{\eta_{max}}{t} \le 0.5$	1	$0 < () < 1$	0	single	1	1	0
$0 < () < 1$	0	0.6 – 0.8	0.8	$1.2 \ge \frac{d_s}{z_c} \ge 0.8\frac{(z_c - t)}{z_c} = 0.8\left(1 - \frac{t}{z_c}\right)$	3	$0.5 \le \frac{\eta_{max}}{t} \le 1.5$	2	$1 < () < 2$	1	continuous	0	2	2
$1 < ()$	1	0.4 – 0.6	0.6	$\frac{d_s}{z_c} < 0.8\frac{(z_c - t)}{z_c} = 0.8\left(1 - \frac{t}{z_c}\right)$	1	$1.5 < \frac{\eta_{max}}{t}$	3	$2 < ()$	3			3	4
		0.0 – 0.4	0.5									4	6

Figure A6.2. Image of user's guide sheet in FCDB network analysis spreadsheet

The following sheets in the program, as explained in the User's Guide, are the input and calculation sheets of the software. The Bridge Parameters sheet is where all the information from the Florida Coastal Database will be imported. The Wave Parameters and the Wave Vulnerability Index Sheets are calculations sheets where equations are programmed into the cell columns and output the values corresponding to the data field category. These two sheets are where the results of the analysis will be available. These three remaining input and calculation sheets are shown below.

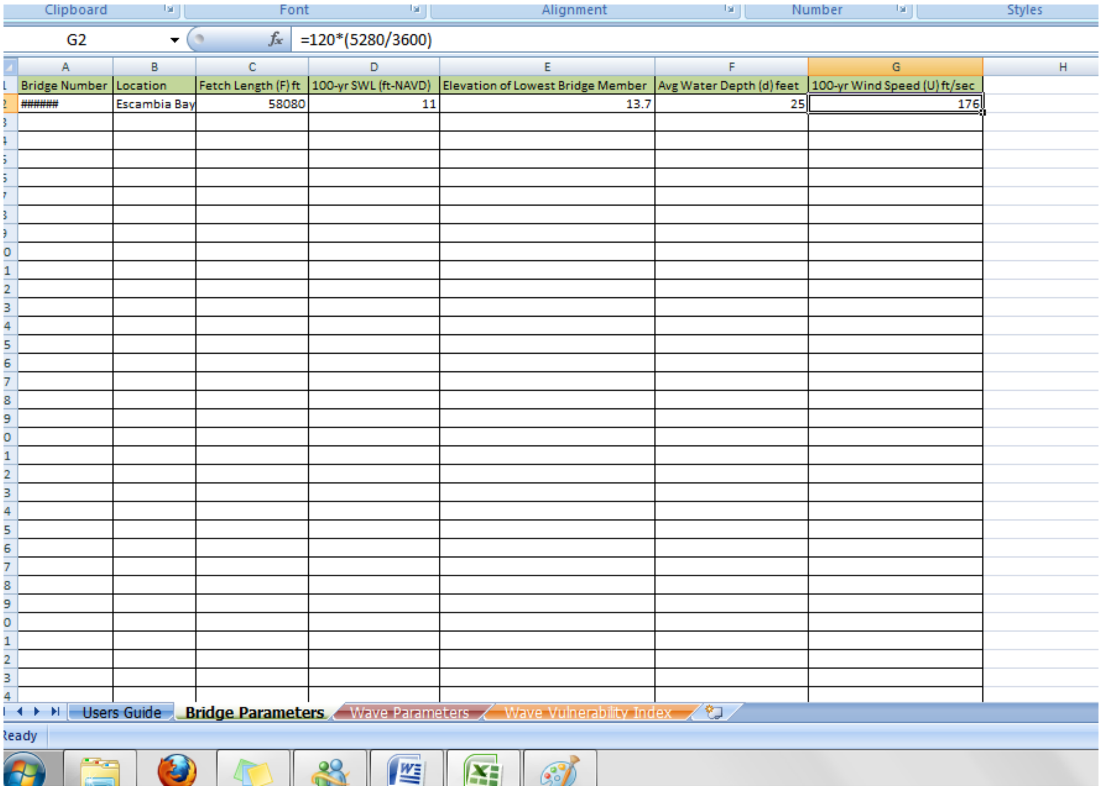


Figure A6.3. Image of bridge parameters sheet of FCDB network analysis spreadsheet

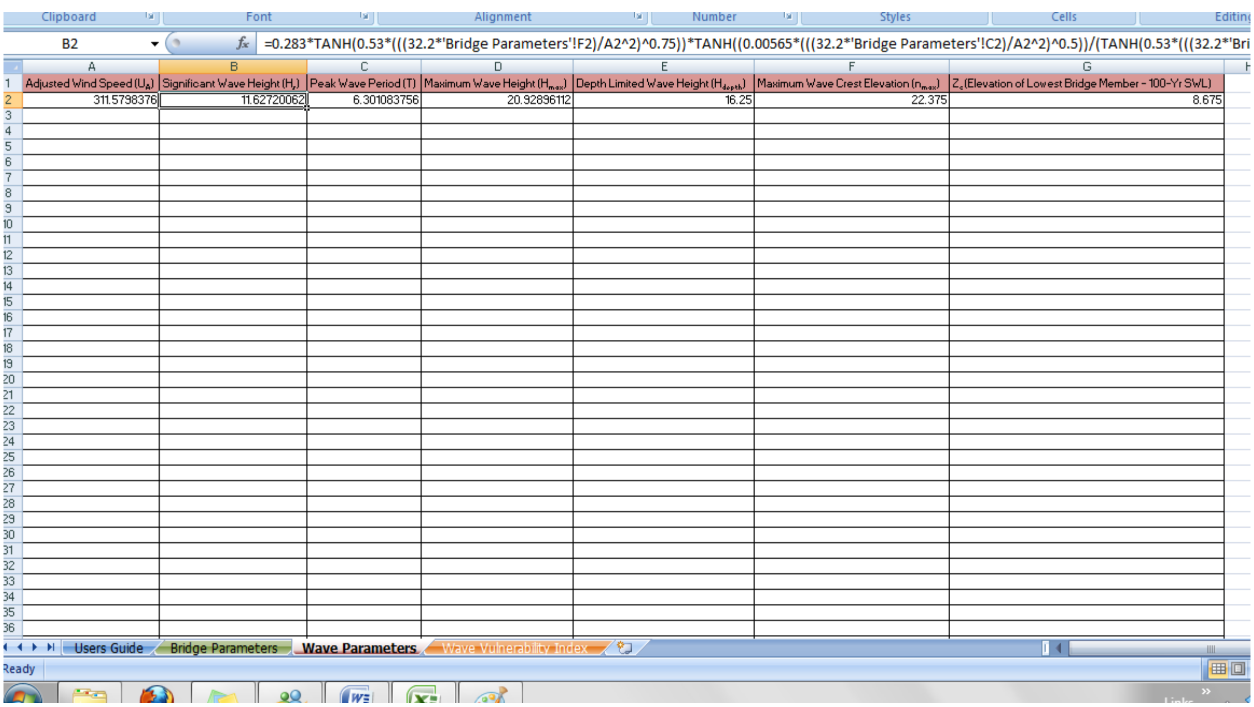
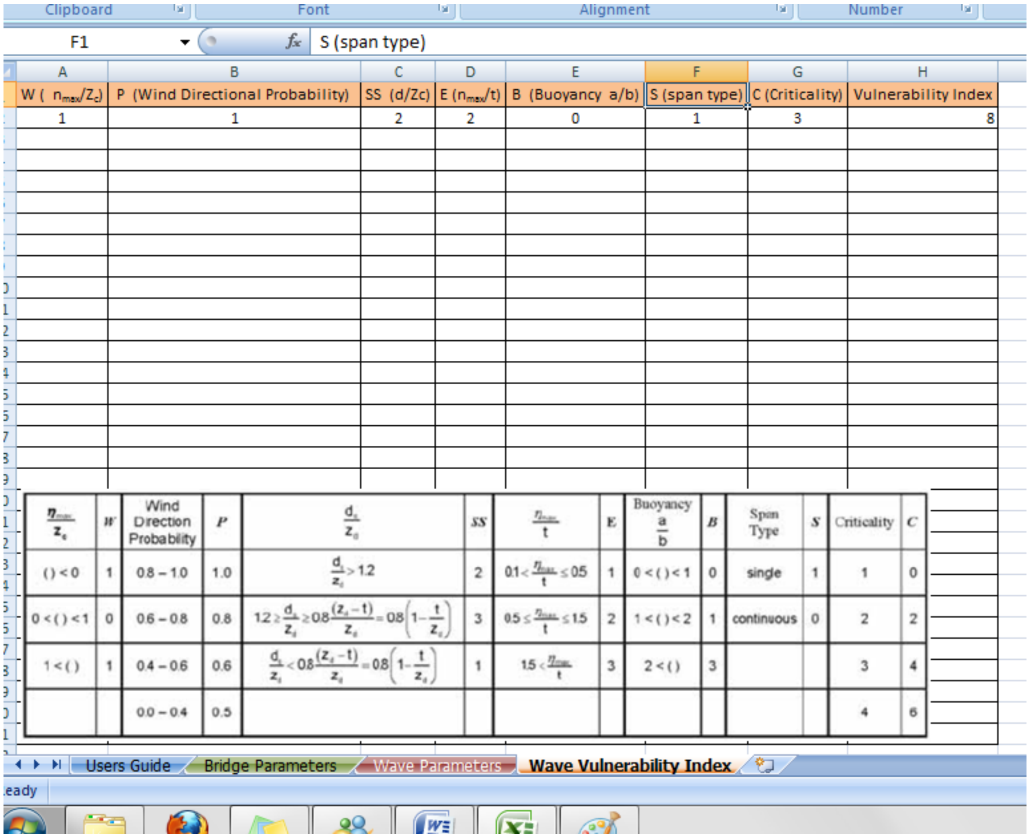


Figure A6.4. Image of wave parameters calculation sheet of FCBD network analysis spreadsheet



W (n_{max}/Z_c)	P (Wind Directional Probability)	SS (d/Zc)	E (n_{max}/t)	B (Buoyancy a/b)	S (span type)	C (Criticality)	Vulnerability Index
1	1	2	2	0	1	3	8

$\frac{\eta_{max}}{z_c}$	W	Wind Direction Probability	P	$\frac{d_s}{z_d}$	SS	$\frac{\eta_{max}}{t}$	E	Buoyancy $\frac{a}{b}$	B	Span Type	S	Criticality	C
$()<0$	1	0.8 – 1.0	1.0	$\frac{d_s}{z_d}>1.2$	2	$0.1<\frac{\eta_{max}}{t}\leq 0.5$	1	$0<()<1$	0	single	1	1	0
$0<()<1$	0	0.6 – 0.8	0.8	$1.2\geq\frac{d_s}{z_d}\geq 0.8\frac{(z_d-t)}{z_d}=0.8\left(1-\frac{t}{z_d}\right)$	3	$0.5\leq\frac{\eta_{max}}{t}\leq 1.5$	2	$1<()<2$	1	continuous	0	2	2
$1<()$	1	0.4 – 0.6	0.6	$\frac{d_s}{z_d}<0.8\frac{(z_d-t)}{z_d}=0.8\left(1-\frac{t}{z_d}\right)$	1	$1.5<\frac{\eta_{max}}{t}$	3	$2<()$	3			3	4
		0.0 – 0.4	0.5									4	6

Figure A6.5. Image of wave vulnerability index of FCDB network analysis spreadsheet

A6.1. References

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