Improving Resilience of Transportation Infrastructure to Hurricane Damage

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16. Abstract

Recent hurricanes and other extreme events have resulted in substantial damage to North Carolina's transportation infrastructure. In some cases, the same locations have been damaged multiple times, but the amount of damage differs, which suggests that DOT strategies were effective. However, without a detailed quantification of the performance differences, NCDOT engineers had had to rely on qualitative and anecdotal evidence as to the effectiveness of various strategies. The research described in this report was conducted to fill this gap by evaluating sites damaged and repaired following Hurricanes Matthew and Florence. A series of detailed case studies were carried out to identify the design factors and repair/maintenance decisions that led to better performance during Hurricane Florence. The main conclusion from this effort is that that the actions taken by the NCDOT hydraulics unit and maintenance operations group following the recent hurricanes have been effective at increasing the robustness and reparability (i.e., resilience) of roadways with pipe crossings. It was also found that when pipes and culverts were redesigned following the hurricane related damage that they were almost always upsized, suggesting an historical bias in pipe sizing. It was also found that headwalls, extended rip rap along the banks or embankments, use of No. 57 stone as backfill or bedding, ensuring sufficient cover and managing headwater to bed-to-crown ratios when making design decisions, using (where possible) non-erodible or less-erodible soils, and mitigating side ditch slope issues had a positive effect on the overall robustness or reparability of culverts. This research resulted in three primary recommendations. First, the NCDOT should continue to follow the practices used following Hurricanes Matthew and Florence. Second, the Survey 123 database should be enhanced to store additional information about the design and repair process. Third, the NCDOT should continue to monitor the performance and monitor flow rates at select sites cataloged in this research project.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Infrastructure resilience has become an important topic for North Carolina. Recent hurricanes and other extreme events have caused more than $450 million in damage to the States's transportation infrastructure. In addition to the cost of the infrastructure, the NCDOT spent considerable resources to redesign and repair many elements after each event. A review of the NCDOT records following Hurricane Florence indicates that more than 3,000 disruptions resulted from that event alone. Some of these locations were identical to those damaged during Hurricane Matthew but, the amount of damage was different between the two events, suggesting that DOT strategies were effective. However, detailed quantification of the performance differences have not been completed and thus NCDOT engineers must rely on qualitative and anecdotal evidence as to the effectiveness of various strategies.

Though many agencies have studied the topic of infrastructure resilience to extreme events, the literature suggests that the generalizability of their findings is limited because of the contextual sensitivity of the available strategies. In this case, data on the effectiveness of design and repair strategies within the context of North Carolina is required. Thus, research is needed to identify and evaluate the specific elements of the new infrastructure that positively contributed to the improved performance during Hurricane Florence and those that did not positively contribute.

With respect to this need, this research project has sought to achieve four objectives: 1) evaluate the design process for roadway infrastructure that was repaired following Hurricanes Matthew and Florence; 2) identify the specific elements of the new infrastructure that positively contributed to improved performance during Hurricane Florence; 3) develop recommendations on improving database and refining management, maintenance and repair guidelines; and 4) develop recommendations on a set of design features and practices that contribute to improved resilience of NCDOT roadways.

The research achieved these objectives by first conducting a literature review on the relevant topics in order to establish a baseline knowledge and ensure that the project addressed critical knowledge gaps. Then locations where roadway infrastructure failed during Hurricanes Matthew and Florence were identified, mapped, and compared. Next, the performance of the maintenance, repair, and reconstruction strategies deployed in the aftermath of Hurricane Matthew were evaluated and quantitatively assessed. From this process, a series of detailed case studies were carried out to identify the design factors and repair/maintenance decisions that led to better performance during Hurricane Florence. Finally, the case studies were examined in order to identify factors that contributed to increased potential for damage and vulnerability were identified.

Based on this research it was first concluded that the actions taken by the NCDOT hydraulics unit and maintenance operations group following the recent hurricanes have been effective at increasing the robustness and reparability (i.e., resilience) of roadways with pipe crossings. These actions include design and repair decisions as well as decisions to create a database to catalog damage assessments. It was also found that when pipes and culverts were redesigned following hurricane related damage that they were almost always upsized. Between approximately 67% and 75% of the damaged sites evaluated were undersized by current design standards prior to them being damaged. This finding does not mean that the same proportion of all sites across North Carolina are under designed, but that, as expected, sites that are under designed with respect to the common design event are more likely to be damaged during an extreme event. The research team also concluded that the following features had a positive effect on the overall robustness or

reparability of pipes; 1) headwalls, 2) extended rip rap along the banks or embankments, 3) use of No. 57 stone as backfill or bedding, 4) ensuring sufficient cover and managing headwater to bed-to-crown ratios when making design decisions, 5) using (where possible) non-erodible or less-erodible soils, and 6) mitigating side ditch slope issues. Finally, the research also identified several features that when existing in combination can negatively affect the vulnerability of a site; 1) presence of erodible soil, 2) surrounding swamps, 3) nearby beaver dams, 4) wide flood plains, and 5) strong flow (indicated by erosion in the bottom of the channel).

This research resulted in three primary recommendations. First, the NCDOT continue to follow the practices used following Hurricanes Matthew and Florence. Second, the Survey 123 database be enhanced to store additional information about the design and repair process. Third, the NCDOT continue to monitor the performance and monitor flow rates at select sites cataloged in this research project.

1. INTRODUCTION

1.1. Overview

Infrastructure resilience has become an important topic for North Carolina. Recent hurricanes and other extreme events have caused more than $450 million in damage to the States's transportation infrastructure. In addition to the cost of the infrastructure, the NCDOT spent considerable resources to redesign and repair many elements after each event. A review of the NCDOT records following Hurricane Florence suggest that more than 3,000 disruptions occurred. Some of these locations were identical to those damaged during Hurricane Matthew. However, the amount of damage was different between the two events, suggesting that DOT strategies were effective. There are many potential reasons for this outcome;

1. when the infrastructure was initially designed and constructed the design codes and standards were not the same as those used post-Matthew,
2. the infrastructure pre-Matthew was older and perhaps had accumulated damage that had weakened the infrastructure,
3. flooding intensities, though similar and well above normal expectations, may have differed, and
4. debris flow/actual capacity due to deferred maintenance may have also differed in the two events.

There may also be other reasons, but whatever the cause, better knowledge about the repair decisions that had the most positive impact would aid in deploying strategies that result in a more resilient transportation infrastructure in North Carolina.

This study has evaluated the specific elements, design features, or repair options used in the new infrastructure to identify what elements had a positive contribution to the improved performance during Hurricane Florence and those that did not positively contribute. Though guidance on improved and/or resilient design exists from the FHWA, AASHTO, NCHRP, and others, these issues are highly context sensitive with many contributing factors including age, maintenance levels, rainfall intensity, etc. that necessitates a North Carolina specific investigation.

1.2. Status of the Literature

A comprehensive review of the literature pertaining to this project is presented in Appendix A, while a summary of most relevant components of this review is presented below.

1.2.1. Overview of Hydraulic Design Practice

The basic process of hydraulic design at a national level was reviewed. According to Federal Highway Administration (FHWA) design philosophy, the primary purpose of highway drainage facilities is to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway. For a given structure that services a specific drainage area, an estimate of the amount of runoff that will occur for a storm is considered to be a major component of the hydraulic design process (Kilgore et al., 2016). A number of hydrologic methods are available in order to analyze and determine peak runoff for a given storm. From these runoff estimates, design engineers utilize the runoff in conjunction with frequency analyses to characterize the risk for a given drainage area and structure. During design, terms of annual exceedance probability (AEP) or recurrence intervals are used to describe the probability of occurrence of a given precipitation event. Based on the probability of occurrence of an event and the peak runoff that will occur for that event, a hydraulic engineer can design the drainage structure to be able to withstand that precipitation event.

1.2.2. National, State, and Regional Hydraulic Design Practices

National guidelines outlined by FHWA for hydraulic design are utilized in order to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway. The NCDOT's 2016 hydraulic design guidelines consolidate and revise previous guidance to address new challenges in drainage system design for NCDOT-funded projects (Chang, 2016). The literature review examined the current state of design guidelines for extreme storm events and best practices in the design process. The review focused on how the guidelines were applied during Hurricane Matthew and Hurricanes Florence to identify any deviations and improve in future guidelines for increased resilience.

Current practices of the NCDOT suggest that hydraulic guidelines to estimate peak storm discharge rates for drainage design be used. These guidelines rely on hydrologic methods and consider watershed characteristics. Designers are advised to select an appropriate method, calibrate the results with historical data, and consider future land use changes. Accurate estimation of discharge rates is crucial for managing erosion, sediment transport, and flooding (Genereux, 2003).

This literature review also examined the design process utilized by national and state agencies. Through this review, the design process and best practices can differ from other state agencies in the region. While some minor differences were identified, all surrounding states follow the same basic approach as the NCDOT and closely adhere to federal guidance. In this review, methods used by the NCDOT including Flood Insurance Study (FIS), Rational Method, NCDOT Method, USGS methods, and National Resources Conservation Service (NRCS) were comprehensively explored (Genereux, 2003; Chang, 2016; Feaster et al., 2009; Feaster et al., 2014). It was found that while the NCDOT follows the same general guidance that the hydraulic design put into practice can vary slightly, as each project has unique circumstances that might require the design engineer to deviate from the guidelines. The design practices including USGS methods, NCDOT Highway Hydrologic Chart and National Resources Conservation Service (NRCS), and Hydraulic Reports were discussed with NCDOT hydraulics engineers.

Once the appropriate method has been selected, the design frequency for that roadway or structure must be determined as well. The design storm frequency for NCDOT drainage structures is determined based on variables such as the roadway classification, traffic volume, level of service, flooding potential to properties, and maintenance costs, among others (Chang, 2016). Overall, these design frequencies ensure that drainage systems can effectively handle flood events within acceptable limits.

1.2.3. Infrastructure Resilience

The literature pertaining to transportation infrastructure resilience can be grouped into one of four main focus areas; frameworks for enhancing resilience, design for resilience, tools for assessment of vulnerabilities, and studies to identify and justify the return on investments in resilience initiatives to decision makers.

Studies pertaining to frameworks for enhancing resilience was reviewed. From this review, it was established that an important component of developing a robust resilience plan is the establishment of a strong framework through which to structure decision making and planning. The framework gathers in one sequence of steps the various activities that will enhance an agency's resilience efforts to natural and human-caused hazards and threats (Dorney et al., 2021). It also guides transportation officials in; 1) understanding what their agency is currently doing with respect to resilience, 2) identifying where new or modified actions could be taken to enhance these efforts, and 3) recommending steps that can be taken to implement these actions. Several different resilience frameworks exist that have addressed various aspects of an organizational perspective on resilience (DOHS, 2021; Parker and Matherly, 2021; Filosa et al., 2017; NIST, 2016). While many frameworks exist, they generally share the same essential concepts including recognition of hazards of different types and severity, the presence of infrastructure elements at various locations across the network, the limited role of design in mitigating these unforeseen and extreme events, and the need for institutional changes to address the challenges brought on by the above. These institutional changes may include larger focus on data collection (inventories, condition assessment, central planning of rehabilitation/replacement plans, etc.).

A self-assessment tool is developed to assess the current status of an agency's efforts to improve the resilience of the transportation system through the mainstreaming of resilience concepts into agency decision making and procedures (Dorney et al., 2021). The self-assessment tool is based on a resilience framework, the Framework for Enhancing Agency Resilience to Natural and Anthropogenic Hazards and Threats (FEAR-NAHT). The framework is based on a series of 10 sequential steps. These steps were explored comprehensively in the literature review. A score is determined for each of the 10 steps in the framework and a total score is summed across all the steps to determine how mature the organization is with respect to undertaking resilience-oriented activities and efforts. Based on the percent score, a series of recommendations by functional areas are provided for the user to achieve or maintain the highest level of resilience capability. The underlying concept is that periodic examination of all agency actions contributing to a resilient transportation system is an important foundation for a resilience-oriented agency.

Other frameworks developed by Caltrans and Hawaii DOT (HIDOT) were also explored. The detail of these frameworks is explained in the literature review. Caltrans conducted a study to assess vulnerabilities in the state highway system due to climate stressors like sea level rise, storms, temperature changes, precipitation, and wildfires. The findings broaden the applicability for other transportation agencies seeking to improve climate change communication and implement effective adaptation measures. HIDOT has adopted a stage-wise resilience framework consisting of short-term, mid-term, and long-term strategies (Sniffen, 2021). This framework complements the FEAR-NAHT framework by addressing specific issues and monitoring the impact of resilience actions. HIDOT aims to achieve early successes while effectively adapting to climate change challenges.

Studies pertaining to design for resilience were reviewed and it was found that the collective decisions made based on lessons learned after previous disasters can be used to improve the design of the infrastructure to be resilient against threats and hazards. These decisions are based on limited information and location-specific analysis, which needs to be monitored and modified over time as new information becomes available. While much of the literature on resilience pertains to institutional approaches to embedding resilience as a guiding principle, there does exist some evidence that improvements in design practices can also be a part of a resilience framework. A review of this literature suggests design improvements can be done with respect to the following: (1) transportation-related hydraulic assets, (2) asphalt mix design, and (3) pavement structure design. The design improvements in these areas can be applied by changing the design thresholds and standards.

When designing transportation-related hydraulic assets like bridges and culverts, the traditional approach is to use a design event with a specified return period or AEP (Kilgore et al, 2016). For instance, NCDOT guidelines specify storm design frequencies based on road type. Designers estimate hydrologic quantities for the target AEP and size the structure accordingly. However, relying solely on return periods can be misleading. Probability theory can help accurately calculate these probabilities.

Approaches to design a resilient infrastructure suggested by FHWA, Broward County in Florida, Port Authority of New York and New Jersey (PANYNJ), and ASCE/SEI 24-14 were reviewed. The detail of these frameworks are explained in the literature review. FHWA is reducing vulnerability by either reducing the sensitivity of the assets to extreme events or by enhancing the adaptive capacity of the assets, or both (Kilgore et al, 2016). The strategies that are part of this design approach include reinforcing roadway components, evaluating the watershed for debris production potential, evaluating stream geomorphology for channel stability, etc. In Broward County Florida, certain improvements were added to the resilience guidelines to mitigate the issues of sea level rise, increased storm intensity, coastal and inland flooding, extreme rainfall and drought, etc. (Jurado, 2021). The Climate Resilience Guidelines (CRG) developed by the Port Authority of New York and New Jersey (PANYNJ) offer a science-based approach to managing climate-related risks, particularly sea level rise (Ensore, 2021). These guidelines supplement building codes and provide a methodology for incorporating projected sea level rise into project design. The CRG works alongside the ASCE standard for Flood Resistant Design and Construction, which sets requirements for construction in flood-prone areas (ASCE/SEI24-14, 2014). Approaches to increasing the resilience of an asset to flood damage and/or operational disruption generally fall into the basic categories of; (a) elevate, (b) relocate, (c) protect, or (d) accommodate. These approaches are discussed in detail in the literature review.

Studies pertaining to vulnerability assessment were also reviewed. From this review, it was established that an important step in improving resiliency is identifying the vulnerable locations to prioritize for improvement. A vulnerability is a consistent part of any resilience framework as it critically assesses hazards, their likely location, and the existence of infrastructure at those locations. There are different frameworks that rely on vulnerability assessment of different infrastructure including vulnerability and resilience framework for Atlanta region. This approach represents a general framework for the assessment of vulnerability of different elements of transportation infrastructure and resiliency of different elements of transportation infrastructure against extreme events (WSP, 2018). NCHRP 20-83(05) provides a guide for an eight-step diagnostic framework for undertaking an adaptation assessment. This framework includes the steps that should be taken if transportation officials want to know what climate stresses the transportation system might face in the future, how vulnerable the system will likely be to these stresses and what strategies can be considered to avoid, minimize, or mitigate potential consequences. Methods to incorporate adaptation concerns into a typical transportation planning process are also described (Meyer et al., 2014).

Studies conducted by Public Infrastructure Engineering Vulnerability Committee (PIEVC), the UK Highway Agency, USDOT, WSDOT, MPO, and FHWA, Connecticut DOT, and other agencies were reviewed (Filosa, 2017; PIEVC, 2008; Parsons Brinckerhoff, 2008; Choate et al, 2014, CDOT, 2014; Lopez-Cantu, 2018; PennDOT, 2017; Bosma et al., 2015; Crow et al., 2014; Bhat et al., 2019; Blandford et al., 2019). Canadian engineers have employed a five-step protocol, created by the PIEVC, to evaluate the vulnerability and adaptability of different types of public infrastructure in the face of climate change (PIEVC, 2008). The UK Highway Agency has developed the Highways Agency's Adaptation Framework Model (HAAFM) to address climate change challenges (Parsons Brinckerhoff, 2008). This framework consists of seven stages that guide decision-making processes, including identifying objectives, assessing climate trends, evaluating vulnerabilities, analyzing risks, exploring options, implementing action plans, and reviewing the adaptation program. It allows the agency to examine various aspects such as standards, specifications, maintenance, and network operation to effectively respond to the impacts of climate change. The U.S. Department of Transportation (USDOT) conducted a comprehensive, multi-phase study of Central Gulf Coast region to better understand climate change impacts on transportation infrastructure and identify potential adaptation strategies (Choate et al, 2014). The project resulted in a detailed assessment of the Mobile transportation system’s vulnerability as well as approaches for using climate data in transportation vulnerability assessments, methods for evaluating vulnerability and adaptation options.

Studies pertaining to return on investment were also reviewed. From this review, it was established that the process of improving resiliency is more of an iterative process since every framework and design needs to be monitored and modified over time. Each decision to change the framework and design needs to be carefully made since any change in this scale should economically be justified. Therefore, benefit-cost and return on investment analyses are another important element of improving resiliency of the infrastructure. The resiliency of the infrastructure consists of various interdependent elements which makes this type of return-on-investment analysis particularly complicated and for this reason there are a few studies in this area which are in their preliminary stages. Approaches and tools to evaluate return on investment suggested by ADOT, the Urban Land Institutes, MPO, MassDOT, PennDOT, and UDOT were reviewed (Olmsted, 2021; Dewberry Engineers, 2020; McGinley, 2021; Lewis et al., 2021; Urban Land Institute, 2020; Holsinger, 2017). These approaches are discussed in detail in the literature review.

1.2.4. Summary and Knowledge Gaps

In this literature review, the North Carolina DOT drainage and hydraulic design was reviewed and compared against other design guidelines (regionally and nationally). The major components considered in the hydraulic design are peak runoff, annual exceedance probability (AEP), and rainfall intensity. These components are reflected in the selection of a ‘design’ event, the impacts of which are used to select the size and other details of a hydraulic structure. The review has shown that these design events have statistical uncertainties, which should be taken into account. These uncertainties, and the probabilistic implications of the uncertainties, become more pronounced with more extreme events (i.e., lower AEP). Much of the current literature also highlights how climate change adds further uncertainty because it implies a non-stationary effect. Consequently, several cited studies have recommended using the most up-to-date precipitation data and projections to properly identify design intensity. The review also found that FHWA has produced a manual, which explicitly recommends incorporating potential effects of extreme events and climate change because it was established that this approach will enhance the life cycle benefits. In a related effort, NCHRP has sponsored several research projects that have produced guides and comprehensive frameworks for considering and incorporating climate change into the design processes for inland and coastal applications.

The literature review found that NCDOT currently follows FHWA guidelines for hydraulic design. The current practice of NCDOT was described in this literature review and it was compared with other states’ practices. The methods for calculating peak storm discharges and design frequencies were compared between NCDOT practice and other states. The important component that is missing from the design guidelines is that they do not use the most up-to-date design events and there are no dynamic guidelines, i.e., in the case of failure due to extreme events, it is not established that how the design should be improved for future events.

The review of the literature showed that infrastructure resilience is becoming an increasingly critical issue for many agencies and organizations. The literature also demonstrates that resilient infrastructure requires a complex and integrated framework of engineering and institutional management, policy, and decision making. It also requires changing the design standards to be more adaptive to impacts of extreme events, conducting vulnerability assessments to identify locations with highest priority to apply necessary changes, and determining the return on investment when an option or multiple options are considered in decision making. These actions can be interconnected and/or be a component of a resilience framework.

While the leading edge of research has not yet produced specific guidance on how to ensure that infrastructure is resilient to extreme events, it has produced frameworks, strategies, and examples to follow towards that goal. Overwhelmingly, these methods begin with collecting and analyzing performance data in order to develop quantified analyses to support further decision making and improvements in policy and practice. Since the codes and standards as well as policy and socioeconomic conditions vary greatly from one state to the next, this data collection must be done by each agency within their own jurisdictions in order to provide accurate and meaningful insights. In addition, the detailed literature review has confirmed that there does not currently exist national guidance to identify when certain repairs, designs, strategies or other approaches are efficient enough to make a system more resilient in long term. Usually, the last step of any framework is to monitor the applied strategy or design, but there is not currently sufficient long term evaluations of the recommended strategy, repair, or design. In another words, the guidelines are in the stage of "what should be done" or "what has been done," further research studies are required to reach the stage of "what has been done and how the system performed after certain events."

1.3. Report Organization

This report is organized into five main chapters and five appendices. Chapter 1 (this chapter) provides an overview of the project and its goals along with an abbreviated summary of the literature. The detailed literature review is shown in Appendix A. Chapter 2 discusses the research methodology and is supplemented with Appendix B. Chapter 3 and Appendices C-E present the results of the research study. Chapter 4 provides the conclusions and recommendations, while Chapter 5 presents the implementation and technology transfer plan. References for citations provided in the body of the report are provided in Chapter 6 (references for the full literature review are provided in Appendix A).

2. RESEARCH METHODOLOGY

2.1. Overview

Overall, 138 sites (damaged in both or one of two events) were selected and evaluated in this project. The sites were chosen after mapping all of damaged sites and sorting by type of damage (roadway, embankment, etc.) along with the cumulative rainfall from the respective storm event across North Carolina. The required rainfall intensity data was obtained from NOAA National Weather Database and the damage data was provided by NCDOT's database (Survey 123) and NCDOT Project 2021-03. The detail of the vulnerability assessment is presented in Section 2.2. For each site, the hydro reports, contracts for work completed, photos, damage reports were comprehensively reviewed for the selected sites. A list of all the selected sites for this study is given in Table C.1 in Appendix C of this document. In addition to detailed assessment of performance and vulnerabilities through case studies, interviews were carried out with division engineers to gain insights into their practices, assessing sites based on information gathered during site visits

2.2. Identify North Carolina Vulnerabilities

2.2.1. Vulnerability Based on Similar Hurricane Intensity

The daily Quantitative Precipitation Estimates (QPE) were extracted from the National Weather Services (NWS) NOAA for the period of October 6 – 10, 2016 and September 13 – 19, 2018 for Hurricane Matthew and Florence, respectively. Since cumulative rainfall causes much of the damage, the cumulative precipitation for the corresponding periods was calculated and mapped in GIS (Figure 1). The detail of this analysis is discussed in Appendix B.

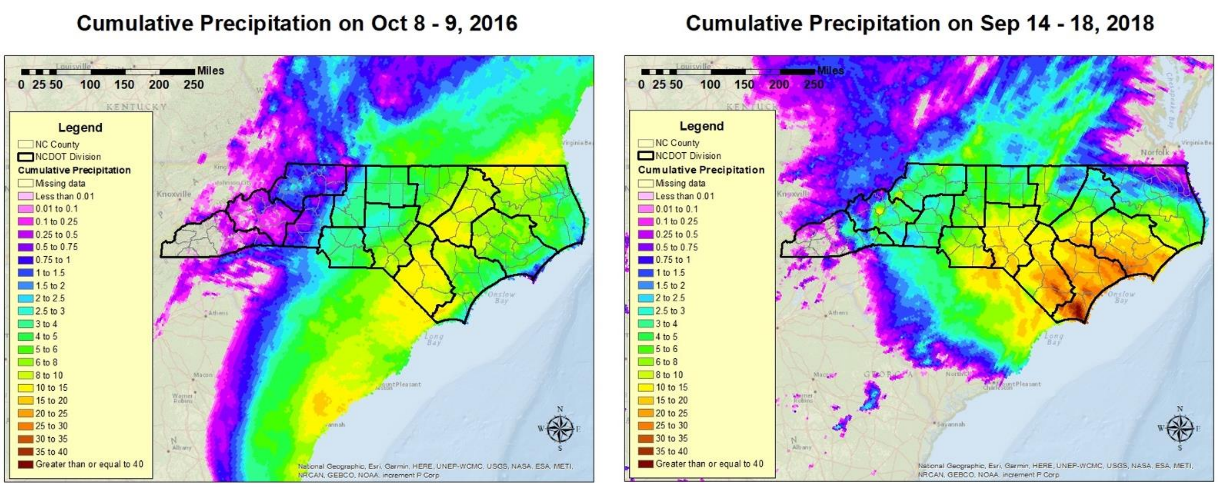

Figure 1. Intensity maps for Hurricanes Matthew (left) and Florence (right).

These maps were overlapped in ArcGIS using the Intersect tool in order to find regions with the same level of precipitation in both events. Since the damaging amount of precipitation is the focus of this study, only areas with precipitation levels higher than 8 inches were considered. The results indicate that NCDOT Divisions 2, 3, 4, 5, 6, and 8 have the same level of precipitation in common. Among these divisions, Division 6 is the key region with the same highest level of precipitation in both events and one of the most damaged locations in Hurricane Florence (Figure 2).

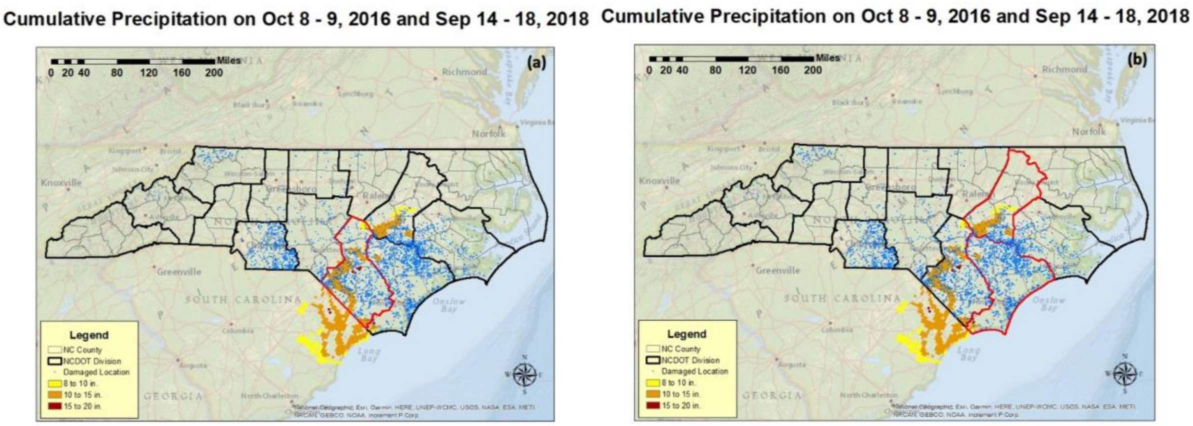

Figure 2. Collocated areas with high level of precipitation, (a) Division 6 as study region and (b) Divisions 3 and 4 for verifications study.

2.2.2. Vulnerability Based on Similar Damaged Locations

The information and GIS layers for damaged locations after Hurricane Matthew and Florence were obtained from NCDOT project RP 2021-03. Additional information and GIS layers for damaged locations after Hurricane Florence were obtained from NCDOT Survey 123. The focus of this analysis is on locations with damaged pipes. Locations damaged after Hurricane Florence were combined from NCDOT RP 2021-03 and Survey 123. The details of this mapping analysis is presented in detail in Appendix B. The GIS layers for damaged locations after Hurricanes Matthew and Florence (Figure 3 and Figure 4) were overlapped and a 500-meter buffer was applied to all sites from the datasets. Locations where the buffers overlapped were considered as candidate sites for being damaged in both Hurricane Matthew and Florence (Figure 5). The number of damaged locations in each county, number of overlapped locations, and percentage of overlapped locations with respect to number of damaged locations after Hurricane Florence are summarized in Table 1. Maps associated with each county including the overlapped locations are presented in Appendix B.

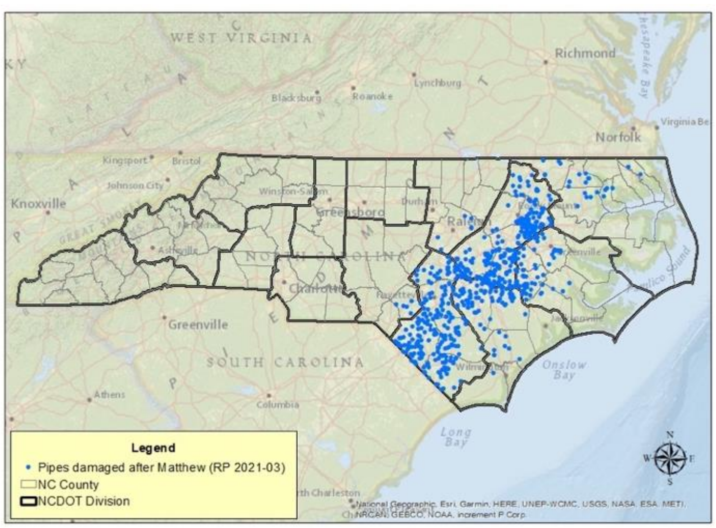

Figure 3. Damaged locations (pipes) after Hurricane Matthew.

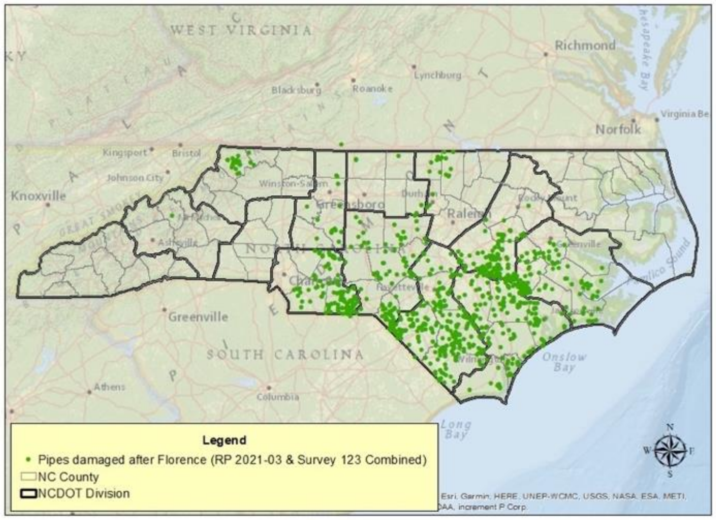

Figure 4. Damaged locations (pipes) after Hurricane Florence.

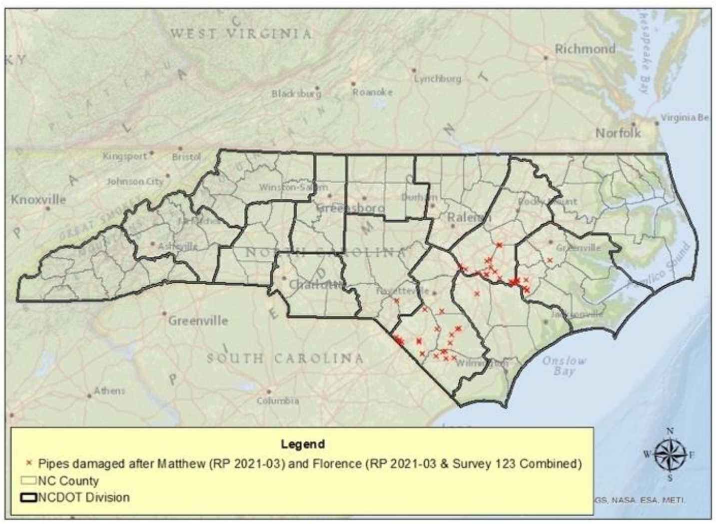

Figure 5. Overlapped damaged locations (pipes) after Hurricanes Matthew and Florence.

Table 1. Number of damaged locations (pipes) in each county.

Division	County	Damaged after Florence	Damaged after Matthew	Overlapped[1]	% Overlapped[2]
	Robeson	57	78	7	12.3
	Columbus	50	40	5	10.0
6	Harnett	16	31	1	6.3
	Bladen	70	37	3	4.3
	Cumberland	41	37	1	2.4
4	Johnston	22	47	4	18.2
	Wayne	95	95	6	6.3
2	Lenoir	33	39	6	18.2

[1] Within 500 meters
[2] With respect to Florence cases

Table C.1 in Appendix C provides a list of all selected sites that were damaged after both Hurricane Matthew and Florence. The identification number assigned to each case comprising the county code, the location number, and letters "MF" to indicate that this site was damaged after both Hurricane Matthew and Florence. For example, the ID "77-1-MF" indicates that this site corresponds to Robeson County (code 77), location number is 1, and was affected after both Hurricane Matthew and Florence (MF).

2.2.3. Vulnerability Based on Damaged Locations Only in One Event

To compare the performance of the pipes during the two events and identify any patterns associated within the data, sites for case studies were identified based on the damage occurring in either or both events, i.e., damaged after Hurricane Matthew and not damaged after Hurricane Florence or damaged after Florence and not damaged after Hurricane Matthew. With respect to the project goals, these sites are as important as the overlapped site (locations damaged in both events) because they represent; 1) cases where design/repair decisions after Matthew also made the structure robust against Florence and 2) cases where the existing design was able to withstand Matthew but were damaged after Hurricane Florence (similar water levels in both). Since the databases for pipes damaged due to Hurricane Matthew and/or Florence contain a large number of cases, a subset of critical cases was identified based on similarities or differences in category of repair cost and level of precipitation. The number of cases that were selected in each county is presented in Table 2 while Appendix B shows the details of the analysis.

Table 2. Number of selected sites that were damaged in only one of two events.

Division	County	Damaged after Matthew	Damaged after Florence
6	Robeson	16	9
	Columbus	6	6
	Harnett	5	3
	Bladen	7	8
	Cumberland	4	4
4	Johnston	0	0
	Wayne	9	12
2	Lenoir	9	7

A list of all selected sites that were damaged after only one of two events (Matthew or Florence) is provided in Table C.1 in Appendix C. Similar to overlapped locations, the identification number assigned to each case consists of county code, the location number, letter "M" for Matthew or letter "F" for Florence (e.g., 77-1-M).

2.3. Conduct Performance Assessment

2.3.1. Site Visits

Site visits to a subset of damaged sites were carried out to verify the data available through the databases and reports provided by the NCDOT and to collect additional supplementary information. Initially, the research team visited 26 sites in Johnston and Robeson County. These initial visits provided insights to define a detailed approach for collecting data during the larger field data collection effort. A sample of survey form is presented in Figure C.1 in Appendix C. As instructed in the survey form, information logged included pipe size, the existence of headwalls and their size, the existence and length of rip rap, the direction of stream flow (verified where unclear with Streamstats), soil type, recent damage or repair, and any other features that may have contributed to vulnerability (sag/crest, wide/narrow streambed and floodplain, etc.). In addition to these details, multiple photographs were also taken. Based on this data collection method, the research team and NCDOT summer interns visited 59 sites in six counties including Wayne, Lenoir, Harnett, Bladen, Cumberland, and Columbus. A summary of all visited sites is provided in Table C.1 in Appendix C. The data was organized and inputs for design analysis, signs of deterioration after the event, possible contributing features, etc. was compiled.

2.3.2. Flow Analysis using Hydraulic Design Tools (HDS5 and HY-8)

For the purposes of this project, the HDS-5 and HY-8 design tools were used to evaluate the hydraulic designs in more detail. In addition to evaluating the pipes for the design flow condition, models for the cases damaged in Florence and Matthew were developed. The HDS-5 software (Figure 6) relies on established nomographs, which varies by pipe type, and requires inputs of peak discharge, inlet arrangement, slope, and culvert diameter in inches. HY-8 (Figure 7), which is a computerized implementation of the same FHWA culvert hydraulic analysis approach outlined in HDS-5, allows the user to more accurately portray the structure and site characteristics by utilizing discharge data, tailwater data, roadway data, culvert data and site data to obtain a ratio of headwater to depth inside the pipe (HW/D).

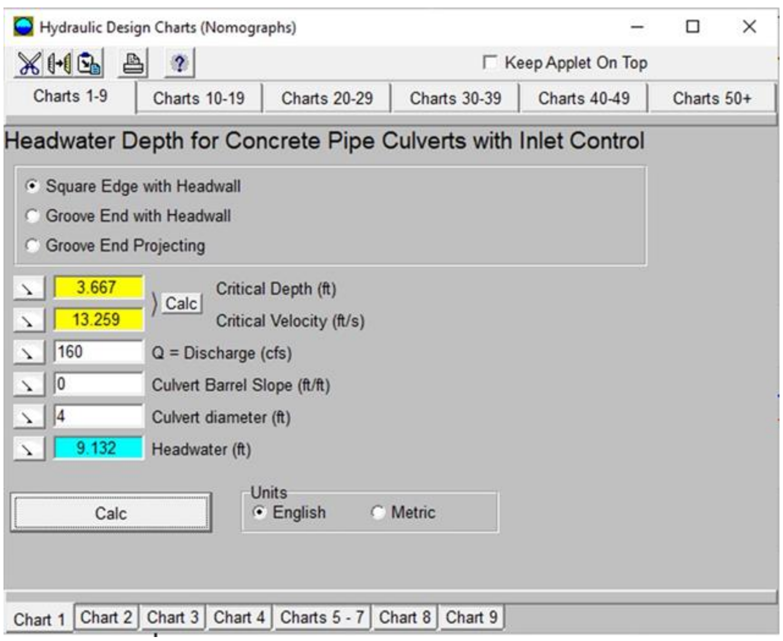

Figure 6. Computerized program version of HDS-5 Nomographs.

The first step for the HDS-5 analysis, as seen on the HDS-5 input screen (Figure 6), was to choose the applicable nomograph for the pipe being analyzed. The specific chart utilized for the design analysis depended on the type of pipe and its features. For other inputs, information from the NCDOT was used. The culvert slope was set to zero and the design discharge was divided by the number of barrels to correctly calculate the headwater depth. Lastly, the diameter of the pipe being analyzed was input. The HW/D ratio mentioned above is then used to determine the hydraulic viability of a pipe, and based on the NCDOT design criterion, any below 1.2 is considered to be of sufficient flow capacity.

The HY-8 analysis, as shown in Figure 7, involved inputs such as discharge data, outlet side details (channel type, channel bottom with and side slope values), roadway data (crest length, crest elevation, and top width), culvert data and site data (outlet station, top width values and number of culvert barrels) derived from both hydro reports and the site visits conducted. In addition to analyzing the pipe using the design discharge, additional discharge rates, calculated as a ratio of the design discharge (0.5, 1.5, 3, and 6) were also utilized.

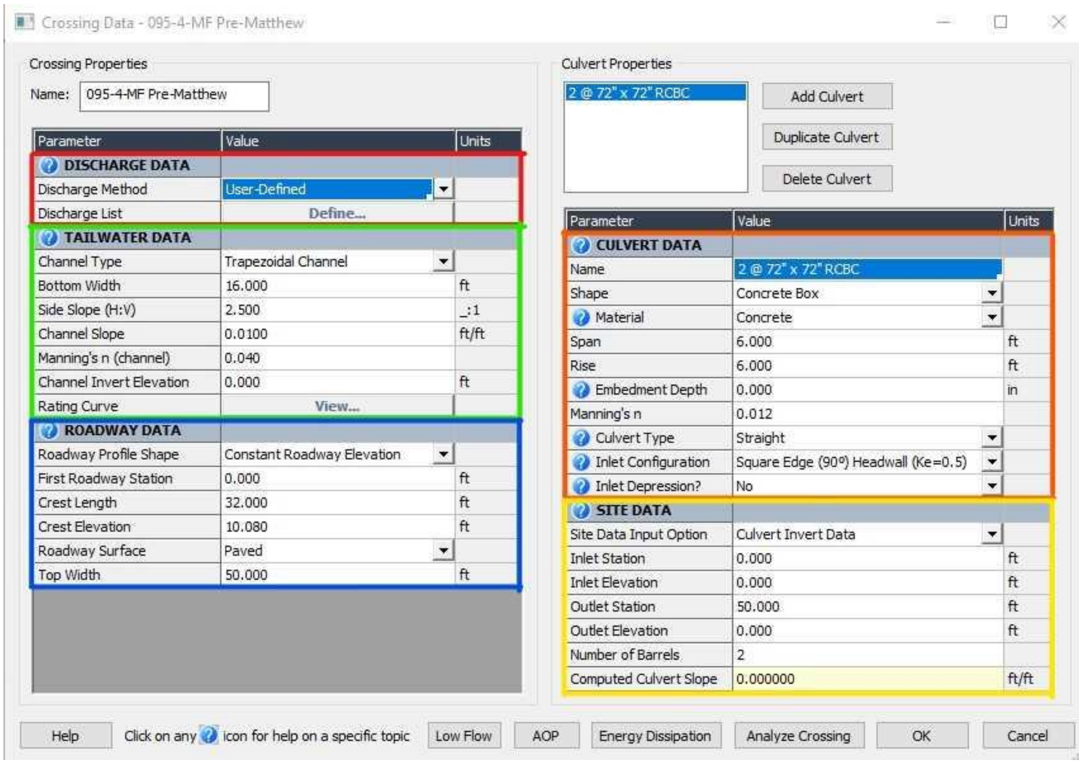

Figure 7. Inputs window in HY-8 software used to build sites and pipe cross sections: Red Discharge Data; Green- Tailwater Data; Blue- Roadway Data; Orange- Culvert Data; Yellow- Site Data.

Similarities and differences in HDS-5 and HY-8 analysis outcomes, for peak design discharge and for both Florence and Matthew storm events, were evaluated for all case study sites. As expected, a majority of the sites damaged in either storm were under-designed according to the NCDOT overtopping criterion of HW/D ≤ 1.20. The flow levels experienced during Hurricanes Matthew and Florence were estimated by first calculating the ratio between the 25-year, 24-hour precipitation levels from NOAA-Atlas 14 and the observed 24-hour precipitation from each event. The observed 24-hr precipitation was calculated in two ways; the NOAA definition of 24-hr event, which is the cumulative precipitation between 7:00 AM and 6:59 AM of the next calendar day (referred to as Method 1), and the heaviest 24-hr period during the event (referred to as Method 2). The ratio calculated with the Method 1 definition was referred to as 'Ratio 1' and the ratio calculated with the Method 2 definition was referred to as 'Ratio 2'. Using the NCDOT overtopping criterion of HW/D ≤ 1.20, undersized structures were determined as sites estimated to have been overtopped in either hurricane Matthew or Florence (HW/D > 1.20) and oversized structures were identified as sites with HW/D ≤ 1.20.

2.3.3. Damage Level Correlations

In addition to assessing whether the structures were under-sized or adequately designed for specific storms, the research team sought to explore other potential failure pathways. Scatter plots were employed to investigate the correlations between the level of culvert damage and other influential parameters during the design event. Influential parameters considered included, Headwater depth to Bed-to-Crown ratio (HW/BC), drainage area (DA), Bed-to-Crown distance (BC), and backfill depth. The aim was to identify whether any vulnerabilities could play a role in making the culverts more (or less) susceptible to damage.

Three culvert damage categories were considered as described in Section 2.4.4: pipe damage, shoulder damage, and pavement damage. For the correlation analysis, the flow values used in the HDS-5 and HY-8 analyses were determined using the 2016 Hydraulic Guidelines. Different parameters such as hydrologic region, design storm frequency, annual exceedance probability (AEP), percentage of impervious area, drainage area, rainfall intensity and runoff coefficient were then used with corresponding hydraulic guideline (i.e. Rational method or USGS equations) to determine the peak design discharge. Drainage area values for each case were determined using StreamStats, a web-based Geographic Information System (GIS) that utilizes spatial data and digital elevation models to estimate streamflow information for specific locations or areas (USGS, 2019). Bed-to-Crown (BC) values were acquired through field measurements conducted during site visits. Lastly, backfill values were calculated by finding the difference between BC and the pipe diameter.

2.3.4. Design Storm Uncertainty

The research team evaluated the design storm uncertainty and its implications on the potential for overtopping. Such analysis is important to the overall scope of the project in order to understand how uncertainty in the underlying data used to estimate flows can affect the vulnerability of an asset to damage and overtopping.

Flow values for different return periods and drainage areas were analyzed along with their corresponding percentiles. For this analysis, raw data from the published USGS data were extracted and used to establish the line of best fit and the confidence interval of this line of best fit for different AEP percentages. The charts utilized to study this relationship were extracted from the USGS Scientific Investigations Reports 2011-5042 (Gotvald and Knaak, 2011), and 2014-5030 (Feaster et al., 2014), which are shown in Figure 8. These are very similar to the data used to generate the regression equations used in North Carolina and so they serve as a useful proxy to the uncertainty in North Carolina flow estimates.

The data from these curves were first digitized using digital graph extraction tools. Then, the line of best fit was characterized along with lines for different confidence intervals. The line of best fit represents the functional form used in the USGS guide to estimate the flow values as a function of drainage area. The equations developed were identical to the flood frequency equations for rural ungauged streams shown in Table 3. Subsequently, the relationship between the flow values (Q) corresponding to specific percentiles and the recurrence intervals of storms was defined as illustrated in Figure 9. The analysis was conducted using drainage areas greater than 640 acres, following the guidelines outlined in the 2016 Hydraulic manual, as detailed in Table 4 based on USGS Rural (2009).

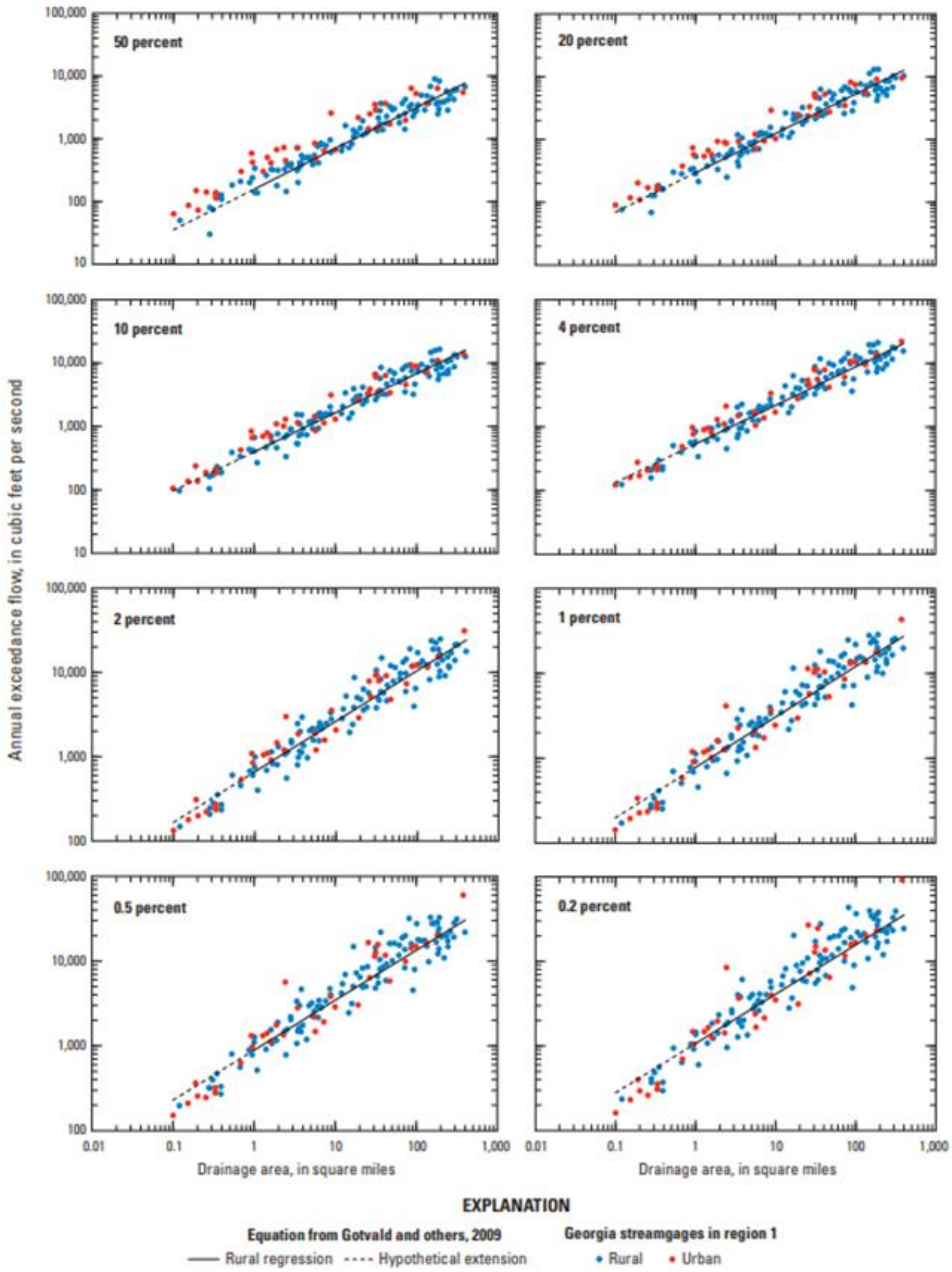

Figure 8. Relation between the annual exceedance probability flows and drainage areas for Georgia stream gages in Hydrologic Region 1 (Gotvald and Knaak, 2011).

Table 3. Rural flood frequency equations for rural, ungauged streams with drainage basins that are within one hydrologic region (figures "5 and 7" referred to in this table are in the USGS reports mentioned and do not refer to tables in this report) (J. Curtis Weaver, 2009).

[*DA*, the drainage area in square miles]

Percent chance exceedance	Hydrologic region (shown in figures 5 and 7)				
	1	2	3	4	5
50	$158(DA)^{0.649}$	$110(DA)^{0.779}$	$25.7(DA)^{0.758}$	$60.3(DA)^{0.649}$	$91.2(DA)^{0.649}$
20	$295(DA)^{0.627}$	$209(DA)^{0.749}$	$44.7(DA)^{0.744}$	$123(DA)^{0.627}$	$200(DA)^{0.627}$
10	$398(DA)^{0.617}$	$288(DA)^{0.736}$	$58.9(DA)^{0.740}$	$174(DA)^{0.617}$	$295(DA)^{0.617}$
4	$537(DA)^{0.606}$	$398(DA)^{0.724}$	$77.6(DA)^{0.736}$	$245(DA)^{0.606}$	$447(DA)^{0.606}$
2	$661(DA)^{0.600}$	$479(DA)^{0.718}$	$91.2(DA)^{0.735}$	$309(DA)^{0.600}$	$575(DA)^{0.600}$
1	$776(DA)^{0.594}$	$575(DA)^{0.713}$	$105(DA)^{0.733}$	$380(DA)^{0.594}$	$724(DA)^{0.594}$
0.5	$891(DA)^{0.589}$	$661(DA)^{0.709}$	$120(DA)^{0.733}$	$447(DA)^{0.589}$	$891(DA)^{0.589}$
0.2	$1,072(DA)^{0.583}$	$794(DA)^{0.704}$	$138(DA)^{0.732}$	$550(DA)^{0.583}$	$1,148(DA)^{0.583}$

Table 4. NCDOT Hydraulic Guidelines 2016 in determining appropriate hydrologic method to use in estimating peak discharge.

0 < Drainage Area < 64 acres	64 < Drainage Area < 640 acres	DA > 640 acres
Rational Method	USGS Urban and Small Rural 2014	USGS Rural 2009

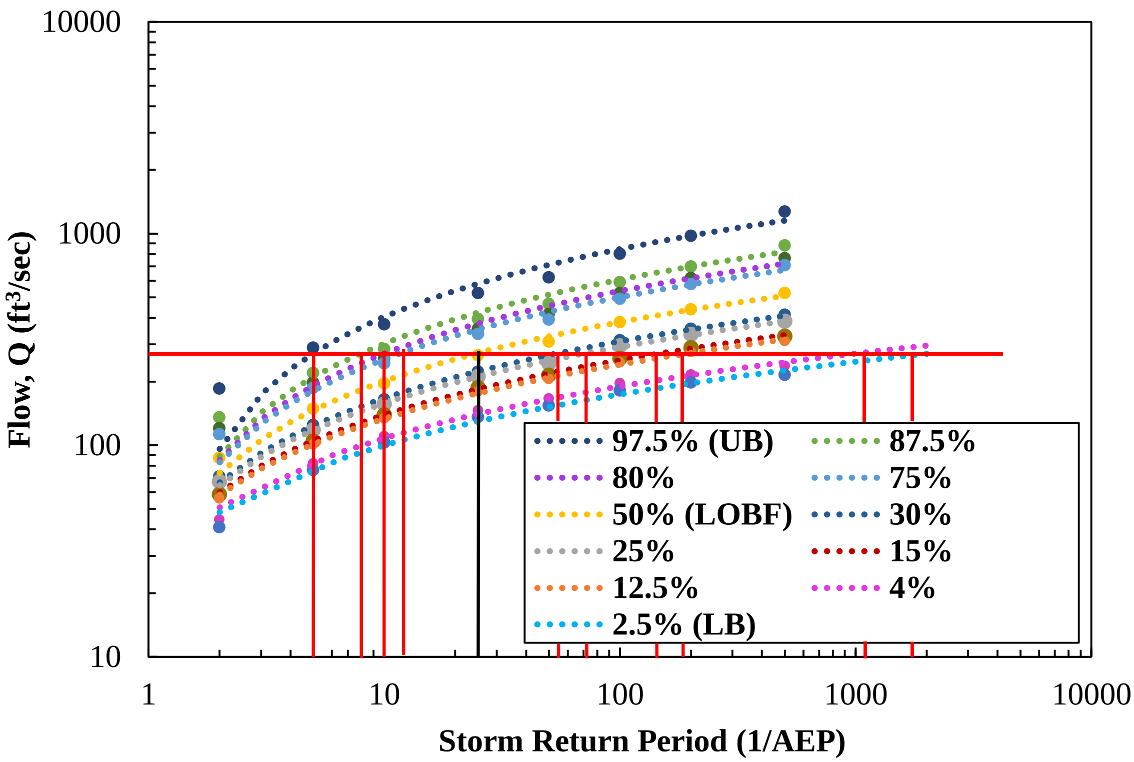

Figure 9. Relationship between flow values at different percentiles and storm return periods with when drainage area (DA) is 1 square mile.

The confidence intervals were used to estimate the flow values at different probabilities. For example, the line of best fit represents the 50th percentile estimate of the flow at a particular drainage area. However, the upper 95% confidence limit represents the 97.5th percentile of flow values for the same drainage area. Similarly, the lower 95% confidence limit represents the 2.5th percentile of flow values for the same drainage area. Subsequent examination of the connection between hydraulic analysis-derived overtopping flow values and corresponding percentiles was undertaken on a sample of nine (9) case study sites: 95-9-MF-M, 95-8-MF-M, 42-3-M, 53-7-MF, 53-5-M, 25-2-F, 95-2-MF, 42-1-MF and,8-4-MF. Figure 10 shows an example of the relationship between Headwater to Bed-to-Crown ratios at different probabilities.

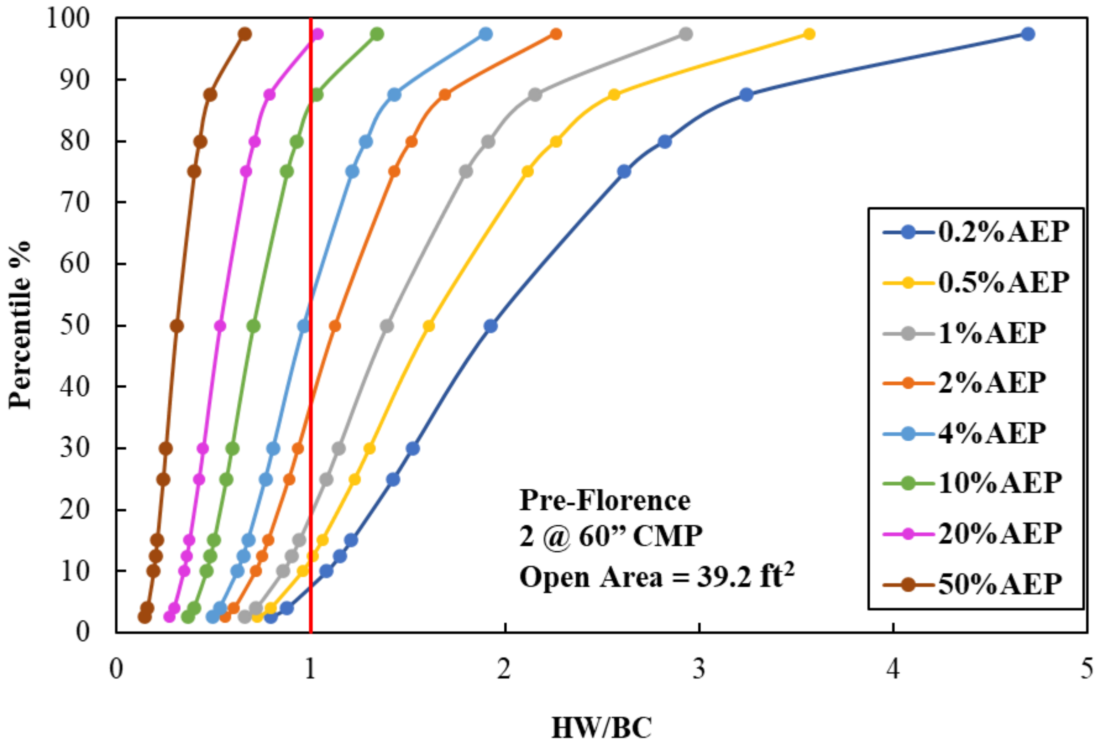

Figure 10. Probabilistic analysis of overtopping flow values across percentiles: case study 25-2-F before Hurricane Florence.

2.4. Identify Factors Contributing to Improved Performance

Multiple methods were used to identify the factors that contributed to pipe performance during the hurricanes. These methods spanned the range from qualitative to quantitative assessments.

2.4.1. Initial Evaluations

Initial observations from the Robeson County sites are summarized in Table 5. As shown in this table, for the initial evaluations, sites were grouped according to the rainfall ratio and the decision to upsize (or not) the pipes after one or both events. Most of the cases were classified as Group A-1 or A-2, which contain sites where the rainfall ratio was higher for Matthew than Florence and the pipe was upsized after one of the events. This approach did provide some insights, for example for Groups A-1 and A-2, it showed that some sites were identified as vulnerable after Matthew, but that replacement may not have been possible or was not deemed necessary after Matthew. In these cases, damage could again occur even with less total rainfall and (at least in some cases) replacement after Florence was prioritized. However, ultimately, this approach, relying on rainfall similarities/differences and pipe sizing decisions was deemed insufficient for more widespread study because it could not provide sufficient insights into the site specific characteristics and circumstances that contributed to vulnerability and damage. In this case, other elements of the design and conditions, such as type of soil, presence of headwalls, time of repair with respect to time of damage, etc., also need to be considered. The details and findings of this categorization will be discussed in subsequent sections of the study.

Table 5. Categorized case studies.

Group Name	Conditions	Assigned Cases
Group A-1	The rainfall ratio was higher for Matthew than Florence. The pipe was not changed/damaged after Matthew. The pipe was significantly upsized after Florence.	Cases 77-2-MF, 77-5-MF, 77-2-F, 77-4-F, 77-7-F, 77-8-F, and 77-9-F
Group A-2	The rainfall ratio was higher for Matthew than Florence. The pipe was upsized after Matthew. The pipe was not changed/damaged after Florence.	Cases 77-7-MF, 77-2-M, 77-3-M, 77-7-M, 77-8-M, 77-9-M, 77-10-M, 77-11-M, 77-13-M, 77-14-M, and 77-16-M
Group A-3	The rainfall ratio was higher for Matthew than Florence. The pipe was not changed after either event.	Case 77-1-F, 77-3-MF, and 77-15-M
Group B-1	The rainfall ratio was lower for Matthew than Florence. The pipe was upsized after Matthew. The pipe was not damaged or changed after Florence.	Cases 77-1-MF, 77-1-M, and 77-4-M
Group B-2	The rainfall ratio was lower for Matthew than Florence. The pipe was not changed after either event.	Case 77-4-MF and 77-3-F
Group C-1	The rainfall ratio was the same for Matthew and Florence. The pipe was upsized after Matthew. The pipe was not changed/damaged after Florence.	Cases 77-6-MF and 77-5-M
Group C-2	The rainfall ratio was the same for Matthew and Florence. The pipe was not changed after either event.	Cases 77-12-M
Non-Group	N/A	Case 77-6-M (pipe downsized after Matthew). 77-5-F and 77-6-F (Lack of sufficient information)

As part of the initial evaluations, the elevation profiles of the case study locations in Robeson County were also extracted using LiDAR data obtained from the North Carolina Spatial Data Download. The objective was to assess the configuration of the bed stream and potentially identify the floodplain. Understanding the characteristics of the bed stream in the case study locations could provide valuable insights, as there might be a correlation between the extent of damage during peak flow and the width of the bed stream. A wider streambed could lead to a higher accumulation of floodwater, increasing the area at risk of failure or erosion. However, this particular method was inconclusive due to limitations in the resolution of the LiDAR data. The data resolution was not fine enough to capture the detailed elevation from the bed to the crown and width of stream. Alternatively, the bed to crown of the current condition was determined in site visits which will be discussed in subsequent sections.

2.4.2. Interviews with Division Engineering Personnel

The research team conducted interviews with personnel from Divisions 2, 4, and 6 to understand their pipe repair and replacement practices. During these interviews the research team questioned the engineers about the failures and repairs at specific sites where sufficient information was not available in the database and repair records. The team also discussed the engineer's general practices regarding repairs, and condition assessment before and following the hurricanes.

2.4.3. Soil Analysis

Soil samples were collected within the downstream side of the shoulder for each of the sites visited. The research team evaluated each of these soil samples using index tests following the ASTM D2488-09, *Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The sites visited had a range of soil types, from well graded sands, which are considered to have high erodibility, to silts with medium or low erodibility. The information obtained from the soil classification was used as supplementary information in helping understand the influential parameters that contribute to the various damage levels, notably the soil erodibility. The erosion categories for the various soil types were determined using Briaud's erosion charts for geomaterials (Briaud et al., 2017) as shown in Figure 11 and Figure 12. In these figures, SP= poorly graded sand, SP-SM= poorly graded sand with silt, SM = silty sand, SW-SM= well graded sand with silt, SW = well graded sand, SC= clayey sand, ML= silt, and MH= elastic silts. The erosion categories ranged from I to III (I = Very High Erodibility, II = High Erodibility, III = Medium Erodibility and, IV = Low Erodibility).

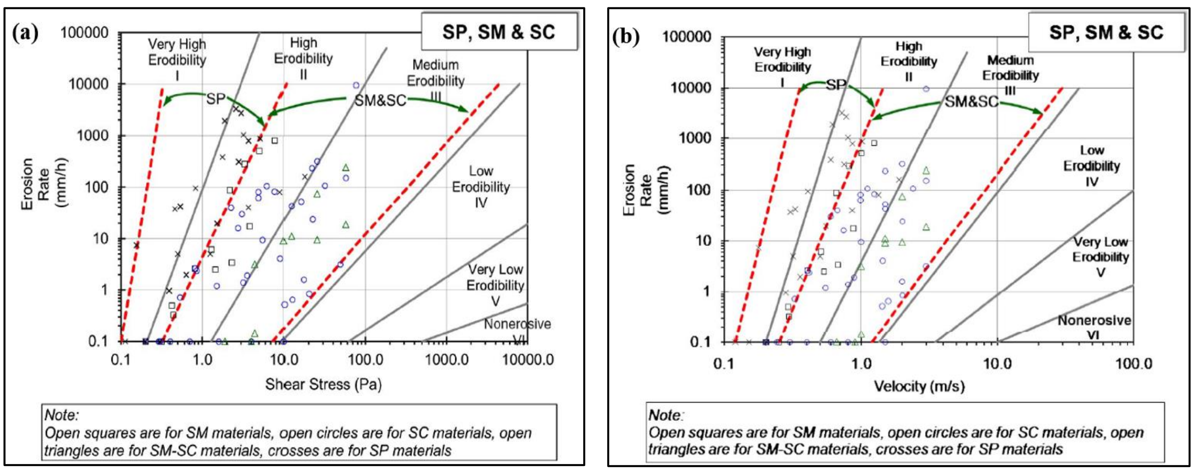

Figure 11. Erosion categories for SP, SM, and SC soils in: (a) the shear stress space and (b) velocity space (Briaud et al., 2017).

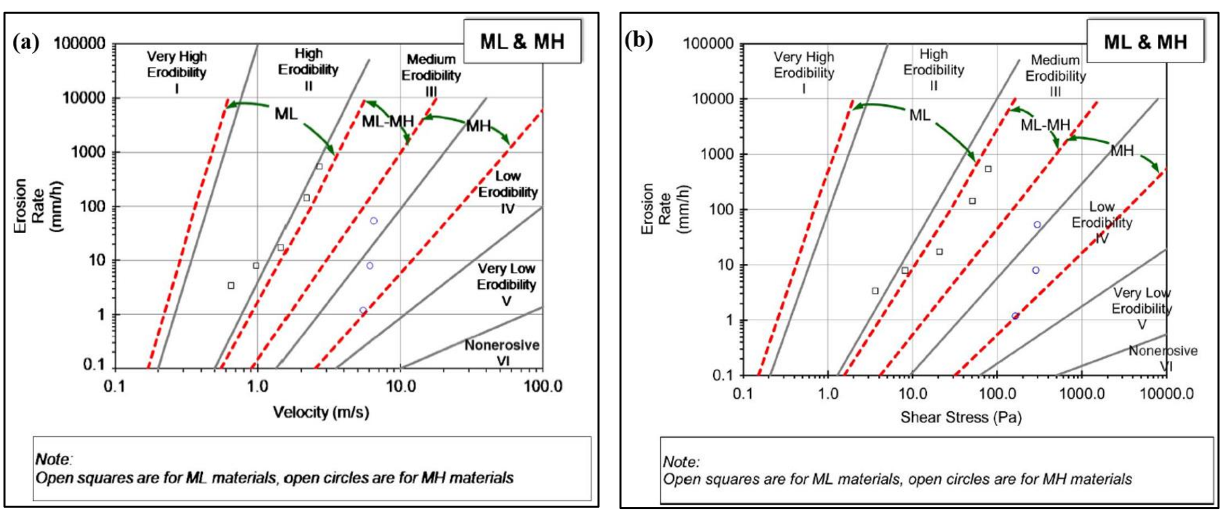

Figure 12. Erosion categories for ML and MH soils in: (a) the shear stress space and (b) velocity space (Briaud et al., 2017).

2.4.4. Vulnerability Analysis

<u>*Level of Damage*</u>

Damage assessment was carried out for each site on the basis of four categories; shoulder damage, pavement damage, pipe damage, and repair mobilization requirements. For each of these categories a score from zero (no damage) to three (highest damage) was assigned. Since all sites evaluated had some damage or another the lowest possible score for a site evaluated in this study was one and the highest possible score was 12, which corresponded to a complete washout of the roadway. The guidelines on assigning the level of damage are explained in Table 6.

Table 6. Damage level guidelines.

Component	**Damage Level**			
	0	**1**	**2**	**3**
Shoulder	No damage	Scour hole(s) present in shoulder.	Shoulder washed away on top of the pipe.	Shoulder washed on top of the structure + on areas beyond the location of pipe.
Pavement	No damage	Less than half of the roadway width is washed away.	Half of the roadway width is washed away.	Total width of roadway is washed away.
Pipe	No damage	Pipe is dislodged, eroded, the joints are separated, or otherwise damaged, but only within the shoulder area	Joint separation that requires pipe change and road closure/rusted pipes that were damaged in some way and needed to be changed.	Pipe is completely washed away and rendered nonfunctioning.
Mobilized Repair on Pavement	No repair needed	Can be repaired without specific equipment (e.g., dump truck, excavator, de-watering equipment, roller compactor, etc.).	Can be repaired with specific equipment.	Whole groundwork and heavy equipment is required.

Once defined, damage level components were plotted in form of radar plots to explore correlation between damage levels and structure characteristics. Examples of these plot are presented in Figure 13. In Figure 13(a) level of damage to four components of structure is plotted and it indicates that damage level of 1 is assigned to shoulder and damage level of zero to other components. Similarly, in Figure 13(b) damage level of 2 is assigned to all four components.

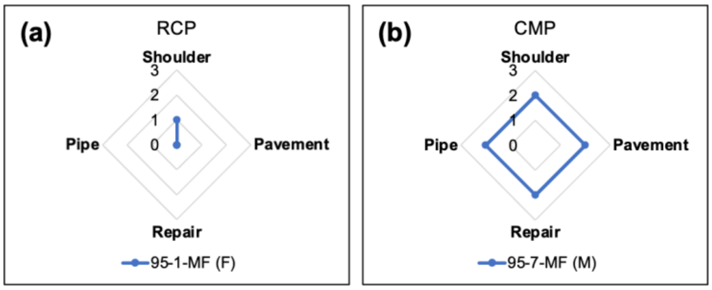

Figure 13. Level of damage in form of radar plot; (a) structure with RCP pipe in Site 95-1-MF(F), (b) structure with CMP pipe in Site 95-7-MF(M).

<u>*Vulnerability Score*</u>

A scoring system based on influential parameters (headwall, rip rap, beaver dam, swamp, material type, flood plain, structure type, channel erosion, cover, suitable design, rainfall intensity, soil characteristics, etc.) was established in order to develop a correlation between level of damage and assigned scores. A flowchart was developed to demonstrate the calculation of vulnerability score (Figure 14). As shown in Figure 14, if headwall, rip rap, and No. 57 Stone does not exist, for each factor, a value of one is added to the cumulative score. If a beaver dam, swamp, wide floodplain, erosion of channel, or low BTC exist, then a value of one is added to the cumulative score for each. To calculate the final score, values of HW/D, rainfall intensity (ratio of overall maximum 24-hr rainfall over 4% AEP rainfall), and soil erosion score (score developed in Section 2.4.3) are added together.

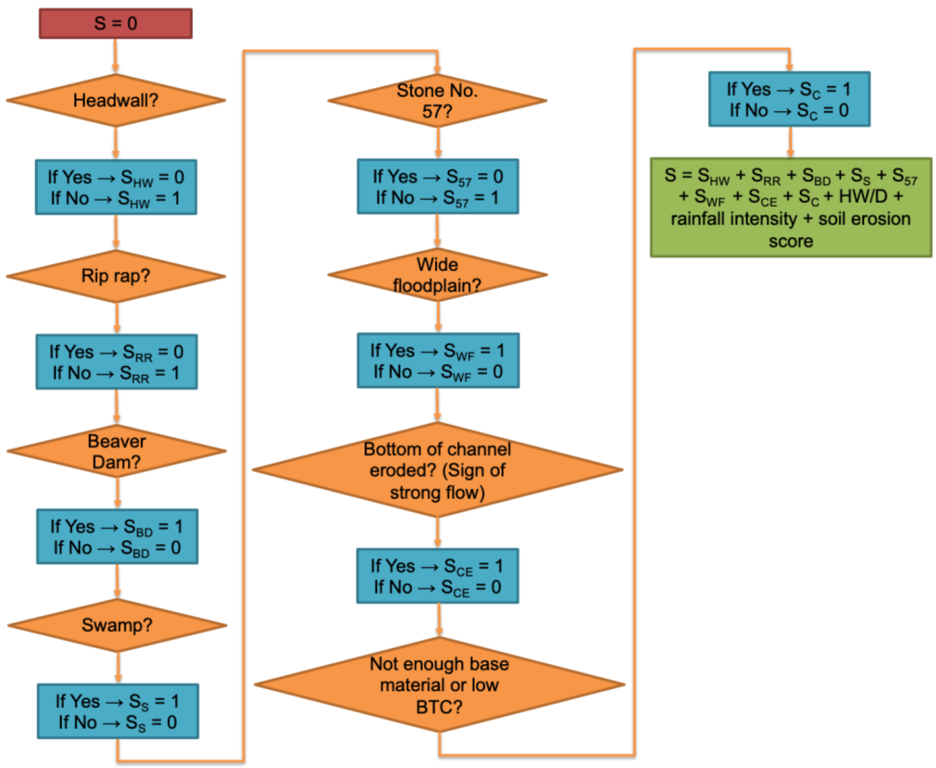

Figure 14. Flowchart to calculate vulnerability score.

Vulnerability score components were plotted in the form of radar plots to explore the correlation between scores and structural components. Examples of these plots are presented in Figure 15. As seen in this figure, the scores for components are plotted and in order to simplify the graphs, the HW/D is presented in form of color of the graph, i.e., the color of green and orange, shows the HW/D with respect to 25-year design event is less than 1.2 and higher than 1.2, respectively. Also, summation of scores for beaver dam, swamp, and wide floodplain is considered as one component called surroundings. In Figure 15(a) since headwall and enough cover do not exist, for each one score of 1 is assigned and since rip rap and No. 57 Stone exist and channel erosion do not exist, for each one score of zero is assigned. Score of 2 is assigned to surrounding features since swamp and wide floodplain exist in this site. For soil scores and rainfall intensity scores of 1.25 and 2.1 is assigned, respectively. In Figure 15(b) since headwall, rip rap and enough cover do not exist, for each one score of 1 is assigned and since No. 57 Stone exist and channel erosion do not exist, for each one score of zero is assigned. Score of 2 is assigned to surrounding features since swamp and wide floodplain exist in this site. For soil scores and rainfall intensity scores of 0.83 and 1.1 is assigned, respectively.

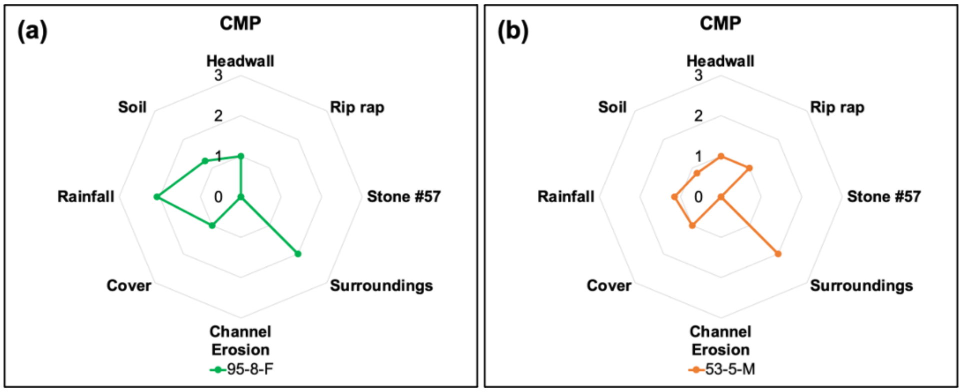

Figure 15. Vulnerability scores components in form of radar plot; (a) structure with CMP pipe in Site 95-8-F, (b) structure with CMP pipe in Site 53-5-M.

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3. RESULTS

This section presents the comprehensive results derived from various aspects of the study, including the analysis of site characteristics, information gathered during site visits, hydraulic analysis, insights from division engineers' experiences, and the vulnerability analysis. The results discussed in this section serve as a foundation for discussion on contributing factors and recommendations for improving the resiliency of the infrastructure.

3.1. Overview of Site Characteristics

Overall, the sites studied exhibited the following characteristics:

- Approximately 63 percent of the sites were situated in watersheds with an area of less than 0.5 square miles.
- During the respective event, about 75 percent of the sites did not have a headwall incorporated in the structure.
- The distribution of sites based on road types was as follows: 89.4 percent were located on secondary roads, 9.4 percent on NC routes, and 1.2 percent on US routes.
- While various types of pipes were damaged during the events, the most common types were CMP, followed by RCP pipes.
- Around 64 percent of the sites did not have rip rap within the structure during the event.
- Approximately 60 percent of the sites (excluding those in Johnston and Robeson County) were surrounded by influential features such as swamps, beaver dams, and wide floodplains.
- A qualitative assessment of damage photos revealed that in 44 percent of the sites (excluding those in Johnston and Robeson County), limited or possibly insufficient pipe cover was observed.
- In 59 percent of the sites (excluding those in Johnston and Robeson County), the upstream side channel near the pipe exhibited a trapezoidal shape. Although this parameter may not be considered a direct causal factor, it was noticed by the research team and may be worth considering in future NCDOT research and studies.

Johnston and Robeson County sites are excluded from some of these assessments because they were the first sites selected for study. The details of the site catalog process were still being developed at the time these sites were evaluated and so some details were not available.

3.2. Site Visits and Typical Results

Among the 138 selected sites, one site was located on a US route, 15 sites located on NC routes, and the rest of the sites were located on secondary routes. Eighty-five (85) of the 138 sites were visited and among these one site was located on US route, six sites were located on NC routes, and the rest of the sites were located on secondary routes. Visiting sites and investigating the collected data provided useful insights for the case study analyses. The highlights of findings are presented here. A summary of the evaluations is presented in Table C.2 in Appendix C.

An investigation of the archived data suggested that among the selected sites;

- In 12 sites in Robeson, Wayne, Lenoir, and Harnett counties complete washout occurred (95-4-MF, 95-8-M, 95-9-M, 53-6-MF, 53-6-M, 42-3-M, 77-1-MF, 77-7-MF, 77-1-M, 77-11-M, 77-12-M, and 77-15-M).

- In 42 sites, the type of structure was RCP and in 12 of these sites in Robeson, Wayne, Lenoir, Bladen, and Columbus counties the joint was separated, which may point to systemic issues with RCP pipes (95-1-MF, 95-2-MF, 95-6-MF, 95-3-F, 53-3-MF, 8-2-MF, 8-3-MF, 23-3-MF, 77-6-F, 77-2-M, 77-13-M, and 77-4-MF).
- In five of the sites in Wayne County where a headwall was present the backfill behind the headwall was washed away in one or both events (95-1-MF, 95-4-MF, 95-5-MF, 95-6-MF, and 95-6-M).

A summary of collected information for three sites is presented as example for each bullet point to demonstrate the observations from archived data.

3.2.1. Complete Washout (Example: Site 53-6-M)

This site is located on Falling Creek Road. It was damaged only after Hurricane Matthew and, the compiled information on this site is summarized in Table 7. The damage photos after Hurricane Matthew are shown in Figure 16. As shown in this figure, the official description of damage is quoted below.

> *The flooding caused by Hurricane Matthew inundated the roadway causing complete pipe failure resulting in the washing out the existing pipe structure and roadway shoulders. Pipe bedding material, debris, asphalt remnants were displaced downstream.*

Table 7. Information on Site 53-6-M.

Pre – Matthew Structure	2 @ 60" CMP
Post – Matthew Proposed Structure	Alt. 1: 1 @ 13’1” x 8’4” STR.PL.PA w/HW Alt. 2: 2 @ 84" w/HW
Post – Matthew Actual Structure	1 @ 156" x 96" CAP w/HW
Costs	$416,041

Figure 16. Photos of damage after Hurricane Matthew in Site 53-6-M.

3.2.2. Possible Issue with RCP Pipes (Example: Site 77-6-F)

This site is located on Townsend Road. It was damaged only after Hurricane Florence and similar to the previous sites described, the compiled information on this site is summarized in Table 8. The damage photos after Hurricane Florence are shown in Figure 17 and the current photos are shown in Figure 18. As shown in Figure 17 the official description of damage is quoted below.

> *Last joint appears to be separated and shoulder has washed out through pipe.*

As shown in Figure 18, it seems that the pipe was not fixed and looks the same as it was after Florence. Last part is disjointed in the inlet side and scours holes can be seen on the shoulder near the second joint.

This site is shown because it is emblematic of the observations made at other RCP sites where end separation was very common. All 12 sites showed at least one segment of separated pipe. In some cases, the pipe had separated but showed significant erosion around what appeared to be the original layout of the pipe such that none of the pipes physically located under the roadway had separated. Out of 12 sites, in two sites the damage was on inlet sides, in six sites the damage was on both sides or middle and in four sites the damage was on the outlet side.

Table 8. Information on Site 77-6-F.

Pre – Florence Structure	1 @ 36" RCP
Post – Florence Proposed Structure	N/A
Post – Florence Actual Structure	1 @ 36" RCP
Costs	$2,449.2

Figure 17. Photos of damage after Hurricane Florence in Site 77-6-F.

Figure 18. Current photos of Site 77-6-F.

3.2.3. Backfill Washout behind Headwall (Example: Site 95-4-MF)

This site is located on NC 55 in Wayne County, and it was damaged after Hurricanes Matthew and Florence. The compiled information on Site 95-4-MF is summarized in Table 9. The photos of damage after Hurricane Matthew are shown in Figure 19. The photos of damage after Hurricane

Florence on the outlet side are shown in Figure 20. The official description of Matthew and Florence damage is quoted below.

> *Hurricane Matthew: Culvert washed out. 34x42x10 fill material for shoulder. ABC and replace the double RCBC. Pull surface over patch.*

> *Hurricane Florence: Permanent repair (9/19/18). Shoulder washed on top of headwall and damaged to adjacent shoulders. Backfill shoulder with compacted material.*

In this site, a substantial repair was performed after Hurricane Matthew to replace the RCBC with a CMPA with headwall. However, damage still occurred during Hurricane Florence when the backfill behind the headwall was washed away. This damage was less substantial than during Matthew but was still evident. In the materials used in the structure repairs after Hurricane Matthew, No. 57 Stone was listed, but it is not evident exactly whether that material was actually used in the backfill or used elsewhere in the repair process. Washout of backfill behind the headwall was observed in five Wayne County sites among those visited. For three of these sites the washout was seen on the inlet side and in two sites it was seen on the outlet side. Among sites investigated, this type of washout was not seen in other counties.

Table 9. Information on Site 95-4-MF.

Pre – Matthew Structure	2 @ 6'X6' (72" X 72") RCBC
Post – Matthew Proposed Structure	1 @ 117"x79" CMPA w/HW
Post – Matthew/Pre – Florence Actual Structure	1 @ 96" CMP w/HW
Costs after Matthew	$173,485
Post – Florence Proposed Structure	N/A
Post – Florence Actual Structure	1 @ 96"x74" CMPA w/HW
Costs after Florence	$14,476

Figure 19. Photos of damage after Hurricane Matthew in Site 95-4-MF.

Figure 20. Photos of damage after Hurricane Florence in Site 95-4-MF.

An investigation of the collected data on site visits suggested that among the visited sites,

- The information recorded for proposed pipes after Florence in 20 sites in Robeson, Wayne, Lenoir, Harnett, Columbus, Cumberland, and Bladen counties is not consistent with what is observed in the site visits in terms of size and/or type of pipe and corresponding costs (i.e., the amount of cost does not seem to cover the expenses for the extent of repair) (95-2-MF, 95-3-MF, 95-4-F, 95-5-F, 95-6-F, 95-7-F, 95-9-F, 53-4-MF, 53-5-MF, 53-6-MF, 42-1-MF, 25-2-F, 23-4-F, 8-2-F, 8-1-MF, 8-2-MF, 8-6-F, 77-2-F, 77-3-MF, and 77-4-MF).
- In four sites in Robeson, Wayne, and Cumberland counties beaver dams were observed on the site, could possibly contribute to the vulnerability (25-1-MF, 95-6-M, 77-5-M, and 77-10-M).
- Evidence of ongoing erosion on shoulder/embankment, pavement, pipe, behind headwall, joints, etc. was observed in 40 sites in all eight counties. In M-only sites, even though they were not flagged as damaged after Florence there was clear evidence of continued erosion since Matthew, which could cause issues in future events (51-4-MF, 95-1-MF, 95-2-MF, 95-6-MF, 95-7-M, 95-9-M, 95-3-M, 95-6-M, 95-1-F, 95-2-F, 95-4-F, 95-7-F, 95-11-F, 53-1-MF, 53-3-MF, 53-4-MF, 53-5-MF, 53-9-M, 53-3-M, 53-5-M, 53-6-M, 53-2-F, 42-1-F, 25-1-MF, 25-2-M, 23-1-MF, 23-2-MF, 23-5-MF, 23-4-F, 23-5-M, 8-4-M, 77-2-M, 77-3-M, 77-4-M, 77-5-F, 77-6-F, 77-3-MF, 77-4-MF, 77-10-M, and 77-13-M).
- In 5 sites in Johnston, Wayne, and Lenoir counties a localized depression from excessive erosion (i.e., "blowhole") was observed on the bottom of channel near the culvert outlet (51-4-MF, 95-11-F, 53-4-MF, 53-5-MF, and 53-6-M).
- In 8 sites in Johnston, Wayne, Lenoir, Harnett, and Columbus counties part of their rip rap was lost or dislocated (51-4-MF, 95-1-F, 95-2-MF, 95-3-M, 53-1-MF, 53-4-MF, 42-1-MF, and 23-2-MF).

3.2.4. Inconsistency with Survey 123 (Example 42-1-MF)

This site is located on Hodges Chapel Road, and it was damaged after both Hurricane Matthew and Florence. The compiled information for this site is presented in Table 10. The photos of damage after Hurricane Matthew on the inlet side is shown in Figure 21. The photos of damage after Hurricane Florence also on the inlet side is shown in Figure 22. The official description of damage is quoted below.

Hurricane Matthew: The flood waters from Hurricane Matthew eroded and washed around a triple (3) 48-inch diameter x 40 linear feet CAAP culverts. The pipes were rendered nonfunctioning which resulted when the supporting soils lost the ability to maintain the designed location of the pipe. This also allowed the pipes to separate exacerbating the saturation and erosion problem. An area of asphalt road base material measuring 20 ft. x 12 ft. x 11 inches became highly saturated, eroded and washed away. These conditions created voids and the material lost the ability to support the asphalt road surface. An area of asphalt road pavement measuring 20 ft. x 12 ft. x 5 inches was damaged when the supporting base material became saturated and was unable to support it. The lost support caused the road surface to wash away/erode which created an unsafe traveling surface.

Hurricane Florence: Hurricane Florence DR 4393-NC, flood waters with velocity eroded and washed around a 95-inch diameter 67-inch triple barrel CMP pipe culvert, on SR 1709, Hodge Chapel Road. Flood waters caused erosion of unclassified fill material on the shoulder and embankment areas and damaged the roadway bed and the asphalt roadway pavement surface and rip rap. Damages include the following: -Unclassified fill on the shoulders and embankments, measuring 54 ft. x 14 ft. x 4 ft., -Aggregate roadway base material measuring 25 ft. x 6 ft. x 4 ft. -Asphalt roadway surfaced course measuring 25 ft x 6 ft x 4 in. -Rip Rap measuring 12 ft x 15 ft.

Table 10. Information on Site 42-1-MF.

Pre – Matthew Structure	3@ 48" CMP or CAAP
Post – Matthew Proposed Structure	1 @ 137" x 87" CMPA w/HW
Post – Matthew/Pre – Florence Actual Structure	3 @ 95" x 67" CMPA (*Possibly the structure pre-Matthew is the same*)
Costs after Matthew	$13,592
Post – Florence Proposed Structure	Alt 1: 25'2" x 7'0" ABC w/ HW, Buried 1-ft Alt 2: 2 @ 11'5" x 7'1" SPPA w/ HW, Buried 1-ft
Post – Florence Actual Structure	1 @ 78" x 252" CMPA w/HW
Costs after Florence	$528,964

Figure 21. Photos of damage after Hurricane Matthew in Site 42-1-MF.

Figure 22. Photos of damage after Hurricane Florence in Site 42-1-MF.

In this site, it is listed that prior to Matthew, three CMP pipes with 48 inch of diameter was in place and after Matthew, three CMPA pipe with 95 x 67 inch were placed and these types of pipes were also explained in the damage descriptions. Comparing the photos from Matthew and Florence, it seems like it is the same pipes. The amount of cost does not seem to cover the pipe replacements. Also, the pipe that it is currently in place (1 CMPA pipe with 76 x 252 inch with headwall) does not match with the type and size of proposed pipes listed in Table 10. This is an example of inconsistencies observed between archived and collected information.

3.2.5. Presence of Beaver Dam (Example: Site 77-10-M)

This site is located on Oakgrove Church Road, and it was also damaged only after Hurricane Matthew. The compiled information for this site is presented in Table 11 and the photos of damage after Hurricane Matthew are shown in Figure 23. The damage at this site occurred on the inlet side. The official description of damage is quoted below.

> *Flood waters with velocity eroded and washed away an area of the shoulder and or embankment measuring 45 ft. x 22 ft. x 2 ft. with an area of vegetation (mulch and grass) measuring 45 ft. x 22 ft. also eroding and washing away. This area had been previously established and maintained for erosion control. Flood waters with velocity eroded and carried away the highly saturated soils creating voids in the fill material between asphalt and aluminum box culvert measuring 48 in x 45 ft. The existing culvert was distorted and damaged beyond repair. An area of road base material measuring 45 ft. x 22 ft. x 4 ft. became highly saturated, eroded and washed away. These conditions created voids and the material lost the ability to support the asphalt road surface. An area of asphalt road measuring 45 ft. x 22 ft. x 6 inches was damaged when supporting base material became saturated and was unable to support the asphalt road. The lost support caused the road surface to wash away which cause an unsafe traveling surface.*

Table 11. Information on Site 77-10-M.

Pre – Matthew Structure	1 @ 24'1" x 6'6" AABC
Post – Matthew Proposed Structure	1 @ 24'4" x 8'2" ABC w/HW
Post – Matthew Actual Structure	1 @ 142" x 91" CMPA w/HW
Costs	$646,150.9

Figure 23. Photos of damage after Hurricane Matthew in Site 77-10-M.

Photographs of the current condition at the site are shown in Figure 24. Measurements taken during the site visit confirm the size listed in Table 11 currently exists. The visit also found the existence of what is believed to be a very large beaver dam on the inlet side that appeared to have at least two major branches; one parallel to the culvert (visible in the first panel) and another perpendicular to the culvert extending to the wood line (not clearly visible in the first panel photo). The red lines in the first pane are drawn to help show where the dam exists. At this site, the ponded water extended as far to the left as was visible through the woods. In addition, as the second panel shows, it also appeared that hole approximately 1 foot deep existed occurring at the edge of the headwall on the side where the perpendicular branch of the beaver dam existed. Some ponding of the water on that side of the culvert can be observed in the third panel of Figure 24.

Figure 24. Current photos of Site 77-10-M.

3.2.6. Evidence of Continued Erosion (Example: 77-4-M)

This site is located on Mt. Moriah Church Road, and it was damaged only after Hurricane Matthew. The compiled information for this site is presented in Table 12. The photos of damage on the inlet side after Hurricane Matthew are shown in Figure 25 and photos of the current condition also on the inlet side are shown in Figure 26. In this site, evidence of erosion can be seen around the pipe and on the shoulder even though it was not flagged as damaged after Florence (as highlighted by yellow arrows in Figure 25 and Figure 26). Evidence of continued erosion was observed in at least nine sites among sites visited in Robeson County. In most cases, the sites were only reported as damaged after Matthew (as was the case with the site shown here), but that was not a consistent pattern, some cases were damaged only after Florence and some cases were damaged after both events. Among the nine sites, in six sites the evidence of erosion is seen on the inlet side and in one site on both inlet and outlet site.

Table 12. Information on Site 77-4-M.

Pre – Matthew Structure	1 @ 48" CMP
Post – Matthew Proposed Structure	2 @ 66" CMP w/HW
Post – Matthew Actual Structure	1 @ 48" CMP
Costs	$1,157.15

Figure 25. Photos of damage after Hurricane Matthew in Site 77-4-M.

Figure 26. Current photos of Site 77-4-M.

3.2.7. Blowhole in Channel (Example: Site 53-4-MF)

This site is located in Lenoir County on Dalys Chapel Road, and it was damaged after Hurricanes Matthew and Florence. The compiled information on Site 53-4-MF is summarized in Table 13. The photos of damage after Hurricane Matthew on the outlet side are shown in Figure 27. The photos of damage after Hurricane Florence on the outlet side are shown in Figure 28. The official description of Matthew and Florence damage is quoted below.

> *Hurricane Matthew: Flooding caused by the torrential rainfall (exceeding 10 inches in Lenoir County) produced by Hurricane Matthew resulted in the washout of the roadway shoulder for approximately 80' and pavement approximately 45' along the existing pipe structure. Some fill material washed downstream. There was also fallen trees across this site and exposed utilities.*
>
> *Hurricane Florence: Flooding caused by the torrential rainfall (exceeding 18 inches in Lenoir County) produced by Hurricane Florence (on September 13, 2018) resulted in joint failures and pavement and material loss which caused further pipe damage.*

The current photos of this site on the outlet side are shown in Figure 29. In this site, based on investigating the current photos (identified with red arrows), evidence of erosion (blowhole) on the bottom of channel was observed. Also, in this part of the channel, rip rap is dislocated into the channel (as identified with a red arrow in Figure 29). This type of erosion is similar to the previously reported site 51-4-MF (Massey-Holt Road in Johnston County). This erosion could show that during the summer (time of site visits) the channel is dry, but during extreme precipitation events the floodwater enters the channel and culvert with a high velocity and might create a vortex-like flow, which could cause ongoing damage and make the structure vulnerable over time. Among sites visited, two sites showed this type of erosion on the channel and both of them were damaged in Hurricanes Matthew and Florence. In one of these sites, the erosion was seen on the outlet (Site 53-4-MF) and in the other site (Site 53-5-MF) it was seen on the inlet side.

Table 13. Information on Site 53-4-MF.

Pre – Matthew Structure	2 @ 60" CMP
Post – Matthew Proposed Structure	Alt 1: 1 @ 112" x 75" CAPA w/HW Alt 2: 2 @ 66' w/HW
Post – Matthew/Pre – Florence Actual Structure	2 @ 60" CMP
Costs after Matthew	$60,655
Post – Florence Proposed Structure	Alt 1: 1 @ 11'-5" x 7'-1" CASPPA w/HW Alt 2: 2 @ 66" Pipe w/HW Alt 3: 2 @ 81" x 59" CAPA w/HW
Post – Florence Actual Structure	1 @ 79" x 134" CMPA w/HW
Costs after Florence	$238,645

Figure 27. Photos of damage after Hurricane Matthew in Site 53-4-MF.

Figure 28. Photos of damage after Hurricane Florence in Site 53-4-MF.

Figure 29. Current photo of Site 53-4-MF.

3.2.8. Rip Rap Lost or Dislocated (Example: Site 95-3-M)

This site is located on Wayne Memorial Drive (Wayne County), and it was damaged only after Hurricane Matthew. The compiled information on Site 95-3-M is summarized in Table 14. The photos of damage after Hurricane Matthew are shown in Figure 30. The official description of Matthew damage is quoted below.

> *The flood waters from Hurricane Matthew washed around and dislodged twin 84"dia x 60 linear ft of CMPs. The event eroded and washed away 20 ft long x 6 ft wide x 4 ft deep roadbed/embankment/shoulder with a 2 to 1 slope. The event damaged an area of asphalt measuring 2 ft long x 20 ft wide x 6 inches deep of asphalt The event damaged 20 ft x 6 ft of vegetation that had been established for erosion control.*

The current photos of this site on inlet and outlet sides are shown in Figure 31. As highlighted in this figure, part of the rip rap has been lost on the inlet side and on the outlet side part of rip rap is dislocated into the channel. As shown in Figure 30, dislocation of rip rap was also seen after Hurricane Matthew. This erosion could be due to issue of the placement of rip rap, size distribution of rip rap, or high velocity flow. Among the sites visited, this type of erosion was observed in eight site and six of these were damaged in both Hurricanes Matthew and Florence. In four of these sites the erosion was seen on the outlet, in three other site it was seen on the inlet side, and in one site it was seen on both sides.

Table 14. Information on Site 95-3-M.

Pre – Matthew Structure	2 @ 84" CMP
Post – Matthew Proposed Structure	Alt 1: 2 @ 112"x75" CMPA w/HW. Alt 2: 1 @ 13'-11" x 8'-5" AASPPA (requested by Div.)
Post – Matthew Actual Structure	1 @ 171" x 115" CMPA w/HW
Costs after Matthew	$421,593

Figure 30. Photos of damage after Hurricane Matthew in Site 95-3-M.

Figure 31. Current photos of Site 95-3-M.

3.3. Hydraulic Analysis of Site Performance

3.3.1. Base Peak Discharge Results

Results obtained from the hydraulic analysis involved predicting the headwater depths, tailwater and/or critical velocity using both HDS-5 and HY-8 models. Table 15 shows a few examples of the headwater to diameter (HW/D) ratios resulting from these predictions for the peak design discharge levels and structures that existed pre-Matthew, post-Matthew/pre-Florence, and post-Florence. A direct comparison with the outputs from HDS-5 and HY-8 was completed to evaluate the similarities and differences in analysis outcomes for all case study sites according to the overtopping criteria of HW/D ≤ 1.20.

Some minor differences are noted when comparing the base discharges results from the two software programs, as shown in Table 15. For example, in cases 95-1-MF, 53-4-MF, and 77-1-MF, their Post-Matthew/Pre-Florence structures have HW/D ≤ 1.20 (adequately designed) for HDS-5 but have HW/D > 1.20 (under designed) in HY-8. Though the HY-8 values for cases 95-1-MF, 53-4-MF, and 77-1-MF are above the criteria for adequate design, as discussed previously, the number of inputs for HY-8 greatly outnumbers the inputs for HDS-5, and any one of those inputs could account for this difference between the two analyses. One input that could cause the HY-8 values to be slightly higher than the HDS-5 value is that the HY-8 model incorporates the length of the pipe into the analysis, whereas HDS-5 does not. Despite the differences in results that HY-8 yields due to incorporating more input values, the HW/D values for the peak design discharge during hurricanes Matthew and Florence seems to be unusually high for case 77-1-MF and 42-1-F respectively as shown in Table 15. The high HW/D can be attributed to insufficient culvert size since before hurricane Matthew, Site 77-1-MF conveying 681 ft^3/s of water had a single 36″ reinforced concrete pipe while Site 42-1-F conveying 310.7 ft^3/s had a single 36″ corrugated metal pipe (CMP). Due to the culvert being under sized by such a substantial amount, high water levels were calculated upstream of the culvert. Upgrading the culvert to a 259″ by 59″ aluminum alloy box culvert in case 77-1-MF (Post-Matthew), reduced the HW/D value to 1.09 in HDS-5 and from 4.61 to 1.32 in HY-8. Case 42-1-F did not have adequate bed to crown height; therefore, the number of barrels was increased to 3, 36″ CMPs to sufficiently convey flow of water.

Table 15. Results of analysis completed utilizing HDS-5 and HY-8 for base peak discharge some identified sites in Robeson County (red cells have calculated HW/D > 1.20 and green cells have calculated HW/D ≤ 1.20).

Case ID	Street Name	Base Discharge HDS-5 HW/D			Base Discharge HY-8 HW/D		
		Base Matthew	Base Florence	Base Current	Base Matthew	Base Florence	Base Current
95-1-MF	Polly Watson Rd	1.04	1.01	1.09	1.8	1.8	1.13
95-5-MF	Corbett Hill Rd	0.88	0.53	0.53	0.95	0.6	0.59
95-4-M	James Hinson Rd	1.63	1.44	1.44	1.62	1.46	1.46
53-4-MF	Dalys Chapel Rd	1.01	1.01	0.71	1.24	1.24	0.78
53-6-MF	NC 903	0.65	0.65	1.2	0.31	0.27	1.01
53-7-MF	Davis Mill Rd	2.61	0.76	0.83	2.12	0.83	0.9
53-5-MF	Eric Sparrow Rd	1.98	1.98	1.55	2.17	2.17	1.29
42-1-MF	Harnett	2.5	0.69	1.01	2.68	0.91	0.8
42-1-F	Wiry Rd	-	36.15	2.36	-	2.43	1.64
25-1-MF	Cumberland	-	0.60	0.49	-	0.68	0.57
77-1-MF	Smith Mill	123.8	1.09	1.09	4.61	1.32	1.32
77- 3-F	Fayetteville Rd	3.61	3.61	3.61	2.19	2.19	2.19

In addition, there are some interesting sites to consider such as cases 95-5-MF, and 53-6-MF, which are adequately designed according to both hydraulic analyses, while cases 95-4-M, 53-5-MF, and 77-3-F are under designed for both hydraulic analyses. Cases 53-7-MF, and 42-1-MF on the other hand were damaged in Matthew, and both upgraded following that damage, but neither was re-damaged in Florence. Hydraulic analysis of HY-8 and HDS-5 indicated both were adequately designed in their post-Matthew/pre-Florence structure using the base peak discharge values. It was then considered that the re-designing of these two sites following Matthew was sufficient in providing additional resilience when Florence occurred. These two sites demonstrate what the NCDOT was hoping to determine, that some of the design practices put in place for Matthew aided in the state's infrastructure resilience for following storm events. A list of all sites analyzed for base peak discharge using HDS-5 and HY-8 is provided in Table D.1 in Appendix D.

As shown in Table 16, using the HDS-5 and HY-8 base peak discharge values (design flows), the research team further determined the proportion of structures that were either adequately designed or under designed given current design standards. For all the structures damaged in both Hurricanes Matthew and Florence (MF-sites), approximately 72%, 55%, and 33% of the sites were under-designed for the Matthew, Florence, and Current base flow estimates respectively. While 28%, 45%, and 66% of all the MF sites were adequately designed for the Matthew, Florence, and Current base flow estimates respectively. A similar trend is observed in the structures damaged in either Hurricane Florence only sites (F-sites) or Hurricane Matthew only (M-sites), with the proportion of under designed structures decreasing from the Pre-Matthew/Pre-Florence events to Post-Matthew/Post-Florence structures. The proportion of adequately designed structures also increased from the Pre-Matthew/Pre-Florence structures to Post-Matthew/Post-Florence structures. As earlier noted, the research team attributed the decreasing proportion of under-designed pipes or the increasing proportion of adequately designed pipes from hurricane Matthew to Florence and finally to current structures to the re-designing (upgrading the pipe sizes and/or changing the structure as well as inclusion of headwalls etc.) of the study sites.

Table 16. Proportion of culverts that were adequately designed and under designed with respect to flooding event.

Site Type	Timing	Base Flow HDS-5		Base Flow HY-8	
		Under-designed	Adequately designed	Under-designed	Adequately designed
MF	Pre-Matthew	72%	28%	76%	24%
	Pre-Florence	55%	45%	62%	38%
	Current	34%	66%	38%	62%
M	Pre-Matthew	64%	36%	64%	36%
	Current	27%	73%	33%	77%
F	Pre-Florence	74%	26%	78%	22%
	Current	37%	63%	37%	63%

3.3.2. Peak Discharge Results Adjusted using Ratios 1 and 2

The research team also estimated the potential flow these structures experienced during the storms. Recall, Ratio 1 (R1) is determined using the NOAA definition of 24-hr event (starting at 12:00 AM GMT or 7:00 AM ET, going until 6:59 AM ET the following calendar day) and Ratio 2 (R2) is calculated from the heaviest 24-hr period. Table 17 shows the sites that were estimated to have overtopped in either hurricane Matthew or Florence calculated using their respective definitions of the 24-hr precipitation levels. For HY-8, almost all sites from the sample selected for the purposes of this report, had HW/D > 1.20 for adjusted peak discharge values, except cases 53-6-MF (Matthew R1 and R2), 95-5-MF and 42-1-MF (Florence R1 and R2), which were adequately sized for the respective storms. Whereas for HDS-5, there were only five combinations of adjusted peak discharge values that provided HW/D values ≤ 1.20.

Table 17. Results of analysis completed utilizing HDS-5 and HY-8 for adjusted peak discharge for some identified sites in Divisions 1, 2, 4, and 5 (red cells have calculated HW/D > 1.20 and green cells have calculated HW/D ≤ 1.20).

Case ID	Street Name	Adjusted Discharge HDS-5 HW/D				Adjusted Discharge HY-8 HW/D			
		Matthew R1	Matthew R2	Florence R1	Florence R2	Matthew R1	Matthew R2	Florence R1	Florence R2
95-1-MF	Polly Watson Rd	1.59	1.59	0.94	0.94	2.04	2.04	1.65	1.65
95-5-MF	Corbett Hill Rd	1.62	1.61	0.68	0.68	1.43	1.43	0.75	0.75
95-4-M	James Hinson Rd	3	3.5	2.72	2.72	1.87	1.9	1.83	1.83
53-4-MF	Dalys Chapel Rd	1.14	1.14	2.01	2.01	1.43	1.43	2.07	2.07
53-6-MF	NC 903	0.73	0.73	1.08	1.08	1.18	1.18	1.69	1.69
53-7-MF	Davis Mill Rd	3.07	3.07	1.58	1.58	2.16	2.16	1.22	1.22
53-5-MF	Eric Sparrow Rd	2.31	2.31	5.08	5.08	2.22	2.22	2.48	2.48
42-1-MF	Harnett	3.37	4.4	0.65	0.65	2.87	3.02	0.85	0.85
42-1-F	Wine Rd	-	-	20.13	36.15	-	-	2.22	2.43
77- 1-MF	Smith Mill	243.90	377.90	1.84	2.59	4.99	5.28	2.40	2.76
77- 3-F	Fayetteville Rd	6.08	8.88	9.77	12.32	2.32	2.42	2.44	2.50

Comparing the values from Table 15 and Table 17, it can be seen that for Case 53-7-MF, the structure was repaired after Matthew, and the new structure was adequately designed for the base peak discharge. However, when assessing the structures for the estimated storm even flows it is found that case 53-7-MF was prone to overtopping with HW/D values of 3.07, and 1.58, for Matthew R1 and R2 respectively, and 2.16 and 1.22 for Florence R1 and R2 respectively. As shown in Table 15, using base peak discharge values, case 95-5-MF was adequately designed using both HDS-5 and HY-8. However, using adjusted peak discharge flow values used to estimate potential flows during each storm as shown in Table 17, case 95-5-MF was under-sized during Hurricane Matthew for both HDS-5 and HY-8 with HW/D values as 1.62 and 1.43 respectively. A list of all sites analyzed for adjusted peak discharge using HDS-5 and HY-8 is provided in Table D.2 in Appendix D.

In summary, as shown in Table 18, for all the structures damaged in both Hurricanes Matthew and Florence (MF-sites), approximately 79%, and 69% of the sites had under-sized structures for the Hurricane Matthew and Hurricane Florence respectively. Similarly, for both the M-sites and F-sites, approximately 67%, and 81% culverts were undersized for Hurricanes Matthew and Florence respectively. Comparing values from Table 16 and Table 18 shows that the proportion of under-sized culverts is slightly higher than the proportion of under designed culverts which suggests that the flow levels in some sites greatly exceeded (about 200 and 500-year return interval) the design during Hurricane Matthew and Florence.

Table 18. Proportion of culverts that were adequately sized and under-sized with respect to flooding event.

Site Type	Timing	Base Flow HDS-5		Base Flow HY-8	
		Under-sized	Adequately sized	Under-sized	Adequately sized
MF	Pre-Matthew	79%	21%	79%	21%
	Pre-Florence	69%	31%	79%	21%
M	Pre-Matthew	67%	33%	73%	27%
F	Pre-Florence	81%	19%	81%	19%

3.3.3. Damage Level Correlations Results

In addition to assessing whether the structures were under-sized or adequately designed for specific storms, other parameters were evaluated; Headwater depth to Bed-to-Crown ratio (HW/BC), drainage area (DA), Bed-to-Crown distance (BC), and backfill depth. As noted in the methodology, the exploration of these parameters was conducted on the basis of flows estimated using the design event. Of the 85 sites that were visited, 53 were considered for the analysis of potential failure pathways and correlation between damage level and design factors. Not all sites could be evaluated because of limitations in the data that could be derived from both hydro reports and sites visits. However, from these 53 sites, a total of 73 different conditions were considered since some sites were damaged in both hurricane Matthew and Florence.

The first analysis evaluated the relationship between culvert damage level HW/BC ratio. In 35 of the 73 cases considered, overtopping (HW/BC ≥ 1) was estimated to occur during the design event: 12 RCPs, 19 CMPs, three CMPAs, and one HDPE culvert. As shown in Table 19, the results of the analysis indicated that 61% of the cases where overtopping is estimated to occur during the design event, had a damage level of 2 for pipe damage, 58% had a damage level of 1 for pavement damage, and 40% had a damage level of 3 for shoulder damage. These findings suggest that calculated overtopping potential is a weak indicator of potential damage during a storm, particularly for pipe and pavement damage. Scatter plots showing this correlation are shown in Figure D.13 in Appendix D.

Table 19. Percentage of culverts overtopped in the correlation between Damage Level and HW/BC.

Damage Level	Overtopped (HW/BC≥1)		
	Pipe Damage	Pavement Damage	Shoulder Damage
0	23%	38%	3%
1	14%	58%	34%
2	61%	6%	23%
3	3%	0%	40%

A second analysis evaluated the relationship between damage levels and drainage area and the results are summarized in Table 20. To interpret the results of this analysis, it should be recognized that design methods change depending on the drainage area. For drainage areas of 0 to 0.1 square miles, the Rational Method is used. For drainage areas between 0.1 and 1 square mile, USGS Urban and Small Rural (2014) is used, while all other drainage areas (>1 square mile) use USGS Rural (2009). For this analysis, the initial expectations were that there would be a potential for higher damage levels with an increase in drainage area. Slightly higher damage levels are observed with respect to shoulder damage where at damage level of three (3), 22% of the 73 culvert cases have a drainage area greater than 1 square mile and 15% have drainage areas between 0.1 to 1 square mile. On the other hand, pavement damage and pipe damage have the highest proportions at damage levels 1 and 2 respectively. These results suggest that the relationship between higher levels of damage and increasing drainage areas is a weak indicator of potential damage during a storm for shoulder, pavement, and pipe damage. Details of these analysis are expressed in scatter plots in Appendix D and illustrated in Figure D.14.

Table 20. Proportion of culverts categorized by damage level in relation to drainage area.

Damage Level	0 Sq.mi < DA < 0.1 Sq.mi	0.1 Sq.mi < DA < 1 Sq.mi	DA > 1 Sq.mi
Shoulder Damage			
0	0%	1%	1%
1	5%	14%	14%
2	4%	8%	14%
3	1%	15%	22%
Pavement Damage			
0	7%	15%	22%
1	3%	21%	21%
2	0%	3%	1%
3	1%	0%	7%
Pipe Damage			
0	1%	11%	16%
1	3%	3%	7%
2	5%	22%	18%
3	1%	1%	10%

The correlation between Damage level and Bed-to-Crown (BC) and backfill depth were also analyzed. In the case of BC, shorter BC depths had no significant impact on the damage level of culverts and hence the relationship between Damage level and shorter BC values does not aid in the resilience assessment of culverts during a storm. Likewise, with respect to backfill, all 73 cases were found to have adequate backfill material and thus no discernable correlation could be detected. The detailed results of these correlations are shown in Figure D.15 and Figure D.16 in Appendix D.

3.3.4. Design Storm Uncertainty Results

Results derived from the analysis of AEP equivalents to the 25-yr storm, are depicted in the example in Figure 32 and in more detail in Appendix D, Figure D.1. The line of best fit for the 25-yr storm (indicated by yellow curved and black vertical lines) corresponds to the 97.5th percentile of a 5-yr storm, 87.5th percentile of an 8-yr storm, 80th percentile of a 10-yr storm, 75th percentile of an 11-yr storm, 30th percentile of a 50-yr storm, 25th percentile of a 60-yr storm, 15th percentile of a 140-yr storm, 12.5th percentile of a 180-yr storm, 4th percentile of a 700-yr storm, and the 2.5th percentile of a 1400-yr storm. This analysis was repeated for multiple drainage areas and, overall, only a minor variation was observed with respect to the AEP equivalents. For example, the 87.5th percentile of the 25-yr storm aligned with the 7.5-yr storm for 1 and 5 square miles and the 8-yr storm for 2.64 and 10 square miles. Similar slight discrepancies are noted in the 25th, 12.5th, and 4th percentiles at different drainage areas. A summary of the results is presented in Figure 32 and in Table 21.

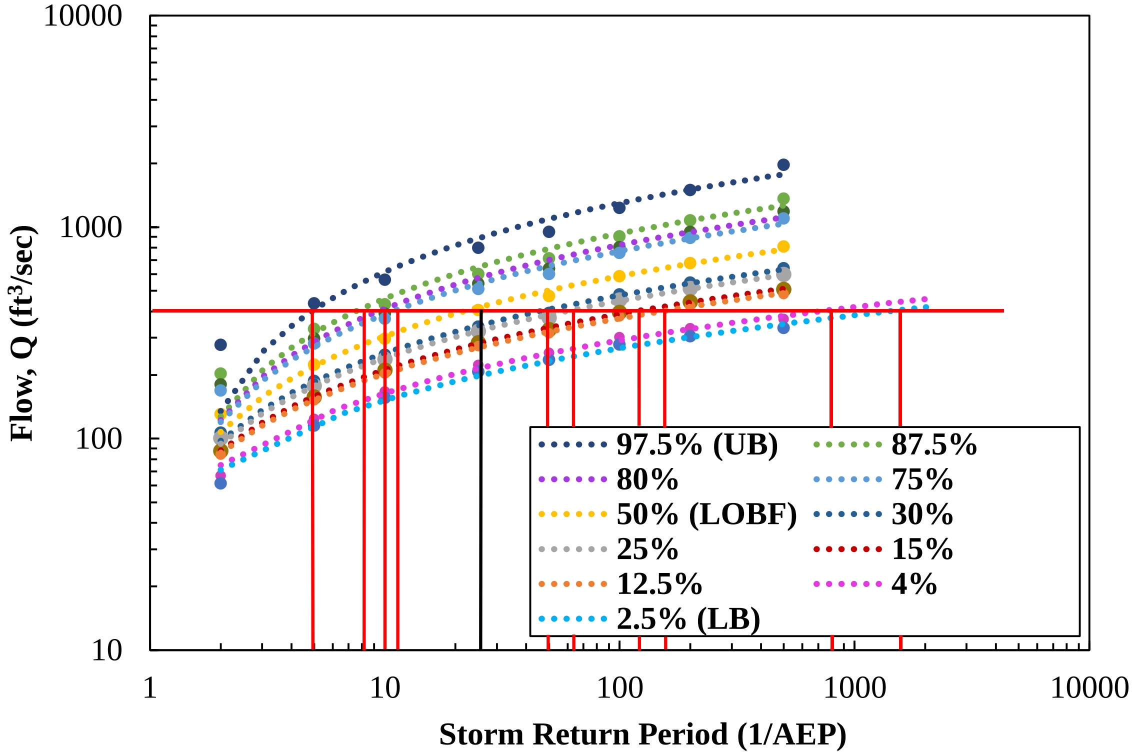

Figure 32. Plot showing relationship between flow values at their respective percentiles and storm return periods (1/AEP) when drainage area is 2 square miles.

Table 21. Storm return period equivalent to the Line of Best Fit of the 25-yr storm.

Storm Return Period	Equivalent Percentile to the LOBF of the 25-yr storm
5-yr	97.5%
8-yr	87.5%
10-yr	80.0%
11-yr	75.0%
50-yr	30.0%
60-yr	25.0%
140-yr	15.0%
180-yr	12.5%
700-yr	4.0%
1400-yr	2.5%

The research team further conducted a probabilistic analysis of flow values at different percentiles. AEP equivalents were developed from these plots by correlating the percentiles of the 25-year storm to various return periods.

summarizes these equivalencies. As an example of how to interpret the table, consider the first row, which provides equivalencies for the 97.5th percentile of the 4% AEP. At this percentile, the 4% AEP corresponds to the 47th, 65th, 80th, and 92nd percentiles of the 500-year, 200-year, 100-year, and 50-year storms, respectively. In this table, the row (in red) corresponding to the 50 percentile of the 4% AEP can be compared to the storm return period equivalent to the Line of Best Fit of the 25-yr storm as shown in Table 21. Results from the design storm frequency analysis show that a comprehensive assessment of how culverts perform under various storms events allows for informed design and mitigation strategies enhancing the resilience of culvert structures.

Table 22. Annual Exceedance Probability and their equivalent percentiles to the 4% AEP.

4% AEP Percentiles	Annual Exceedance Probability and Equivalent percentiles						
	0.2%AEP (500-yr)	0.5%AEP (200-yr)	1%AEP (100-yr)	2%AEP (50-yr)	10%AEP (10-yr)	20%AEP (5-yr)	50%AEP (2-yr)
97.5	47	65	80	92	-	-	-
87.5	25	39	53	75	-	-	-
75	15	25	35	57	94	-	-
50	7.5	11	16	33	84	95	-
25	-	4	6	12.5	58	86	-
12.5	-	-	2.5	7.5	40	75	97.5
2.5	-	-	-	-	12.5	40	87.5

Subsequent examination of the relationship between overtopping flow values derived from peak design discharge and corresponding percentiles yielded a range of results. The analysis primarily focused on eight culvert cases that experienced damage during Hurricanes Matthew and Florence. Some cases (95-9-MF-M, 95-8-MF-M, and 42-3-M) showed instances where low probability for overtopping did not align with actual culvert performance during the storms, challenging conventional overtopping criteria. In some instances, the inclusion of headwalls and upsizing of culverts (as seen in Figure D.12 through Figure D.12) improved culvert performance during the design event. However, observed culvert damage documented in the hydro report contradicted hydraulic analysis results and showed the complexities of storm-induced damage.

Case 42-3-M illustrated how down-sizing the culvert size increased the probability of overtopping after Hurricane Matthew, suggesting that altering culvert dimensions can influence culvert response during a storm. Cases 53-5-M and 25-2-F demonstrated a 47% probability of overtopping during Hurricane Matthew; however, subsequent upsizing reduced overtopping probabilities to 30% and 21%, respectively. Cases 95-2-MF and 42-1-MF initially had high overtopping probabilities (65% and 61%, respectively). Upsizing these culverts lowered overtopping probabilities, indicating the impact of design modifications. Case 8-4-MF presented a notably high overtopping probability during Hurricane Florence, underscoring the vulnerability of culverts to peak design discharge at specific percentile thresholds.

These findings while acknowledging that they are not universally applicable to all culvert case studies, reflect the intricate interplay between hydraulic analyses, culvert design modifications, and actual performance during storm events, suggesting the need for more nuanced evaluation of culvert resilience with respect to Design Storm Uncertainty. A detailed graphical representation of these cases are shown in Figure D.13(a), (b), (c), (d), (e), (f), (g), and (h) in Appendix D.

3.4. Division Practices and Lessons Learned

As mentioned previously, the research team conducted interviews with personnel from Divisions 2, 4, and 6 to gain comprehensive understanding of their practices and decisions. These interviews yielded valuable insights that significantly contribute to research study. The detailed findings from these meetings are provided in Appendix E. Based on the totality of the interviews with each division the following findings are noted:

- The common practice for backfilling is different between counties: in Lenoir and Bladen County, No. 57 Stone layer is placed as bedding until the top of the pipe; however, in Wayne County, the No. 57 Stone layer is placed as bedding until half of the pipe. Differences in practices might result in different performances and explain relatively higher number of damaged sites in Wayne County.
- A certain amount of time (depending on the time of year and climate of the location) is needed after repair for the vegetation to become reestablished and the slope stabilized. If sufficient time between the repair and an event is not given, then the pipe might not perform as well as expected.
- Practices to reduce erodibility include but are not limited to proper compaction, matting, armoring with rip rap on both sides, properly placed side ditches, and use of No. 57 Stone for joint issues.
- All three divisions unanimously suggested that an erosion or damage is not often detected until it becomes a problem; therefore, preemptive and more frequent monitoring is needed.
- Washout of top layer on shoulder is not concerning if the pipe is not damaged and headwall anchors are not compromised.
- In some cases, old construction guidelines for pipe and headwall, e.g., not embedding the concrete headwall, might have aggravated the damage that occurred during Hurricane Matthew.
- Some practices to reduce erodibility are not possible because of limitations due to environmental considerations or placement of utilities.

Other contributing elements include utilities cut through pipes, upstream features, e.g., dam, acidic soils, and toe wall depth.

3.5. Vulnerability Analysis

3.5.1. Level of Damage

A representative radar plot showing the vulnerability assessment for each type of structure is shown in Figure 33. Various patterns emerged through this graphical evaluation approach. For example, Figure 33(a) has a triangle shape, which was typical of RCP pipes and shows that at locations with RCP pipes the pipe itself and the shoulder are the most vulnerable components. Figure 33(b) (a CMP pipe) has a diamond shape, which shows that in CMP pipes, the pipe and shoulder are damaged, but it is also common to see pavement damage. Figure 33(c) has a symmetric diamond shape, which shows that CMPA pipes, when damaged, experienced damage to all elements and also required substantial mobilization to repair. Figure 33(d) has a reversed-triangle shape, which shows that in RCBC pipes the pipe can remain largely undamaged, but the shoulder and pavement can get heavily damaged. It should be noted that the observations are limited to the cases of this study.

Radar plots were also evaluated in each site for Hurricanes Matthew and Florence and current conditions. Figure 34, Figure 35, and Figure 36 are presented here as examples. Figure 34(a) and (b) show the radar plots for Site 53-4-MF after Matthew and Florence, respectively. In this case, the pipe was the same in both events and there was no headwall in the structure. Figure 34(b) in comparison to Figure 34(a) has higher damage in all components. It could be due to more severe stressors or due to the accumulation of damage or vulnerabilities through the two events. Figure 35(a) and (b) show the radar plots for Site 23-5-M after Matthew and in the current condition, respectively. In this case, the pipe was kept the same after Matthew and there was no headwall in the structure. Figure 35(a) shows a minimal damage and Figure 35(b) indicates no damage. This result suggests that the repair that was done after Matthew was successful, and it is still in good shape. Figure 36(a) and (b) show the radar plots for Site 25-2-M after Matthew and in the current condition, respectively. In this case the pipe was upsized, and headwall was added to the structure. Figure 36(a) shows a significant damage and Figure 36(b) indicates no damage. It shows that damage after Matthew required the pipe change and the improvement and addition of headwall was successful so that it was not damaged after Florence, and it is still in good shape.

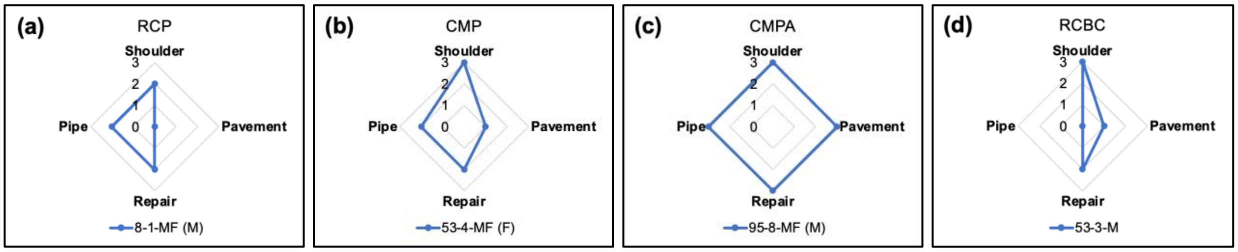

Figure 33. Representative radar plots of damage level components for: (a) RCP, (b) CMP, (c) CMPA, and (d) RCBC.

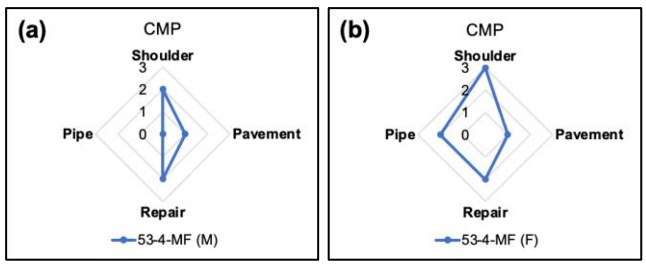

Figure 34. Radar plot for Site 53-4-MF after: (a) Matthew and (b) Florence.

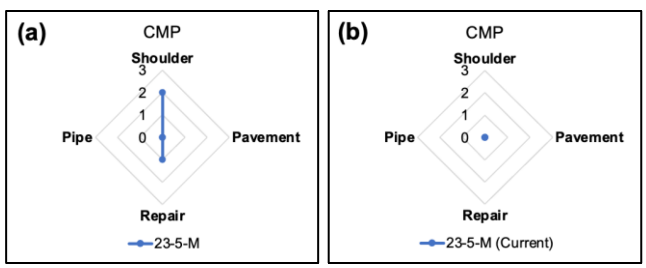

Figure 35. Radar plot for Site 23-5-M: (a) after Matthew and (b) in current condition.

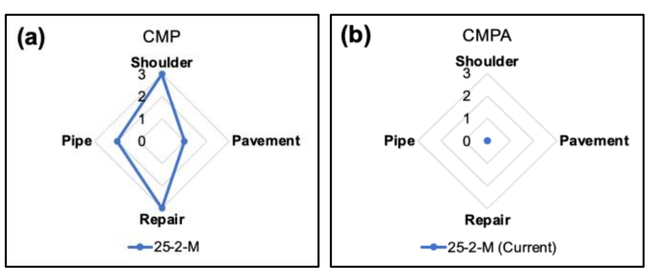

Figure 36. Radar plot for Site 25-2-M: (a) after Matthew and (b) in current condition.

Further investigation of the damage level radar plots indicated that in most of the CMP cases, the same type of structure existed in both events. The pipe may have been the same size, or it could have been upsized. This situation was not the case for RCP pipes. In three cases the RCP was considerably damaged after Matthew and pipe was upsized to a CMP pipe, damage after Florence was relatively lower, especially on pipe and pavement component. For example, radar plots for Site 95-6-MF are presented in Figure 41(a) and (b).

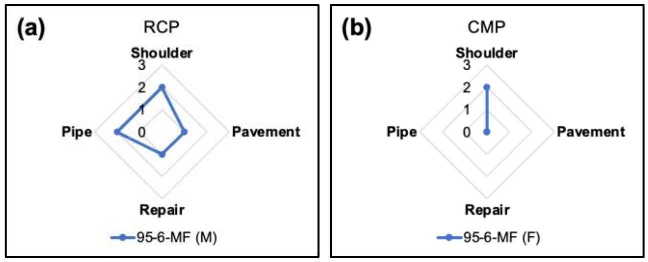

Figure 37. Radar plot for Site 95-6-MF after: (a) Matthew and (b) Florence.

3.5.2. Vulnerability Score

The damage levels and vulnerability scores were plotted with respect to one another to examine the correlation between these values. This plot is presented in Figure 42. Though there is considerable scatter, the trend, as expected, is that as the level of damage increases the vulnerability score increases. This relationship appears to be consistent (though subject to considerable scatter) up to a damage level of 10, beyond which the trend deviates. However, damage levels above 10 indicate complete or nearly complete washouts and initial investigations showed that all these cases are from Matthew event. Thus, there is greater uncertainty associated with these cases. Also, most of these cases have CMPA structure type and their washout might be explained by elements other than the ones considered in the vulnerability scores, such as shallow toe wall information on which is not available.

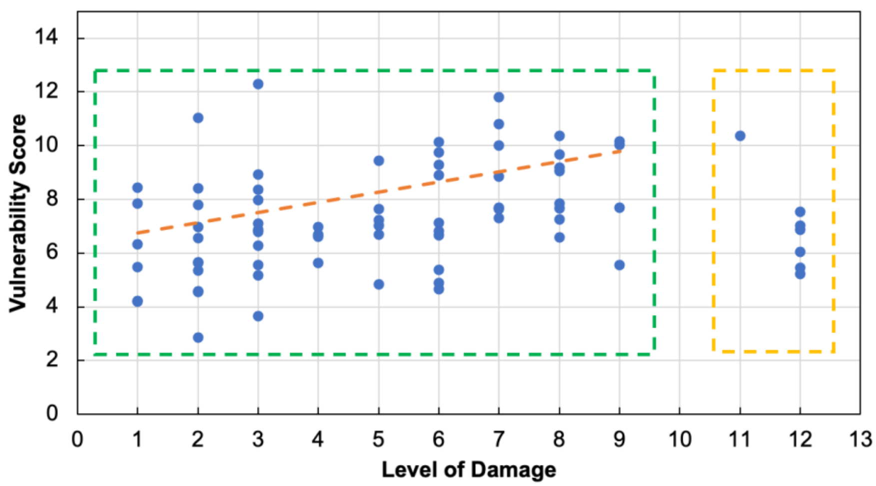

Figure 38. Correlation between level of damage and vulnerability scores.

In the initial investigation of radar plots for CMP pipes, the plots were categorized based on the visual pattern. Then within each shape category, radar plots of scores and damage levels were compared to explore correlations. Figure 39 and Figure 40 are presented here as examples. Figure 39 shows the radar plots for Site 53-4-MF as one of the categories. The radar plots of damage levels were previously discussed in this report. As showed in Figure 39(c) and (d), the radar plots of score components for Matthew and Florence are the same except rainfall intensity and surrounding features (in this case flood plain) is worse in Florence, so the higher damage level in Florence can partly be explained by this factor.

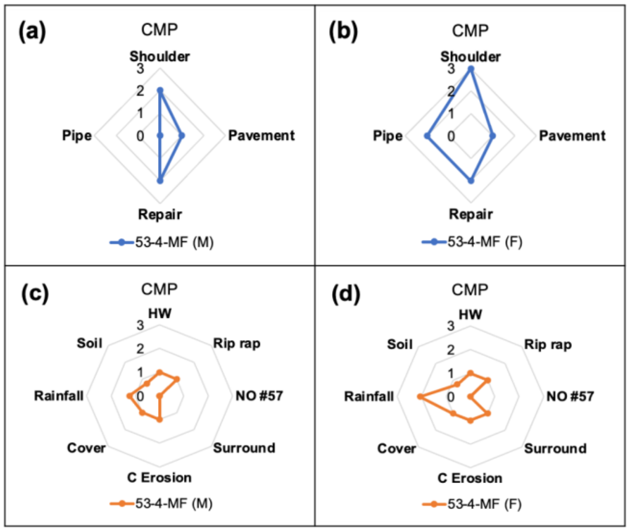

Figure 39. Radar plots of Site 53-4-MF for: (a) damage levels after Matthew, (b) damage levels after Florence, (c) vulnerability scores after Matthew, and (d) vulnerability scores after Florence.

Figure 40 shows the radar plots for Sites 95-4-MF, 95-6-MF, 95-5-MF, and 95-2-M and shows a second category of vulnerability behaviors. Comparing the plots of Sites 95-4-MF and 95-6-MF, Figure 40(a), (b), (e), and (f), with Sites 95-5-MF and 95-2-M, Figure 40(c), (d), (g), and (h), it can be seen that the level of damage is significantly different between these two groups. These differences are present even though their vulnerability score plots are almost similar except for the headwall component. The former group, Figure 40(a), (b), (e), and (f), have headwall, but the latter group does not have headwall, Figure 40(c), (d), (g), and (h), so the higher level of damage in the latter group can be partly explained. Also, it is noted that the color of the graphs in Figure 40(f) and (g) (green) indicates that in these cases the HW/D with respect to the 25-year design event is less than 1.2. The orange color in Figure 40(e) and (h) indicates that the HW/D is higher than 1.2. However, this parameter does not provide further insight on the correlation patterns, therefore, further investigation is needed in exploring the correlations between vulnerability score and damage level component.

Figure 41(a – h) shows the radar plots for Site 95-1-MF, 95-5-MF, and 23-1-MF to show a third category of the vulnerability radar plots. In this particular case, a combination of the following factors contributed to the absence of damage to the pavement and pipe, requiring only minor repairs:

- Presence of rip rap and No. 57 Stone, along with sufficient cover (due to lack of data during the events, the cover is qualitatively evaluated based on damage photos): These measures provided adequate protection against erosion and mitigated the potential for damage.
- Absence of surrounding features (i.e., swamp, beaver dam, and wide floodplain) and channel erosion: The lack of nearby vulnerability and strong flow causing the erosion of channel minimized the risk of impact on the pavement and pipe.

- Low intensity of rainfall and less erodible soil type: The rainfall during the event was of low intensity, coupled with the presence of less vulnerable soil, further reducing the likelihood of damage.

In this context, the influence of the lack of headwall, Figure 41(g) and (h), and HW/D ratio (low in case on Figure 41(f) and high in cases of Figure 41(e), (g), and (h)) was overshadowed by the combined effect of these other factors.

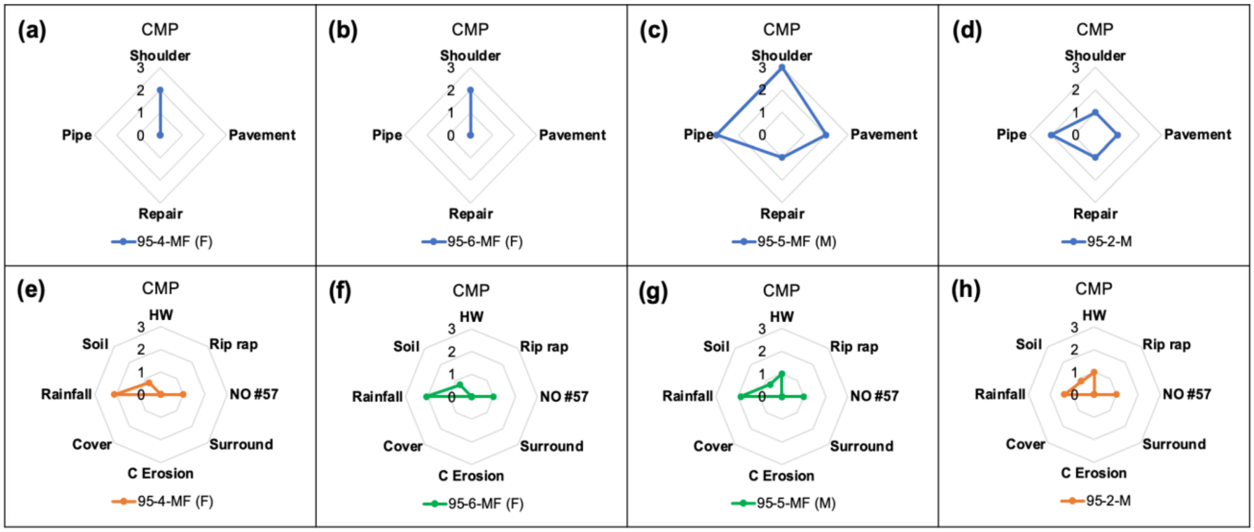

Figure 40. Radar plots of damage level components for: (a) Site 95-4-MF after Florence, (b) Site 95-6-MF after Florence, (c) Site 95-5-MF after Matthew, (d) Site 95-2-M after Matthew, and radar plots of vulnerability scores for (e) Site 95-4-MF after Florence, (f) Site 95-6-MF after Florence, (g) Site 95-5-MF after Matthew, and (h) Site 95-2-M after Matthew.

Figure 42(a – f) shows the radar plots for Site 25-1-MF and 95-9-F as another category of vulnerability radar plots. In this case, a combination of the following factors contributed to the absence of damage to the pavement and the need for only minor repairs:

- Use of No. 57 Stone and headwall: The utilization of No. 57 Stone and a headwall provided sufficient protection and stability for the infrastructure, preventing significant damage.
- Absence of channel erosion: When no channel erosion is observed it implies that the storm flow was not strong. Due to the lack of strong flow, the pavement and infrastructure remained intact, avoiding any significant damage.
- Less erodible soil type and low rainfall intensity: The soil's less erodible nature and the low intensity of rainfall reduced the risk of damage to the pavement.

However, it is worth noting that the presence of surrounding features and the absence of rip rap had adverse effects including requiring minor repairs, Figure 42(a), or damaging the pipe, Figure 42(b) and (c). Comparatively, Site 95-9-F, Figure 42(c), exhibited more significant damage to the pipe and shoulder due to insufficient cover and a high HW/D ratio when compared to Site 25-1-MF after both Matthew and Florence, Figure 42(a) and (b).

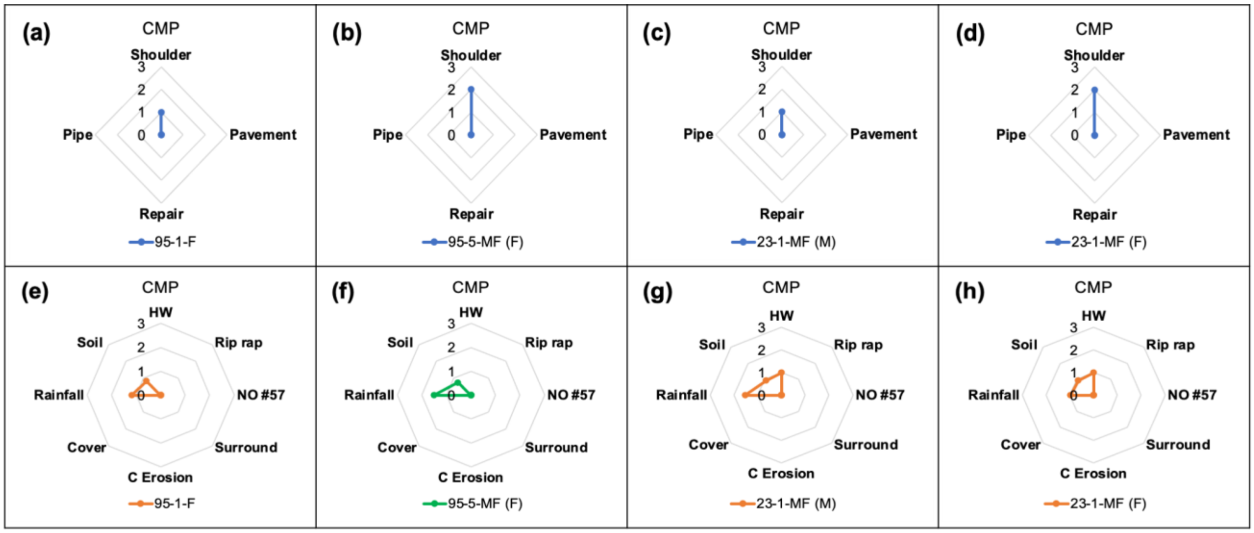

Figure 41. Radar plots of damage level components for: (a) Site 95-1-F after Florence, (b) Site 95-5-MF after Florence, (c) Site 23-1-MF after Matthew, (d) Site 23-1-MF after Florence, and radar plots of vulnerability scores for (e) Site 95-1-F after Florence, (f) Site 95-5-MF after Florence, (g) 23-1-MF after Matthew, and (h) Site 23-1-MF after Florence.

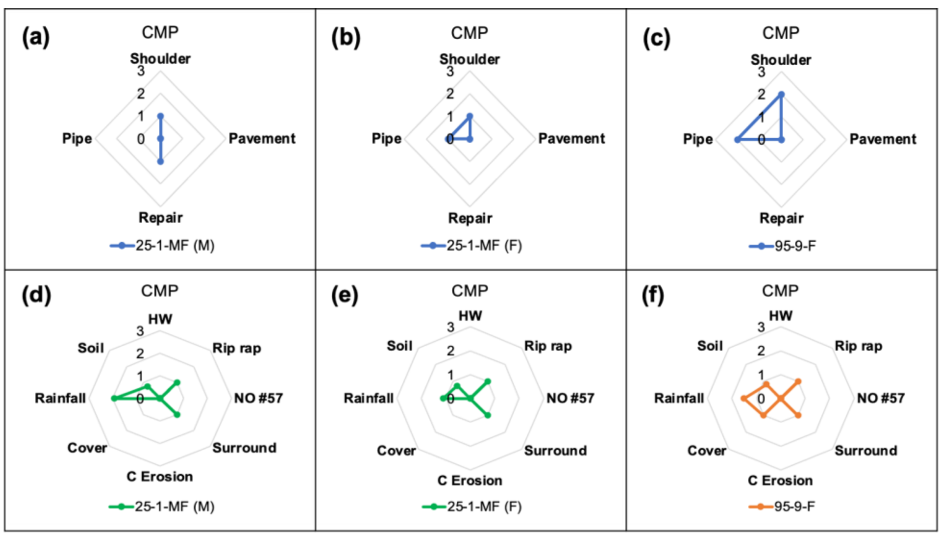

Figure 42. Radar plots of damage level components for: (a) Site 25-1-MF after Matthew, (b) Site 25-1-MF after Florence, (c) Site 95-9-F after Florence, and radar plots of vulnerability scores for (d) Site 25-1-MF after Matthew, (e) Site 25-1-MF after Florence, and (f) Site 95-9-F after Florence.

Figure 43 (a – j) shows the radar plots for Site 95-8-F, 53-1-MF, 53-5-M, and 8-2-MF as another category of vulnerability radar plots. In this case, a combination of factors contributed to damage to all components resulting in a diamond shape in radar plot and particularly extreme damage to the shoulder. The factors involved in this case are as follows:

- Absence of headwall, rip rap, and cover: The lack protective measures such as headwall, rip rap, and cover left the site vulnerable, leading to substantial damage to shoulder and pipe. It also required moderate to heavy repair.

- Presence of surrounding features (i.e., swamp, beaver dam, and wide floodplain) and high intensity rainfall: The presence of surrounding features and occurrence of high-intensity rainfall heightened the risk of damage to all components at these sites.
- Use of No. 57 Stone: The utilization of No. 57 Stone provided protection and stability for the pavement.
- Absence of channel erosion: The absence of channel erosion indicates the absence of a strong flow, which played a role in preventing damage.

It should be noted that low HW/D value, which suggests sufficient design helped prevent damage to pipe and pavement, Figure 43(f). However, the combination of factors mentioned above still had a significant impact on shoulder, resulting in considerable damage, Figure 43(f).

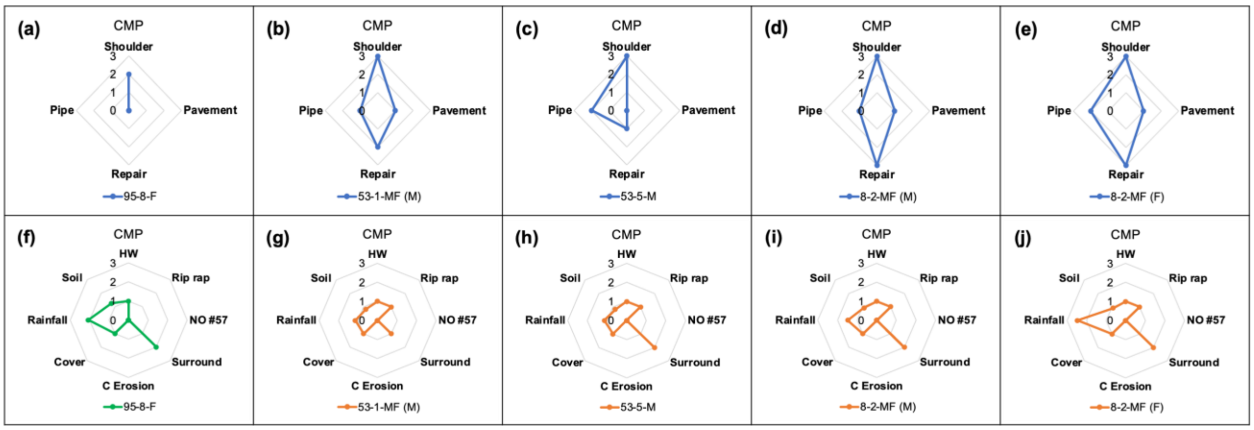

Figure 43. Radar plots of damage level components for: (a) Site 95-8-F after Florence, (b) Site 53-1-MF after Matthew, (c) Site 53-5-M after Matthew, (d) Site 8-2-MF after Matthew, (e) Site 8-2-MF after Florence, and radar plots of vulnerability scores for (f) Site 95-8-F after Florence, (g) Site 53-1-MF after Matthew, (h) Site 53-5-M after Matthew, (i) Site 8-2-MF after Matthew, and (j) Site 8-2-MF after Florence.

Similarly, the radar plots for RCP pipes were evaluated and the plots were categorized based on the visual pattern. Then within each shape category radar plots of scores and damage levels were compared to explore correlations. Figure 44 (a – d) shows the radar plots for Site 95-4-F and 95-6-F as one category of vulnerability radar plots. In this case, a combination of factors contributed to damage to pipe and shoulder resulting in a triangle shape in radar plot. The factors involved in this case are as follows:

- Absence of headwall and rip rap: The lack protective measures such as headwall and rip rap left the site vulnerable, leading to substantial damage to shoulder and pipe.
- Absence of surrounding features (i.e., swamp, beaver dam, and wide floodplain) and channel erosion: The lack of nearby vulnerability and strong flow causing the erosion of channel minimized the risk of impact on the pavement.
- Use of No. 57 Stone and sufficient cover: The utilization of No. 57 Stone and presence of sufficient cover provided protection and stability for the pavement.
- Moderate intensity of rainfall and erodible soil type: The rainfall during the event was of moderate intensity, coupled with the presence of vulnerable soil, further increasing the likelihood of damage to pipe and shoulder. More specifically, in Site 95-6-F, Figure 44(d), the soil is more vulnerable than that of Site 95-4-F, Figure 44(c), the damage to the shoulder

is higher in Site 95-6-F, Figure 44(b), even though the rainfall is less intense in Site 95-4-F.

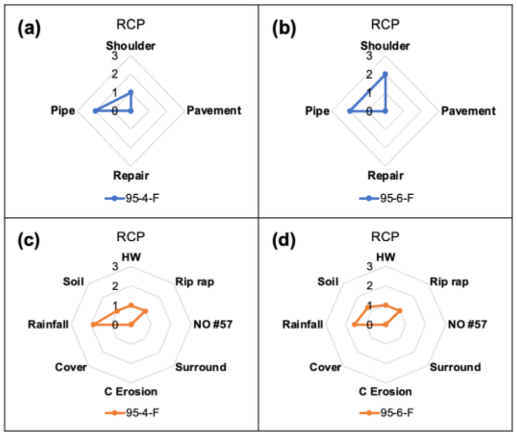

Figure 44. Radar plots of damage level components for: (a) Site 95-4-F after Florence, (b) Site 95-6-F after Florence, and radar plots of vulnerability scores for (c) Site 95-4-F after Florence, and (d) Site 95-6-F after Florence.

Similarly, the radar plots for CMPA pipes were evaluated and the plots were categorized based on the visual pattern. Then within each shape category radar plots of scores and damage levels were compared to explore correlations. Figure 45 (a – f) shows the radar plots for Site 95-8-M, 95-2-MF, and 95-6-MF as one category of vulnerability radar plots. In this case, a combination of factors contributed to damage to all components.

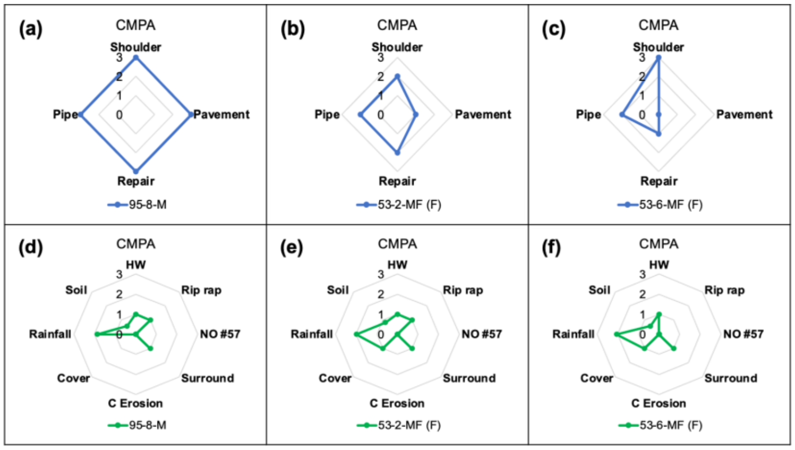

Figure 45. Radar plots of damage level components for: (a) Site 95-8-M after Matthew, (b) Site 53-2-MF after Florence, (c) Site 53-6-MF after Florence, and radar plots of vulnerability scores for (d) Site 95-8-M after Matthew, (e) Site 53-2-MF after Florence, and (f) Site 53-6-MF after Florence.

The factors involved in this case are as follows:

1- Absence of headwall: The lack protective measures such as headwall left the site vulnerable, leading to substantial damage to shoulder, pipe, and pavement.
2- Presence of surrounding features (i.e., swamp, beaver dam, and wide floodplain): The presence of surrounding features heightened the risk of damage to all components at these sites.

3- Moderate intensity of rainfall and erodible soil type: The rainfall during the event was of moderate intensity, coupled with the presence of vulnerable soil, further increasing the likelihood of damage to pipe and shoulder.

It should be noted that presence of sufficient cover in Site 95-2-MF and 95-6-MF provided protection for the components and the level of damage for these sites is less than that of Site 95-8-M. Comparing Site 95-2-MF and 95-6-MF, it can be seen that use of rip rap provided additional protection and pavement was not damaged in Site 95-6-MF despite of Site 95-2-MF.

3.5.3. Soil Erosion Vulnerability Scores

Results as shown in Table 23 derived from the visual manual classification of soils showed the sites visited had a range of soil types from well graded sands (SW), which are considered to have high erodibility, to silts (ML – Low plasticity silts), which are considered to have medium or low erodibility.

Table 23. Soil types collected at respective case sites.

Case ID	Street Name	Soil Type	Soil Group Symbol
95-2-MF	Mark Herring Rd	Well graded Sand with Silt	SW-SM
95-4-MF	NC 55	Well graded Sand with Silt	SW-SM
95-9-MF	North Center Street	Silty Sand	SM
95-8-MF	Raynor Mill Rd	Sand with Silt	ML w/S
95-3-M	Wayne Memorial Drive	Silty Sand	SM
95-3-F	Pinkney Road	Poorly Graded Sand with Silt	SP-SM
95-4-F	US 117	Poorly Graded Sand	SP
53-6-MF	NC 903	Silt with Sand	ML w/S
53-3-MF	N Croom Bland Rd	Silty Sand	SM
53-6-M	Falling Creek Rd	Clayey Sand	SC
53-2-F	W. Pleasant Hill Rd	Poorly Graded Sand with Silt	SP-SM
42-1-F	Wine Rd	Silty Sand	SM
42-3-M	Brick Mill Rd	Well graded Sand with Silt	SW-SM
25-2-F	LA Durham Rd	Poorly Graded Sand	SP
8-4-MF	Brown Creek Church Rd	Silty Sand	SM
8-2-F	Everette Byrd Road	Well Graded Sand	SW
23-1-MF	Union Valley Rd	Poorly Graded Sand with Silt	SP-SM
23-5-MF	Jordan Rd	Silty Sand	ML
23-4-F	Peacock Rd	Sandy Silt	ML

The soil erosion vulnerability scores ranging from 0 to 1 were then determined using Equation (1) shown below and the erosion categories obtained from Briaud's erosion charts for geomaterials (Briaud et al., 2017). It should be noted, as shown in Figure 11 and Figure 12, that Briaud's erosion charts for geomaterials have erosion categories one (1) to six (6) covering a range of soils. However, our cases, as per the soil types analyzed, only cover a narrow range of soils with the highest being category 3. Table 24 shows soil erosion vulnerability scores information obtained from the visual manual soil classification. The results obtained were then used as supplementary information to understand the influential parameters that contribute to soil vulnerability scores.

$$SES = \frac{(-1 \times EC) + 4}{3} \tag{1}$$

where;

SES = Soil Erosion Vulnerability Scores, and
EC = Erosion Categories.

Table 24. Relationship between soil erodibility and soil type.

Soil Group	Erosion Categories (EC)	Soil Erosion Vulnerability Scores (SES)
SP	1	1
SP-SM	1.25	0.92
SM	1.5	0.83
SW-SM	1.75	0.75
SW	1.75	0.75
SC	2	0.67
ML	2.25	0.58
MH	3	0.33

As shown in Table 24, soil erosion vulnerability scores are highest in sands and lowest in silts. Therefore, soils with higher fines content such as silts (MH, ML) would help reduce vulnerability of culverts with regards to erosion as compared to soils with less fines content.

3.6. Discussion on Contributing Factors

After conducting analyses on the correlation between the level of damage and vulnerability scores in Section 3.5, Figure 39 through Figure 45, specific features have emerged as vulnerabilities that require consideration in the management process. These vulnerabilities encompass the presence of surrounding features and erodible soil. Notably, features such as swamps, beaver dams, wide flood plains, and strong flow (indicated by erosion in the bottom of the channel) should be acknowledged as potential vulnerabilities. The combined presence of these features can elevate the susceptibility of the structure to damage. However, it is important to highlight that the existence of only one of these features does not necessarily indicate an increased vulnerability.

Section 3.5 highlights that the analyses conducted on the correlation between the level of damage and vulnerability scores have identified specific design elements that play a crucial role in offering protection for the structure. These elements encompass the headwall, rip rap, No. 57 Stone, sufficient cover, and less erodible soil. It is vital to recognize that the combination of these elements collectively contributes to the overall protection and resilience of the structure. It should be noted that the presence of only one of these elements may not be adequate to provide the desired level of protection.

Another parameter to be considered is the presence of side ditches. Side ditches help guide floodwater through channels and the total volume of water that they feed into the culvert crossing is likely accounted for during the design process. However, an aspect that may be overlooked during the design is the potential for cascading effects to the channel from damage in the ditches or turbulent flows that are induced when the ditch empties into the channel. For instance, the photos from Google Earth in Site 95-1-F, located on Nahunta Road show a case where side ditch and main channel damage occurs at their intersection. As highlighted with yellow arrows in Figure 46, the floodwater can enter the stream from side ditches. Drawing from the interviews conducted with division engineers, as detailed in Section 3.4.3, successful practices were implemented in response to roadside slope washouts observed in the aftermath of Hurricane Matthew. The implementation of measures such as flattening side ditch longitudinal slopes and/or back/foreslopes (where feasible) and installing gutters to regulate floodwater proved to be effective. Division engineers noted in cases where such mitigation steps were taken that no issues were reported following the occurrence of Hurricane Florence.

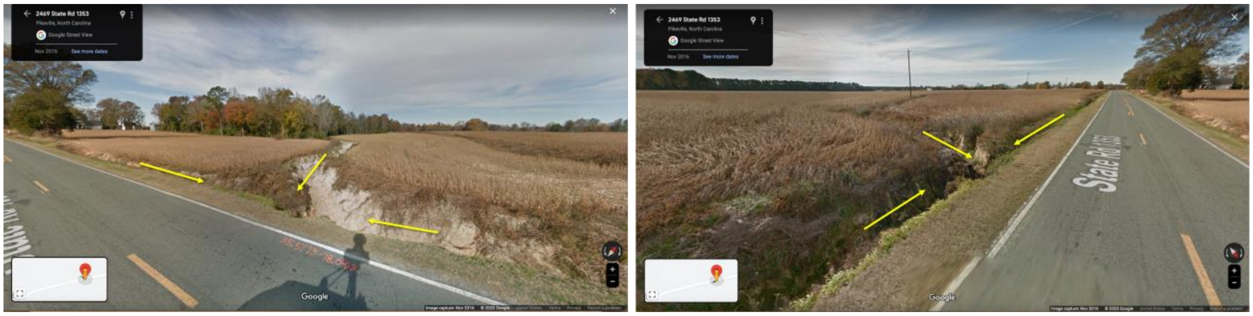

Figure 46. Photos from Google Earth in Site 95-1-F on the inlet side (left) and on the outlet side (right).

As detailed in Section 3.2.6, evidence of ongoing erosion in channels or feeder ditches was observed during site visits. When discussing this matter with division engineers, it was clarified that these erosion issues are frequently not detected until they reach a problematic stage and begin to impact the road or traffic. Early detection of such erosion could serve as a warning that a site is vulnerable to or subject to turbulent flows and/or a weakening or failing drainage system.

Based on the analysis of available data and surveys, it was observed that some sites were not previously flagged as damaged in events prior to Hurricane Florence. However, when evaluating satellite photos from Google Earth for these sites, indications such as patches or overlays on the pavement were evident in different years. For instance, the photos from Google Earth in Site 95-2-F, located on North Washington Street were investigated. As highlighted in Figure 47, during different times overlay on top of the location of pipe is detectable. This site was only flagged after Hurricane Florence, but the overlay and sign of recurrence issue is evident prior to Hurricane Florence and in current condition.

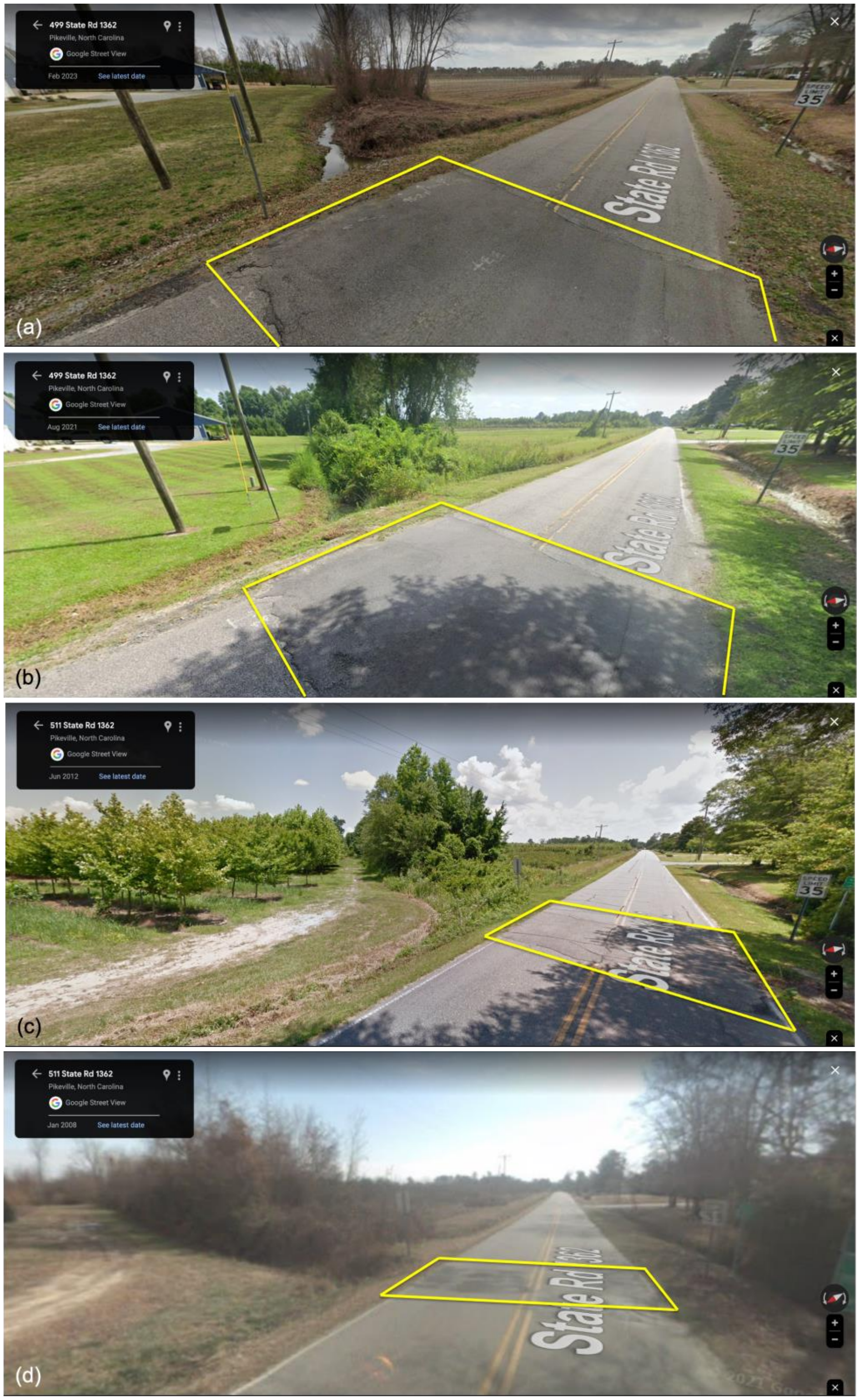

Figure 47. Photos from Google Earth in Site 95-2-F on (a) February 2023, (b) August 2021, (c) June 2012, and (d) January 2008.

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4. Conclusions and Recommendations

4.1. Conclusions

Based on the interviews, site visits, and data analysis conducted in this research, the following conclusions are made.

- The actions taken by the NCDOT hydraulics unit and maintenance operations group following the recent hurricanes have been effective at increasing the robustness and reparability (i.e., resilience) of roadways with pipe crossings. These actions include design and repair decisions as well as decisions to create a database to catalog damage assessments.
- When pipes and culverts were redesigned following either Hurricane Matthew, Hurricane Florence, or both, they were almost always upsized. Between approximately 67% and 75% of the damaged sites evaluated were undersized by current design standards prior to them being damaged.
- Beyond upsizing, some design features were found to provide protection for the structure and reduce/mitigate the damage from extreme flows. These elements include:
    - Headwall: The presence of a headwall helps provide structural stability and protection for the pipe during flood events.
    - Rip rap: The use of rip rap, especially along the banks or embankments, helps prevent erosion and provides additional protection to the structure.
    - No. 57 Stone: The utilization of No. 57 Stone for bedding and backfill offers protection and stability for the pipe as well as the pavement.
    - Sufficient cover: Having sufficient cover over the structure helps shield it from external forces and potential damage.
    - Soil erodibility: Constructing the structure on and/or with less erodible soil reduces the risk of soil displacement and erosion, enhancing the overall resilience.
    - Side ditch mitigation: In some cases, adjustments to the slopes (longitudinal, backslope, and/or foreslope) can mitigate damage not only to the side ditches, but also to the channel and pipe.
- The following features are considered warning signs of potential vulnerabilities: presence of erodible soil and/or surrounding features such as swamps, beaver dams, wide flood plains, and strong flow (indicated by erosion in the bottom of the channel). The presence of only one of these features does not necessarily indicate heightened vulnerability.

4.2. Recommendations

On the basis of the study reported here, the research team makes the following recommendations.

- <u>Design and Repair Strategies</u>: The NCDOT continue to utilize the same design and repair strategies enacted after Hurricanes Matthew and Florence.
- <u>Survey 123 Database Enhancements</u>: The NCDOT should consider enhancing the Survey 123 database to include more details on the decision-making process from the design stage through the construction stage (e.g., storing hydraulic designs and as-built data into Survey 123) including the data of final pipe placement. When a hydraulic design is provided for a culvert, often multiple options are provided; the precise details of the design (or a modification based on available resources) that is installed on site is not recorded within the 123 Database and may not be centrally reported. Also, the design practices at the time of installation may significantly impact a pipe's vulnerability to flooding. This information

would be crucial for assessing the performance of infrastructure over time and understanding the aging effects on its resilience. It is also recommended that minor repairs or maintenance activities be recorded comprehensively in the same database where catastrophic damage and repairs are recorded. It is recognized that recording these additional data in Survey 123 is redundant for the division engineers. However, the lack of such detailed information was a hurdle for the research team in this project. Having more complete information centrally stored would greatly simplify postmortem evaluations and continual assessment and identification of potential improvement areas.

- Selected Site Monitoring: It is recommended that the NCDOT select a subsample of sites from this project and conduct continual monitoring of the overall performance and flow characteristics at the site. The research team noted substantial uncertainty with respect to the design flows estimated using current USGS equations and continued damage at some sites. By better understanding the characteristics of sites that lead to higher or lower real flows than those estimated by the design equations, the NCDOT could make more precise and accurate flow estimates and decisions on pipe sizing.

5. Implementation and Technology Transfer Plan

The Maintenance Operations and Fleet Management group and the Hydraulics Unit of the NCDOT will be the primary users of this product. The products of this research will be used to improve existing maintenance and hydraulic design specifications, which will result in cost savings. This research can be used to improve specifications through the recommendations made in the previous chapter.

For follow-up activities, the research team believes that the NCDOT could consider the following activities:

- allocating resources to evaluate the vulnerability and damage factors from this study on additional sites across eastern and western North Carolina. It is likely that additional contributory factors could be identified in western counties (i.e., channel longitudinal slope) and used to improve the recommendations from this project;
- allocating resources to enhance the Survey 123 database and data collection training. The research team noted some inconsistencies across Divisions/Counties and more consistent photo logging of pre-repair and post-repair conditions might be beneficial; and
- allocating resources to continue to monitor a subset of sites identified in this study including stream monitoring and subsequent modeling work to improve upon the flow estimation process used for design.

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APPENDIX A: DETAILED LITERATURE REVIEW

INTRODUCTION

Infrastructure resilience has become an important topic for North Carolina. Recent hurricanes and other extreme events have caused more than $450 million in damage to the States' transportation infrastructure. In addition to the cost of the infrastructure, the NCDOT spent considerable resources to redesign and repair many elements after each event. A review of the NCDOT records following Hurricane Florence suggest that more than 3,000 disruptions occurred. Some of these locations were identical to those damaged during Hurricane Matthew. However, the amount of damage was different between the two events, suggesting that DOT strategies were effective. The potential reasons for this including but not limited to:

1. when the infrastructure was initially designed and constructed (pre-Matthew) the design codes and standards were not the same as those used post-Matthew (for example installation of headwalls),
2. the infrastructure pre-Matthew was older and perhaps had accumulated damage that had weakened the infrastructure,
3. flooding intensities, though similar and well above normal expectations, may have differed, and
4. debris flow/actual capacity due to deferred maintenance may have also differed in the two events.

The current study will identify and evaluate the specific elements, design features, or repair options used in the new infrastructure that positively contributed to the improved performance during Hurricane Florence and those that did not positively contribute. Though guidance on improved and/or resilient design exists from the FHWA, AASHTO, NCHRP, and others, these issues are highly context sensitive with many contributing factors including age, maintenance levels, rainfall intensity, etc. that necessitates a North Carolina specific investigation. This research will a) evaluate the design process for roadway infrastructure that was repaired following Hurricanes Matthew and Florence, b) identify the specific elements of the new infrastructure that positively contributed to improved performance during Hurricane Florence, and c) develop recommendations on design elements that improve the resilience of NCDOT roadways.

In order to carry out this investigation, a review of some important topics is necessary. This review is divided into five primary sections. Section 1 (this section) provides an overview of the research plan and description of the literature review organization. Section 2 describes the relevant studies regarding basic process of hydraulic design. Section 3 reviews the most recent and relevant guidelines for hydraulic design on national, regional, and state level. Section 4 reviews the ongoing or recently completed research studies to understand more fully what other agencies have done to improve their designs in the face of extreme events and make them more resilient. Finally, Section 5 provides a summary of the literature review and points out the important knowledge gaps in the previous studies.

OVERVIEW OF HYDRAULIC DESIGN PRACTICE

In the following sections, the basic process of hydraulic design will be outlined at a national level. According to Federal Highway Administration (FHWA) design philosophy, the primary purpose of highway drainage facilities is to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway. Two disciplines utilized in highway drainage design that this research project will focus on are hydrology and hydraulics. The determination of the quantity and frequency of runoff is the hydrologic portion of the design process. The hydraulic design of a drainage structure is determining the appropriate capacity to divert water from the roadway, remove water from the roadway, and pass collected water under the roadway.

For a given structure that services a specific drainage area, an estimate of the amount of runoff that will occur for a storm is considered to be a major component of the hydraulic design process (Kilgore et al., 2016). A number of hydrologic methods are available in order to analyze and determine peak runoff for a given storm. From these runoff estimates, design engineers utilize the runoff in conjunction with frequency analyses to characterize the risk for a given drainage area and structure. During design, terms of annual exceedance or recurrence intervals are used to describe the probability of occurrence of a given precipitation event. Based on the probability of occurrence of an event and the peak runoff that will occur for that event, a hydraulic engineer can design the drainage structure to be able to withstand that precipitation event.

When designing for drainage facilities, a range of discharges with a range of flood frequencies are used, typically termed the "base flood" and "super flood". A base flood is defined as the flood or storm having a 1 percent chance of being equaled or exceeded in any given year, or 1% annual exceedance probability (AEP). Owing to the fact that the inverse of the AEP is a whole number indicative of occurrence in a given year (i.e., a 1% AEP is equivalent to a 1 in 100 probability), this event is often referred to as simply the 100-year flood. This terminology may give the impression that there is certainty that this event will only occur once every 100 years. However, in reality it is simply a probabilistic assessment of its likelihood. Thus, a 1% AEP event has a 39.5% probability of occurrence at least once during a given 50-year time frame, a 8.9% chance of happening at least twice over the same 50 year time frame, and a 1.4% chance of happening three times over the same time period.

In the following sections, this literature review will examine the design process utilized by national and state agencies, outline hydrologic methods utilized to inform those design decisions, and design practices of North Carolina Department of Transportation (NCDOT) and how the design process and best practices can differ from other state agencies in the region.

Resources for Estimating Probability of Annual Exceedance

There are many methods in use to estimate rainfall intensity. Peak discharge estimates from these methods are dependent on precipitation data recorded from national agencies, specifically the National Weather Service (NWS) and National Oceanic and Atmospheric Administration (NOAA). The most recent widespread analysis of precipitation data for North Carolina is presented in NOAA Atlas 14, Vol 2. This volume was released in 2006 and the last data for the estimates presented therein was gathered in 2004. Mention of an updated contract with NOAA indicated that the Southeastern US dataset will be updated again in 2023, published as Vol 13.

Rainfall intensity is the rate at which precipitation occurs. Intensity is usually stated irrespective of the duration of the rainfall, although it can be stated as total rainfall in a particular time period or duration. Frequency is expressed as the probability of a given rainfall intensity being equaled or exceeded (Kilgore et al., 2016). Rainfall data are used to derive intensity-duration-frequency curves necessary in hydrologic analysis, as mentioned in the Rational Method. Two methods for selecting rainfall data used in such frequency analyses are: (1) annual-series and (2) partial-duration series. Annual-series analysis considers only the maximum rainfall for a given year and ignores the remaining rainfalls, even though these lesser rainfalls could exceed the maximum of other years. The partial-duration series analysis considers all of the high rainfalls, regardless of the number occurring within a given year. The FHWA guidelines recommend when designing highway drainage facilities for return periods greater than 10 years, the difference between the two series is unimportant and can be ignored. However, when the return period or design frequency is less than 10 years, the partial-duration series is believed to be more appropriate.

Uncertainty and Extreme Events Consideration in Current Guidelines

Design events carry statistical uncertainty from the estimation process due to the sample size and statistical techniques adopted. The uncertainty can be translated into confidence intervals using the mean intensity or the mean return period. For example, the largest value of a record of 50 samples may be assumed as the expected value of the 50-year event. The exceedance probability of this event is often estimated through the Weibull plotting position as (1 – 50/51) = 0.0196, whose 95% confidence interval has been demonstrated to be included between 0.0005 to 0.071 (Serinaldi et al., 2015). These values, in turn, correspond to return periods of 2000 and 14 years, respectively. Similar considerations can be drawn when the sample is analyzed statistically by fitting a probability distribution function. To account for this type of uncertainty, NOAA Atlas 14 provides the expected value for the precipitation intensity associated with a given return period and duration and the 90% confidence intervals, a feature that was not provided in previous governmental releases of this precipitation information. Despite this, design is almost always based on mean estimates. For example, precipitation data was pulled from one NOAA station located at North Carolina State University to illustrate the available data. For a 24-hour storm duration, the mean 1% AEP is 7.57 in. with a 90% upper limit estimate of 8.18 in. and a 90% confidence lower limit estimate of 6.97 in. This mean precipitation frequency estimate and its associated limits place the band of uncertainty of the 1% AEP equivalent to a 1.7% AEP and a 0.6% AEP. These differences translate into a range of probabilities that a 24-hour storm that produces 7.57 in. of precipitation will occur at least once in a 30-year period somewhere between 39.6% and 16.4%. Although the probabilities of these events occurring would classify them as rare events, the high impact of these rare events have been given special values in many newly rising fields such as smart city and autonomous driving (She et al., 2019).

Effects of Non-Stationarity and Climate Change in Hydraulic Design

Hydraulic designers are well aware of the fact that the built environment is non-stationary and that historical precedence is not always a great predictor of future conditions. This effect is evident in cases where new developments or other socio-economic/demographic changes change run-off levels and affect existing hydraulic structures. It has also become increasingly evident that climatological factors represent another type of non-stationarity that may need to be considered when defining design storm events (Serinaldi et al., 2015; Cheng and Aghakouchak, 2014; Salas and Obeysekera, 2014; CHy, 2012; Milly et al., 2008; Jain and Lall, 2001).

This effect has been examined within the NOAA Atlas 14, Vol. 2 release. Here, extreme event precipitation and its change over time was evaluated by performing a linear trend test on the 1-day maximum precipitation levels and its variance. Linear models were fitted to the time-series data for stations with a minimum of 50 years-worth of precipitation data. The results of this analysis are shown in Figure A.1 (trend of means) and Figure A.2 (trend of variances). Stations where the mean or variance increased are denoted with a ‘+’ symbol in green and those where the trends decreased are denoted with a red '-' symbol. From these figures, a number of stations in Eastern NC show a positive linear trend with respect to the magnitude and variance, suggesting that it is very important (more so than many other locations in North Carolina and around the region) to consider the most up-to-date precipitation data in order to properly identify the design intensity levels.

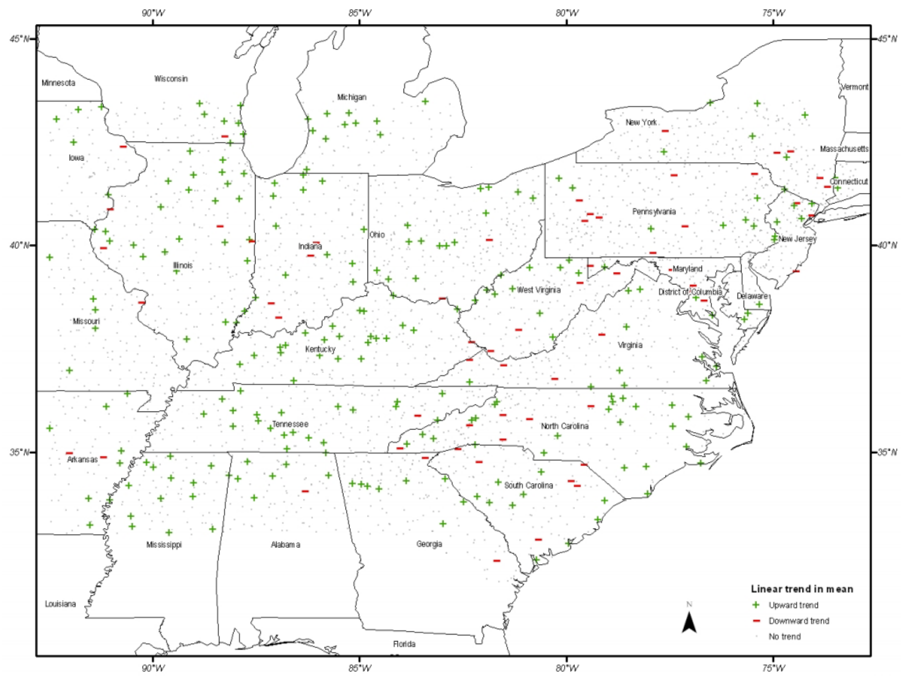

Figure A.1. Linear trend of 1-day annual maximum from rainfall stations with minimum of 50 years data, NOAA Atlas 14 Vol 2.

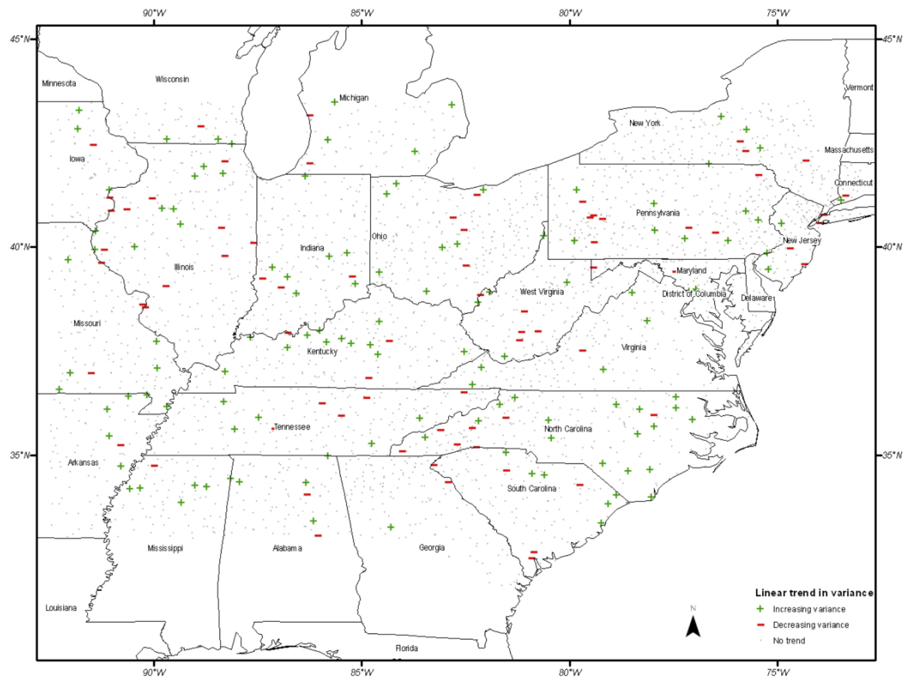

Figure A.2. Variance of 1-day annual maximum from rainfall stations with minimum of 50 years data, NOAA Atlas 14 Vol 2.

FHWA has released a manual to provide technical guidance grounded in the best available and actionable engineering and scientific data and approaches with a framework that is adaptable to future design situations (Kilgore et al., 2016). The manual provides specific information on risk and vulnerability assessments, planning activities, and design. The FHWA further believes that incorporating the potential effects of extreme events and climate change on flooding and then designing transportation system for more resilience when exposed to extreme flood events will enhance the lifecycle benefits.

The North Carolina DOT mentions that the Hydraulics unit has made a commitment to follow FHWA policy in regard to climate change and its impact on infrastructure design (Chang, 2016). Specifically, NCDOT highlights that "infrastructure is designed to handle impacts of a changing climate, such as sea level rise, increased frequency and magnitude of heavy precipitation and tropical storms, etc. Preparing for extreme weather events is critical to protecting the integrity of transportation and ecological (floodplain and wetland) systems and prudent investment of taxpayer dollars. The NCDOT staff will seek to follow FHWA's policy and guidance to develop cost-effective strategies to minimize climate and extreme weather risks and protect transportation infrastructure. For example, the design engineer will follow the FHWA publication Highways in the River Environment – Floodplains, Extreme Events, Risk, and Resilience, HEC-17 (FHWA-HIF-16-018), June 2016 (26)."

In addition, the National Cooperative Highway Research Program (NCHRP) produced a guide to provides a comprehensive framework for considering and incorporating climate change into the design processes for inland and coastal applications. Climate science and modeling is a dynamic field that is constantly changing and advancing and this guide is based on the current state of knowledge and understanding of possible future conditions developed by the climate community. The models from the Coupled Model Intercomparison Project (CMIP), are utilized to project a wide range of possible changes in future climate conditions. The main objective of these projections is to provide the data for engineers to better understand past, present and future climate changes arising from natural, unforced variability or in response to changes in radiative forcing in a multi-model context. The objective of the NCHRP guide is not to replace existing state DOT or other guidance, however it does provide additional tools, notably the CMIP5 climate processing tool, for evaluating the potential effects of climate change on transportation infrastructure (Kilgore et al., 2019).

NATIONAL, STATE, AND REGIONAL HYDRAULIC DESIGN PRACTICES

As mentioned previously, national guidelines outlined by FHWA for hydraulic design are utilized in order to prevent surface runoff from reaching the roadway and to remove rainfall or surface water efficiently from the roadway. National guidelines produced from FHWA do not differ greatly when looking at state guidelines, as the state guidelines reference often the FHWA reports. In particular, North Carolina's hydraulic design guidelines recommend engineers referencing the NCDOT's guidelines to also be up to date on FHWA guidelines (Chang, 2016).

NCDOT's hydraulic design guidelines, titled *Guidelines for Drainage Studies and Hydraulic Design,* was published in 2016 and is the result of consolidation and revised guidance from previous Guidelines with the emergence of new environmental, regulatory, and design challenges (Chang, 2016). These Guidelines are for use in design, analysis, and maintenance of drainage structures and systems designed and constructed by or in association with NCDOT-funded projects. The guidelines outline recommendations for design of drainage systems, and highlights the methods and procedures for calculations of runoff and storm discharges for a designed structure. The literature review that focused on the state of hydraulic design guidelines can be described in 2 parts: 1) the current state of design guidelines for extreme storm events and 2) best practices during the design process. In particular decisions made between the current state of design guidelines and how they may deviate from the guidelines when put into practice will be applied to the major storm events, Hurricane Matthew and Hurricanes Florence, which occurred in 2016 and 2018, respectively.

Current State of Practice in North Carolina

Current hydraulic guidelines utilized by the NCDOT rely on hydrologic methods that estimate peak storm discharge rates. Quantitative knowledge of these storm rates from watersheds is relevant to understanding and controlling a number of environmental processes, including erosion and sediment transport, pollutant loadings and travel times, and most notably for the purposes of this project, flooding and drainage (Genereux, 2003). Accurate estimation of peak storm discharge rates from watersheds is important to the design of drainage works along roadways and related infrastructure. The NCDOT guidelines state that the design engineer should select from a number of peak discharge methods, depending on the site's watershed characteristics. The methods utilized by the NCDOT for calculating peak storm discharges are based on the type of structure being designed, as shown in Table A.1. Once a hydrologic method has been selected and implemented, the results from that hydrological method calculations should be calibrated and compared with historical site information. In addition, the design engineer should consider potential future land use changes within a watershed over the life of a roadway structure and include this effect when estimating design discharges.

Table A.1. Hydrologic methods utilized by NCDOT from NCDOT Guidelines for Drainage Studies and Hydraulic Design, 2016.

Feature \ Hydrologic Method	FIS (for NFIP compliance)	USGS Methods	Rational Method (up to 20 ac)	Highway Hydrologic Charts	NRCS Method (for routing)
Bridges	X	X			X
Culverts	X	X			X
Storm Drain Systems			X	X	X
Cross Pipes (≤ 72 in. dia.)	X	X	X	X	X
Gutter Spread			X		
Ditches and Channels	X	X	X	X	
BMP Devices			X		X
Natural Stream Design	X	X	X		X
Storage Facilities					X
Floodplain Impacts	X	X			X

Methods used by the North Carolina DOT

<u>Flood Insurance Study (FIS) Method</u>

If a project study site is on a FEMA-regulated stream that is included in a published effective FEMA Flood Insurance Study (FIS), in conjunction with the National Flood Insurance Program (NFIP), then the discharges specified in the FIS should be used in the hydraulic model to demonstrate FEMA regulatory compliance. Streams studied by limited detailed methods will list the 100-year discharge and this information can be used directly by the designer. This method is utilized when structures or roadways have been designated to be within the NFIP and are required for FEMA compliance. The method is used in conjunction with a Floodway Map to determine whether or not a site is located in a Special Flood Hazard Area (SFHA), V Zone (front row beachfront properties), or a floodway.

Rational Method

The Rational Method is a simplified approach of calculating peak runoff based on rainfall intensity, drainage area, and land use coefficient, as seen in Equation (1) below.

$$Q = CIA \tag{1}$$

where;

Q = peak discharge (ft^3/s)
C = runoff coefficient (units are consistent with other terms),
I = rainfall intensity (in./hr), and
A = drainage area (acres).

The Rational Method is utilized for corresponding structures as outlined in Table A.1, and is employed when structures are being designed with drainage areas up to 64 acres. Typical runoff coefficients can be found in Table A.2 (Genereux, 2003). The rainfall intensity (I) can be obtained from NOAA Atlas 14, where they have already been calculated for a range of durations and storm event frequencies at specific locations. Further discussion of NOAA Atlas 14 Vol 2 and how it relates to NCDOT's design practices can be found elsewhere in this review.

Table A.2. Typical runoff coefficients to be used in rational method calculations.

Type of Surface	C
Pavement	0.7 – 0.9
Gravel surfaces	0.4 – 0.6
Industrial areas	0.5 – 0.9
Residential (single-family)	0.3 – 0.5
Residential (Apartments, etc.)	0.5 – 0.7
Grassed, steep slopes	0.3 – 0.4
Grassed, flat slopes	0.2 – 0.3
Wood/Forest	0.1 – 0.2

NCDOT Method

The NCDOT method uses a series of design charts known as the 'Highway Hydrologic Charts'. This method is only suggested for use in sizing small pipes (Chang, 2016). The method utilizes a hydrologic contour map of North Carolina (Figure A.3) and the corresponding contour is used in conjunction with the runoff chart (Figure A.4) to estimate peak runoff at a 50-year design frequency (Q50) (Chang, 2016). There are correction factors that can be used to obtain peak runoff at other design frequencies, which can be seen in Figure A.4. If the drainage area is larger than 20 acres, NCDOT recommends that the design engineer should consider if the Rational Method would provide a more appropriate estimate for peak discharge.

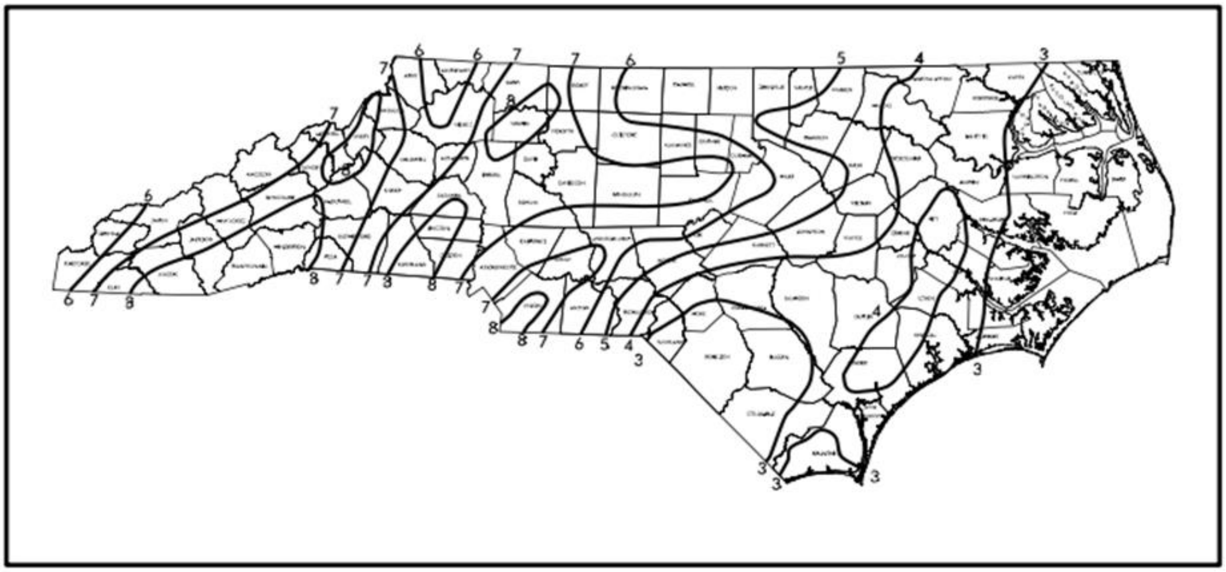

Figure A.3. Hydrologic contour map for North Carolina utilized in the NCDOT Method.

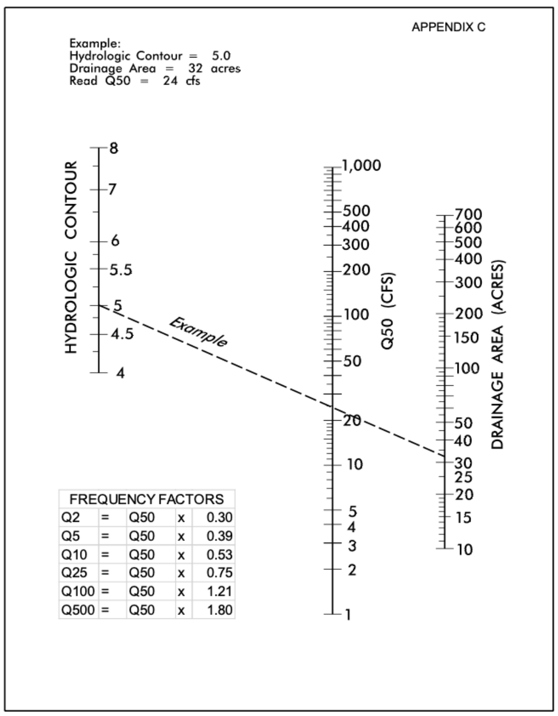

Figure A.4. Example of Runoff Chart for rural or urban drainage areas from NCDOT Method (1973) along with frequency correction factors.

<u>USGS Methods</u>

The United States Geologic Survey (USGS) has a number of reports that describe the estimation of peak discharge based on regional statistical regression analysis of watershed area and other characteristics such as land use. These reports outlines methods and procedures for utilizing stream gage data to calculate peak discharge for a specific location. The NCDOT recommends that precedence should be given to this analysis when a USGS stream gage is available at or near the study site. NCDOT also outlines peak discharge estimation procedure for sites where gages are available and unavailable, as presented in USGS report SIR 2009-5158 (Feaster et al., 2009) and USGS report SIR 2014-5030 (Feaster et al., 2014). For sites with gaged data available, there are three types of estimates for peak discharges that USGS provides:

- the recorded annual regulated peak flows are fitted to the log-Pearson Type III distribution,
- the appropriate regionalized regression equation developed for the hydrologic area of the gage location is used, and
- the first two types are used to make the estimate and are then combined using a weighted estimate method.

Additionally, if the site is not located at a reference stream gauge station, and the drainage area is within 50% of the drainage area of the reference gauge station, then the peak discharge estimate from the reference station can be adjusted (or transposed) for the study location. If the ungauged site is located between two gaged stations on the same stream, NCDOT guidelines recommend that two peak discharge estimates can be made using the above procedure and engineering judgment applied to determine which is the more appropriate of the two estimates.

Lastly, two reports have been produced by USGS outlining how the above procedures and methods for calculating and estimating magnitude and frequency of floods vary from land use type and drainage area. The USGS method (2014) is utilized for any drainage area under land use designated as urban, and any rural drainage area that is from 64 acres up to 1 square mile (640 acres). The USGS, 2009 is utilized for any rural drainage area from 1 square mile up to 400+ square miles.

<u>National Resources Conservation Service (NRCS) Method</u>

The Natural Resource Conservation Service produced a method that estimates discharge primarily based on land use and soil mapping as input parameters. Soil conditions such as hydrologic cover, soil type, and runoff conditions that are incorporated into the estimation method as Curve Numbers (*CN*), which can be found for urban and rural areas in Table A.3 and Table A.4 respectively. These *CN* values and rainfall estimates (*P*) can be utilized in conjunction with Figure A.5 to determine a runoff estimate for a given site's soil conditions.

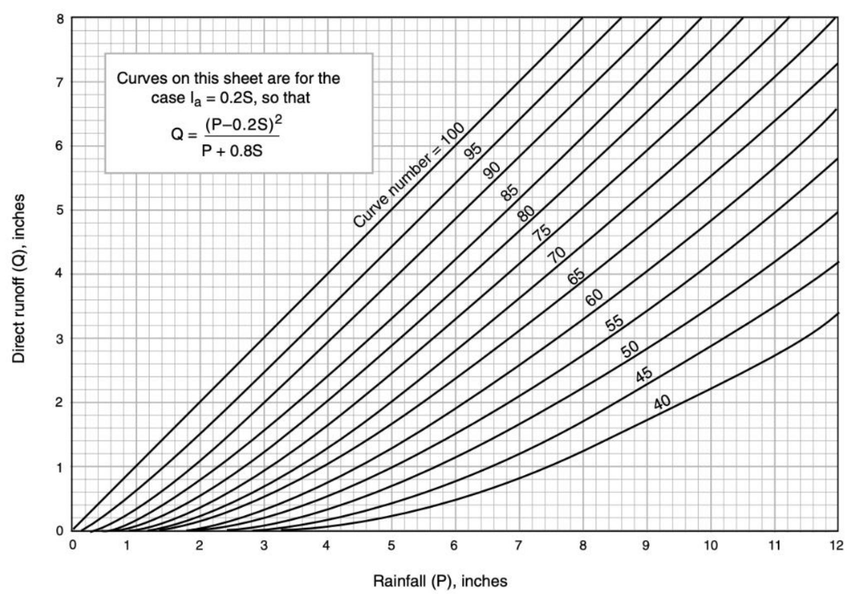

Figure A.5. Solution for runoff equation to be used in NRCS Method.

Design Frequency

Once the appropriate method has been selected, the design frequency for that roadway or structure must be determined as well. The design storm frequency for NCDOT drainage structures is determined based on variables such as the roadway classification, traffic volume, level of service, flooding potential to properties, and maintenance costs, among others (Chang, 2016). A summary of these frequencies as they relate to the peak discharge calculations mentioned previous shown in Table A.5. These return period based (frequencies) flood events that have been established as being an acceptable level for roadway overtopping, or when roadway overtopping is not involved, it will be the level of flood used for establishment of freeboard and/or backwater limitations.

Table A.3. Runoff Curve Numbers (CN) for urban areas.

Cover Description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 70%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right of way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right of way)		98	98	98	98
Paved; open ditches (including right of way)		83	89	92	93
Gravel (including right of way)		76	85	89	91
Dirt (including right of way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel much and basin borders		96	96	96	96
Urban districts					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residentual districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Table A.4. Example of Runoff Curve Numbers (CN) for agricultural (rural) areas.

Cover Description			Curve numbers for hydrologic soil group			
Cover type	Treatment	Hydrologic condition	A	B	C	D
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR+CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C+CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR+CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C+CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+CR	Poor	60	71	78	81
		Good	58	69	77	80
	SR	Poor	66	77	85	89
Close seeded or		Good	58	72	81	85
broadcast	C	Poor	64	75	83	85
legumes or		Good	55	69	78	83
rotation meadow	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Table A.5. Storm design frequency for NCDOT structures from NCDOT Guidelines for Drainage Studies and Hydraulic Design, 2016.

Roadway Classification	Frequency			
	Bridges, Culverts, and Cross Pipes	Storm Drain System: On Grade	Storm Drain System: At Sages (without relief)	Ditches
Major Arterials (e.g., I, US, NC)	50	10	50	10
Minor Arterials, Collectors, and Local Roads	25	10	25	10
Temporary/ Detours	10	-	-	10

NCDOT Practices

Unless otherwise noted, the design criteria from NCDOT are guidelines, not policies. As such, the hydraulic design that is put into practice can vary slightly, as each project can have unique circumstances that might require the design engineer to deviate from the guidelines. Discussions with NCDOT hydraulics engineers on design practices and how they might differ from the guidelines provided the following:

- <u>USGS Method</u>
  - Of note, USGS estimates for Region 3 (Sand Hills Region) are greatly underestimated. The Hydraulics Unit has adopted a composite calculation when designing structures within the Sand Hills area, utilizing a percentage of the estimates the USGS method(s) provides for Region 3 and from either Region 1 or Region 4.
- <u>NCDOT Highway Hydrologic Chart</u>
  - Both Western and Eastern NC divisions have indicated that this method is somewhat outdated and underestimates discharge values for design calculations. Discussions with the Eastern Hydraulics Unit indicated that the design process started moving away from this method around 2009. However, this method is still used on occasion according to brief interactions with the Western Hydraulics Unit.
- <u>NRCS (Natural Resources Conservation Service) Method</u>
  - Although rarely utilized currently, there is increasing interest to put into practice this method. However, from discussions with Hydraulics Unit members, the Rational Method is likely a faster method to develop an adequate discharge estimate, and allows the design engineer to make more general assumptions on variables and inputs to the design procedure.
- <u>Hydraulic Reports</u>
  - Hydraulic reports are produced from NCDOT when an existing structure suffers failure or damage from an extreme storm event. These reports consist of the Hydraulics Unit recommendation for upgrading the structure based on one of the previously mentioned hydrologic methods (USGS, Rational, etc.). These reports became more detailed and structured after Hurricane Matthew in 2016.

State of Practice from Nearby State Highway Agencies

For comparison, hydraulic guidelines for the states surrounding North Carolina were examined, and a brief summary of highlights is presented. The surrounding states mentioned below were considered for regional comparison to North Carolina. As mentioned previously, state agencies guidelines did not differ greatly from national guidelines outlined by FHWA. However minor variations on terms and definitions can have an impact on how the design process can be done.

Virginia (2002, revised 2021)

Virginia DOT utilizes hydrologic methods including the Rational Method, USGS Method (Rural and Urban), the NRCS method and a Modified Rational Method, and the Anderson Method. The Andersen Method was developed by USGS to evaluate the effects of urban development on floods in Northern Virginia, and is therefore not recommended for use outside of this region (VDOT, 2002). The max drainage area recommended for the application of the Anderson method is 570 sq.

miles. Similar to the NCDOT guidance on the Rational Method, the Rational Method and the Modified Rational Method in Virginia are recommended for sites up to 200 acres

Tennessee (2021)
Tennessee's hydraulic design guidance has indicated that there are preferred methods for hydrologic design calculations. The Rational and USGS Methods are preferred when the drainage area is less than 100 acres or greater than 100 acres, respectively. As mentioned previously, this drainage area criterion differs from NCDOT guidance (200 acres for Rational Method). However, there are certain situations where additional methods are required and utilized. Depending on the extent of man-made structures and the size of the drainage area, hydrograph methods such as the NRCS method and similar to the NCDOT Highway Hydrologic Charts may be utilized (TDOT, 2012). Additionally, the TDOT has determined that based on the design location, there are differing drainage area limits that affect the applicability of the USGS methods.

South Carolina (2009)
South Carolina has minor variations to the overall design process for hydraulic structures. For definitions, the agency refers to all drainage structures that are greater than 20 feet in length as bridges. All others (i.e. less than 20 feet in length) are labeled as culverts.

SCDOT uses different design frequencies for hydrologic estimates based on roadway classification (i.e. major primary routes, secondary routes, etc.). For secondary roads, 25-year peak discharge is used, as these are frequently smaller drainage areas and service less utilized roadways. Over designing for these secondary routes would present South Carolina with increased costs (Hulbert, 200). Primary and interstate routes utilize 50-year discharge, and all stream crossings are analyzed for 100-year events, similar to current practices employed by NCDOT.

Typical hydrologic methods utilized include the Rational Method, USGS Method (Rural and Urban), and NRCS method. Differing from NCDOT, the Rational Method in South Carolina is recommended to be used for sites up to 100 acres. The NRCS method is recommended to be utilized for locations from 100 acres to 640 acres. For locations with greater than 640 acres (1 square mile), the two USGS methods are employed depending on land usage (urban or rural).

Georgia DOT (2020)
The Georgia DOT hydraulic guidelines are similar to those of the SCDOT guidelines, but differ on the drainage area limitations for recommending which hydrologic method to be used in a hydrologic analysis (GDOT, 2020). The 3 methods recommended and their drainage area limitations were:

- Rational method – up to 200 acres, with a recommendation of utilizing for drainage areas less than 64 acres (similar to NCDOT),
- NRCS Method – up to 2000 acres and hydrologically homogenous, and
- USGS Methods – following referenced methodologies' recommendations.

Florida DOT (2012)
Although a comparative study undertaken by Genereux (2003) suggests that Florida uses USGS for some hydraulic design calculations, Florida's guidance from 2012 makes no mention of either USGS method (FDOT, 2000). It does suggest that the Rational Method is utilized along with FHWA HEC-12, which appears to be a precursor to the NCDOT Highway and Hydrologic Charts. Design frequencies for drainage systems range from 3-year to 50-year discharges, with the most common being 3-year design frequency.

Cataloging Location, Condition, and Performance of Structures

National Bridge Inspection Standards (NBIS)

Mention of NBIS during discussions with NCDOT prompted the research team to better understand these guidelines. NBIS sets regulations and requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of a state bridge inventory. Specifically, FHWA defines bridges that should be listed in the National Bridge Inventory (NBI), as “any road or street under the jurisdiction of and maintained by a public authority and open to public travel” (FHWA, 2004).

For the purposes of the research project’s goal of resiliency in structures, inspection intervals were determined to be a key factor. Under NBIS regulations, inspection intervals can be 12, 24, 48, or 72 months, with the base standard being 24 months. Lengthening or shortening the base inspection interval for bridges can be requested through an application process with the FHWA (FHWA, 2004). This involves a more rigorous process outlined, which such requirements as: Risk Assessment Panel (RAP) developed policy; risk categories, probability & consequence levels defined; Damage modes and attributes defined; Classification of each bridge into 1 of 4 risk categories; Risk process, criteria, & intervals documented. Lastly, according to NBIS regulations, culverts with multiple barrels with relatively small pipes can meet the definition of bridges. As such, bridges over 20 ft long are inspected on a 24-month cycle, subject to change based on the above criteria and FHWA approval.

Survey 123

Survey 123 is conducted by NCDOT after Hurricane Florence to document the locations and damages associated with this event on road infrastructure. The NCDOT has developed a GIS map containing the damaged locations. The following information is recorded for each location: road name, type of road, site configuration, type of damaged site (road, culvert, pipe, and/or bridge), extent of damage, photos of damaged sites, cost estimates, hydro report, information on utility damage, and supplementary notes. Data for approximately 3,000 locations across the North Carolina were recorded in this survey.

RP2021-03

RP2021-03 is an on-going NCDOT research project that is focused on predicting roadway washout locations during extreme rainfall events. Recent extreme rainfall events have revealed the transportation network’s vulnerabilities to road washouts. Currently, NCDOT reacts to these problems as are reported from the field. This inability to predict where washouts are likely to occur leads to long response times and inefficient positioning of resources. The availability of high-quality statewide elevation data, historical rainfall records and advances in computer processing presents the opportunity to modify and develop programs to predict where washouts are likely to occur during extreme rainfall events. The purpose of this project is to develop models and test several approaches for predicting crossing washouts based on forecasted rainfall. Since the data used for RP2021-03 project has overlaps with the current project, those data were added to data resources for the current study. This data contains information on damaged pipes after Hurricane Matthew and Florence and specific information on damaged pipes after Hurricane Matthew including existing pipes during the event, proposed pipes to be replaced, etc.

Resources to Estimate Event Intensities

RP2018-34

RP2018-34 is a NCDOT research project completed in coordination with North Carolina State Climate Office at NC State University. The Climate Office has developed an automated tool for NCDOT that provides rainfall monitoring and alert services using precipitation estimates derived from weather radar combined with available surface rain gauges. This project will continue the maintenance of the current Precipitation Alert Tool, enhance and modernize the web-based interface and continue the evaluation of using such data for precipitation monitoring and alerts. The researchers are continuing to investigate the tool further and will utilize as project gets into later tasks, which may add to our evaluation of future case studies and comparisons.

NOAA Rasterization

The Advanced Hydrologic Prediction Service in National Weather Service (NWS) website records the short-term observed and climatic trends of precipitation across the lower 48 United States (CONUS), Puerto Rico and Alaska (NWS, 2021). The observed precipitation is a product of NWS operations at the 12 CONUS River Forecast Centers (RFCs), and is displayed as a gridded field with a spatial resolution of roughly 4x4 km. Observed data is expressed as a 24-hour total ending at 1200 GMT (Greenwich Mean Time). The precipitation data are quality-controlled, multi-sensor (radar and rain gauge) precipitation estimates obtained from National Weather Service (NWS) River Forecast Centers (RFCs) and mosaicked by National Centers for Environmental Prediction (NCEP). The daily Quantitative Precipitation Estimates (QPE) raster maps for the duration of Hurricane Matthew and Florence are freely available for download from the Advanced Hydrologic Prediction Service in NWS website.

INFRASTRUCTURE RESILIENCE

The literature pertaining to transportation infrastructure resilience can be grouped into one of four main focus areas; frameworks for enhancing resilience, design for resilience, tools for assessment of vulnerabilities, and studies to identify and justify the return on investments in resilience initiatives to decision makers.

Frameworks for Enhancing Resilience

An important component of developing a robust resilience plan is the establishment of a strong framework through which to structure decision making and planning. The framework gathers in one sequence of steps the various activities that will enhance an agency’s resilience efforts to natural and human-caused hazards and threats (Dorney et al., 2021). It also guides transportation officials in; 1) understanding what their agency is currently doing with respect to resilience, 2) identifying where new or modified actions could be taken to enhance these efforts, and 3) recommending steps that can be taken to implement these actions. Several different resilience frameworks exist that have addressed various aspects of an organizational perspective on resilience (DOHS, 2021; Parker and Matherly, 2021; Filosa et al., 2017; NIST, 2016). While many frameworks exist, they generally share the same essential concepts including recognition of hazards of different types and severity (both in space and time), the presence of infrastructure elements at various locations (coordinates) across the network, the limited role of design in mitigating these unforeseen and extreme events, and the need for institutional changes to address the challenges brought on by the above. These institutional changes may include larger focus on data collection (inventories, condition assessment, central planning of rehabilitation/replacement plans, etc.). Here only the ones most relevant and well known with respect to transportation resilience are described.

A self-assessment tool is developed to assess the current status of an agency's efforts to improve the resilience of the transportation system through the mainstreaming of resilience concepts into agency decision making and procedures (Dorney et al. 2021). The self-assessment tool is based on a resilience framework, the Framework for Enhancing Agency Resilience to Natural and Anthropogenic Hazards and Threats (FEAR-NAHT). The framework is based on a series of 10 sequential steps shown in Figure A.6.

Figure A.6. The FEAR-NAHT framework (Dorney et al., 2021).

Although the self-assessment steps are presented in a logical sequence of what should come first before other steps are undertaken, in some cases, the steps could occur in parallel. For example, Step 8A, Assess Strategies for Enhancing Emergency Response Capabilities, is placed where it is because any enhancements to such capabilities relate to gaining a better understanding of the types of impacts that are expected (analyzed in Steps 5 to 7). However, some actions could occur for Step 8A prior to this (indeed, many are ongoing at DOTs already). Given that every agency will be different, the self-assessment tool is designed to allow transportation officials to enter into the self-assessment process in several different ways.

For those agencies just starting out – These agencies should begin with Step1 and proceed through all ten steps. The intent is for the self-assessment tool to provide a systematic and comprehensive examination of the agency's capabilities in all aspects of transportation system resilience.

For those agencies who consider themselves to have strong resilience-oriented capabilities – It would still be useful to begin at Step 1, which assesses what the agency is currently doing with respect to transportation system resilience. Based on this determination, they can then jump to the steps in the self-assessment tool where they think additional effort might be necessary or use the functional area templates in this guide to identify specific actions to enhance the capabilities where the agency believes there might be gaps.

For those agencies (or agency managers) concerned about specific agency functional areas and how resilience-oriented concerns could be better mainstreamed – They are still encouraged to start with Step 1 to get a good sense of what the agency is currently doing with respect to resilience-oriented efforts. The functional area templates in the guide also provide very useful guidance on what agency managers should consider enhancing capabilities in their functional area responsibility.

The necessary actions based on the score of each step is recommended for each step and each functional area of the agency. The starting point of this self-assessment framework is understanding the current state-of-practice of the organization's resilience activities to determine where enhancements to these efforts can occur. Improving the current state can be organized internally (What can the agency do better?) and externally (How can the agency better interact with key partners and stakeholders to improve collective efforts to improve the resilience of the transportation system?). Based on experiences with system disruptions, effective communications among the many different participants responding to an incident or disruptive event is critical to the overall success of an agency's resilience efforts. Understanding the current state and organizing internal and external ways to improve the system can identify strategies/actions that can be taken in the short term, with low costs, and limited need for time-consuming data analysis. The agency should implement these early wins for the following reasons:

1. They yield immediate (though perhaps not major) improvements in system resilience.
2. They provide a signal to both the public and others within the DOT that it is moving forward.
3. They send a message to outsiders and insiders regarding the agency's willingness to make changes.
4. They provide concrete examples of what types of actions fall under the agency's overall strategy.
5. They can identify the barriers/constraints that need to be overcome to make later implementation more successful.

In order to further improve the resiliency of the system in the long term, the sources and magnitude of the likely hazards and threats facing a transportation system should be examined and understood. Specific assets within the transportation system that are more vulnerable or at-risk (i.e., have greater exposure and/or higher consequences of failure) need to be identified and prioritized for more detailed study of adaptation options. State DOT experience with system resilience activities has shown that the following agency functional areas are strong candidates for making improvements: 1) emergency response, 2) operations and maintenance programs, 3) project design and development (assuring a more adaptive design approach), and 4) asset management plans and programs. In these functional areas, resilience projects will likely be part of the normal project programming process although this step assumes that some special considerations be given when doing so. Such consideration might also be applied to projects being undertaken for purposes other than enhancing resilience but that incorporate resilience treatments. Ultimately, influencing the types of projects implemented by the agency is one of the most important output measures for an agency's resilience program. In Step 10 of the FEAR-NAHT framework, the agency examines how resilience concepts can be incorporated into transportation system performance monitoring and how agency actions aimed at enhancing the resilience component of this performance can be better managed. Many transportation agencies have adopted performance-based decision-making and program management approaches for identifying the most cost-effective investments. Such approaches are data-driven, performance-based, and results-oriented.

The basis for this self-assessment system is a comprehensive literature review, specifically feedbacks and discussions from Transportation Resilience Innovations Summit and Exchange (RISE) Conference, an international conference on transportation system resilience held in October 2018. This system was also tested in several state DOTs such as Oregon DOT (ODOT) for further verification. The underlying technical component in this tool is a capability maturity model (CMM), which is a matrix system of assessment. The user is first given a series of questions about how their agency is handling different situations and their current practices pertaining to one or more of the 10 steps in the FEAR-NAHT framework (Dorney et al., 2021; Flood and Meyer, 2021). For example, one question pertaining to the agency's ability to assess current practice might be "As part of your self-assessment process, have you examined best practices from other agencies and organizations?" or "Are maintenance data reviewed to identify assets with previous impacts / repeat failures?". Then, users can choose from three pre-selected answers to these questions. Each answer reflects a level of maturity. The higher the level, the higher the score and the more resilience oriented. For example, to answer the first question mentioned above, an agency might answer one of the following three ways.

- Level 1 (1 point) – We identify best practices primarily from the literature and from what we hear at conferences and meetings.
- Level 2 (2 points) – We have proactively identified function-specific best practices from other organizations and have used them as indicators to measure our progress in specific functional areas.
- Level 3 (3 points) – We have proactively identified best agency-wide practices from other organizations and used them as benchmarks to measure our own progress. Agency leadership is involved in this comparison and identification of improvements to the agency.

A score is determined for each of the mentioned 10 steps in the framework and a total score is summed across all the steps to determine how mature the organization is with respect to undertaking resilience-oriented activities and efforts. Based on the percent score, a series of recommendations by functional areas are provided for the user to achieve or maintain the highest level of resilience capability. The underlying concept is that periodic examination of all agency actions contributing to a resilient transportation system is an important foundation for a resilience-oriented agency.

Caltrans is currently conducting a study on developing a framework for statewide vulnerability assessments (Ongoing FHWA Pilot Project). The assessments will identify vulnerabilities along the state highway system from climate stressors including sea level rise, storm surge, changes in temperature, precipitation, and increased wildfires. Effective communication is imperative for education and outreach, both internal to Caltrans and external partners to develop and integrate adaptation measures. The result of the grant will be translatable to a range of transportation agencies given the increased need for climate change communication.

Another approach to develop a resilience framework is a stage-wise framework which is built in the current practices of Hawaii DOT (HIDOT) (Sniffen, 2021). HIDOT considers resiliency in three stages of short term, mid-term, and long term. In the short-term stage, quick fixes such as bags along coastal roads, clear streams and bridges, slope failure warning upon movement are applied. These quick fixes are in place until more fundamental decisions are made. In the mid-term stage, the infrastructure and system operation are maintained by small realignments, consideration of use of less costly and more efficient fixes for facilities in areas forecasted to be impacted by sea level rise (SLR) and using "sandsaver" (Figure A.7) for reduction of erosion rate and shoreline stabilization. In the long-term stage, more decisions will be made for the climate adaptation action plan to address concerns before the formal planning process starts. The FEAR-NAHT framework provides a detailed step-by-step self-assessment framework to first determine the issue and recommend the actions in each area, so this framework is more focused on identifying the issues and then take actions specific to the identified issue, but HIDOT framework is a combination of taking actions in different stages and dynamic monitoring the impact of the actions. Considering early wins in the FEAR-NAHT framework and actions in short-term or mid-term stage in HIDOT framework represent the same concept in a resilience framework.

Figure A.7. A type of sandsaver (Sniffen, 2021).

Design for Resilience

The collective decisions made based on lessons learned after previous disasters can be used to improve the design of the infrastructure to be resilient against threats and hazards. These decisions are based on limited information and location-specific analysis, which needs to be monitored and modified over time as new information becomes available. While much of the literature on resilience pertains to institutional approaches to embedding resilience as a guiding principle, there does exist some evidence that improvements in design practices can also be a part of a resilience framework. A review of this literature suggests design improvements can be done with respect to the following: (1) transportation-related hydraulic assets, (2) asphalt mix design, and (3) pavement structure design. The design improvements in these areas can be applied by changing the design thresholds and standards.

The classic approach for designing transportation-related hydraulic assets (e.g., bridges, culverts, channels, or storm drains) is to use a design event (Kilgore et al., 2016). For example, under the guidelines of the NCDOT hydraulic standards, culverts built on minor arterial or local roads have storm design frequency of 25 years (a 4% AEP). This approach is well advised since it will help designers manage risk and also provides a specific event magnitude that needs to be considered in engineering calculations to size the elements of the structure. The required design events are usually specified in policy documents by a specific value or a range of values for probability of exceedance. The hydrologic quantities such as peak discharge, runoff volume, and peak discharge corresponding to the target AEP should be estimated, and the structure should be designed to manage that quantity. Storm design frequency, and therefore AEPs, vary based on the type of roadway and traffic volume as referenced in Table A.5. On the other hand, the return period description might be statistically misleading. For example, a 50-year event (2% AEP) has a 33% probability of occurring at least once in any sampled 20-year period. It also has a 6% and 0.7 probability of occurring at least twice and three times in the same period respectively. These probabilities can be readily calculated from simple probability theory.

Another approach to design a resilient infrastructure suggested by FHWA is reducing vulnerability by either reducing the sensitivity of the assets to extreme events or by enhancing the adaptive capacity of the assets, or both (Kilgore et al., 2016). The strategies that are part of this design approach include reinforcing roadway components, evaluating the watershed for debris production potential, evaluating stream geomorphology for channel stability, etc. There are also adaptation strategies specific to pavements and soils which are suggested by FHWA (Choate et al., 2017) including adjusting mix design to compensate for the higher temperatures and high intensity, short duration rain events and adjusting the pavement structural design by increasing steel content and using a stiffer binder in the asphalt overlay during rehabilitation.

In Broward County in Florida, certain improvements were added to the resilience guidelines to mitigate the issues of sea level rise, increased storm intensity, coastal and inland flooding, extreme rainfall and drought, etc. (Jurado, 2021). The modified resilience standard of the tidal flood barrier requires five (5) feet to the North American Vertical Datum (NAVD) by 2050, but allows four (4) feet to NAVD until 2035 for the two (2) foot sea level rise.

The Climate Resilience Guidelines (CRG) was presented by the Port Authority of New York and New Jersey (PANYNJ) with the purpose of adopting a science-based approach to manage climate-related risks and supporting the incorporation of climate change projections—particularly sea level rise—into the full range of engineering and architectural design standards (Ensor, 2021; PANYNJ, 2018). This guideline serves as a supplement to applicable building code requirements and providing a clear methodology for factoring projected future sea level rise into project design criteria, while maintaining the flexibility of project teams to develop cost-effective design solutions. The Port Authority takes a code-plus approach to design for future sea level rise, meaning that the Climate Resilience Guidelines supplement, but do not supersede, applicable codes and standards. The American Society of Civil Engineers (ASCE) standard Flood Resistant Design and Construction (ASCE/SEI, 2014) is fully incorporated into New Jersey Building Code and serves as the basis for New York City Building Code Appendix G (Flood-Resistant Construction). ASCE 24 dictates that construction in the FEMA 1% (“100-year”) AEP floodplain is subject to specific, safety-driven requirements, most notably the establishment of a Design Flood Elevation (DFE) comprising the base flood elevation (BFE) and the freeboard. The project-specific FEMA BFE is the elevation of the 100-year flood including waves and is derived from the FEMA Flood Insurance Rate Map(s). Freeboard is a factor of safety usually expressed in feet above the BFE, as dictated by the requirements of ASCE 24 or the applicable code.

The Climate Resilience Guidelines supplement ASCE 24 and applicable local codes in two primary ways:

- Adjustment of the BFE for Sea Level Rise: The Guidelines augment the applicable FEMA BFE by adding the relative increase in future sea levels (based on the NPCC projections) over the project's expected service life.
- Consideration of future floodplain expansion: Rising sea levels may also lead to expansion of the 100-year tidal floodplain over time, depending on local conditions. Therefore, the Guidelines apply to projects sited in or proximate to today's 0.2% AEP ("500-year") floodplain or in the projected future tidal 100-year floodplain, in addition to the current FEMA 100-year floodplain.

Approaches to increasing the resilience of an asset to flood damage and/or operational disruption generally fall into the basic categories of; (a) elevate, (b) relocate, (c) protect, or (d) accommodate. These approaches include:

- Coastal protection, including wave attenuation (placement of levees, berms, or living shorelines) [protect],
- Site selection and relocation (placement of structures on higher ground or within flood protected areas) [relocate],
- Perimeter protection (placement of flood walls and/or deployable protection measures to limit flood risk within a defined perimeter) [protect],
- Elevation (raising an entire structure above the DFE) [elevate],
- Elevation of utilities and critical equipment such as controls, outlets, generators, etc. [elevate],
- Wet floodproofing (allowing floodwaters to enter and exit certain non-critical, generally unoccupied portions of a structure to equalize flood loads, subject to code restrictions) [accommodate],
- Dry floodproofing (placement of permanent, deployable, and/or temporary mitigation measures to prevent intrusion of flood waters into a structure) [accommodate],
- Pumps (to prevent build-up of incidental leakage in a dry floodproofed structure or perimeter protected site) [accommodate], and
- Backflow prevention (the installation of devices to prevent surge intrusion through storm or sanitary sewers) [accommodate].

ASCE/SEI 24-14 is the standard that provides minimum requirements for flood resistant design and construction of structures that are subject to building code requirements and that are located in Flood Hazard Areas. This standard applies to the following: (1) new construction, including subsequent work to such structures and (2) work classified as substantial improvement of an existing structure that is not an historic structure (Figure A.8).

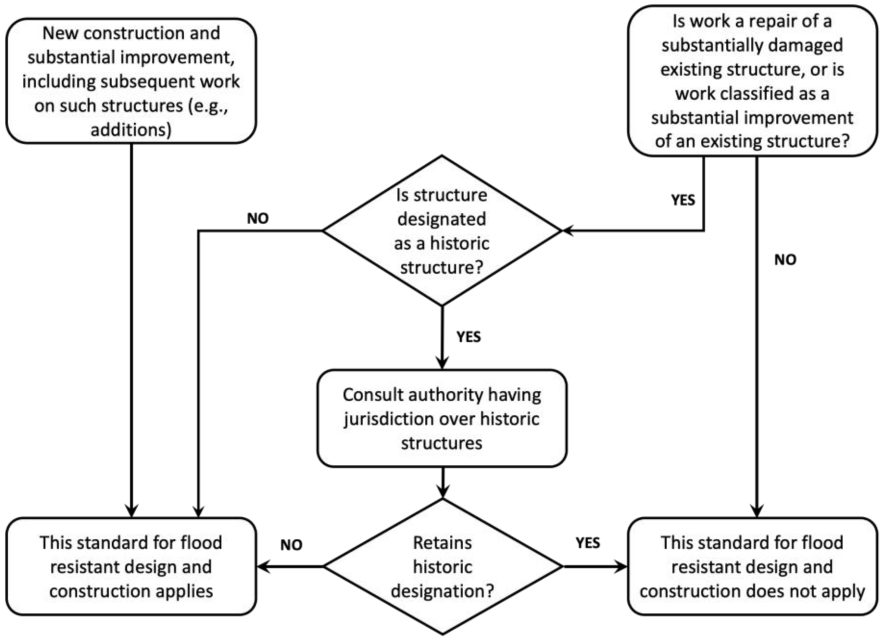

Figure A.8. Illustration of application of ASCE 24 standard (*32*).

Vulnerability Assessment

An important step in improving resiliency is identifying the vulnerable locations to prioritize for improvement. A vulnerability is a consistent part of any resilience framework as it critically assesses hazards, their likely location, and the existence of infrastructure at those locations. There are different frameworks that rely on vulnerability assessment of different infrastructure including vulnerability and resilience framework for Atlanta region. This approach represents a general framework for the assessment of vulnerability of different elements of transportation infrastructure and resiliency of different elements of transportation infrastructure against extreme events (WSP, 2018). NCHRP 20-83(05) provides a guide for an eight-step diagnostic framework for undertaking an adaptation assessment. This framework includes the steps that should be taken if transportation officials want to know what climate stresses the transportation system might face in the future, how vulnerable the system will likely be to these stresses and what strategies can be considered to avoid, minimize, or mitigate potential consequences. Methods to incorporate adaptation concerns into a typical transportation planning process are also described (Meyer et al., 2014).

Canada's engineers conducted studies to ensure infrastructure adapts to the impacts of anticipated climate changes (PIEVC, 2008). The Public Infrastructure Engineering Vulnerability Committee (PIEVC) presented a protocol for infrastructure vulnerability assessment and adaptation to a changing climate which consists of a process to assess the infrastructure component responses to impacts of changing climate. Their five-step protocol provides a procedure for sifting through data for developing relevant information on specific elements of the climate and characteristics of a given infrastructure. The protocol then considers how this information might interact and result in the infrastructure being vulnerable or adaptive to climate change. The assessment was conducted on four categories of Canadian public infrastructure: a) stormwater and wastewater, b) water resources, c) roads and associated structures, and d) buildings.

The United Kingdom Highway Agency developed an adaptation framework to determine and apply responses to the challenges of climate change (Parsons Brinkerhoff, 2008). This framework provides a platform for decision makers to examine their individual business areas, including standards, specifications, maintenance, and the development and operation of the Highways Agency network. The Highways Agency's Adaptation Framework Model (HAAFM) provides a seven-stage process that defines the objectives and decision- making criteria, identifies climate trends that affect the Highways Agency, determines Highways Agency vulnerabilities, evaluate risks, determine options analysis, develops and implements adaptation action plans, and reviews adaptation program.

The U.S. Department of Transportation (USDOT) conducted a comprehensive, multi-phase study of Central Gulf Coast region to better understand climate change impacts on transportation infrastructure and identify potential adaptation strategies (Choate et al., 2014). For Phase 2 of the study, USDOT developed methods for evaluating vulnerability and adaptation measures that could be used by other transportation agencies and pilot tested them on the transportation system in Mobile, Alabama. The project team evaluated the impacts on six transportation modes (highways, ports, airports, rail, transit, and pipelines) from projected changes in temperature and precipitation, sea level rise, and the storm surges and winds associated with more intense storms. The project resulted in a detailed assessment of the Mobile transportation system's vulnerability as well as approaches for using climate data in transportation vulnerability assessments, methods for evaluating vulnerability and adaptation options, and tools and resources that will assist other transportation agencies in conducting similar work.

Generally, in these frameworks, the agency identifies the critical assets of the system and determines the stressors that affect those critical assets. Then the vulnerability assessment is conducted for each set of assets and affecting stressor, i.e., the assessments are conducted in a narrow way to focus on only certain types of infrastructure (e.g., asphalt pavements) or elements of the infrastructure (e.g., pipes) subjected to a single independent hazard such as coastal flooding, extreme heat, wildfire related flooding, etc.

Three different approaches that practitioners can follow to assess vulnerability. The stakeholder input approach, indicator-based desk review approach, and engineering-informed assessments (Filosa et al., 2017). The first two approaches are primarily used for systems level or area analyses, while the third approach, focuses on a specific transportation asset. Each approach differs by the types of stakeholders involved, the forms of information required, the formats of the final vulnerability assessment findings, and/or scale. These approaches are not mutually exclusive; often a vulnerability assessment includes elements of each approach. A stakeholder approach may involve conducting interviews with local transportation practitioners, such as maintenance and operations staff, engineers, and emergency responders. These individuals have local knowledge of how the study assets are used, and they have experience with what climate-related issues currently exist and how changes in climate may impact the assets.

Washington DOT (WSDOT), Oahu metropolitan planning organizations (MPO), and FHWA in collaboration with Netherlands applied this approach to qualitatively assess the facility vulnerability. Under an indicator-based desk review approach, a study team uses quantitative data on assets (e.g., elevation, geo- graphic location, and existing flood protection) and projected climate stressors (e.g., sea level rise, temperature increases, and changes in streamflow) to serve as indicators to evaluate potential vulnerabilities. Southeast Florida, Maryland SHA, MnDOT, and FHWA in collaboration with Netherlands used this approach to evaluate the vulnerability of the assets.

Engineering-informed adaptation studies are characterized by a greater level of asset specific data and analysis than a broader assessment that assesses multiple assets. A detailed engineering assessment offers a way to evaluate risks to particular transportation assets in response to climate stressors. Engineering assessments that consider future climate change are integral to identifying where and to what extent assets may incur damage from climate stressors. These assessments also help agencies anticipate the effectiveness of specific adaptation measures and their respective return on investment if adopted. An engineering assessment involves the following elements: (a) understand site context and future climate; (b) test the asset against future climate scenarios; (c) develop, evaluate, and select adaptation; (d) review additional considerations in terms of socioeconomic, budgetary, and political considerations; and (e) monitor and revisit as needed measures. FHWA in collaboration with Alabama and State DOTs and local transportation agencies in the New York-New Jersey-Connecticut region conducted vulnerability assessment on different assets in these locations. The approaches described below mostly fall into the second and third type of assessment approaches. To be most effective, these assessments are often performed continuously to provide the necessary information to iteratively improve the planning, design, and management of infrastructure systems so that they are more resilient. Specific examples of vulnerability assessments identified in the literature are summarized in Table A.6 below and more detailed descriptions of a selection of these are provided in the paragraphs below.

Table A.6. Summary of vulnerability assessments.

Agencies	Hazard/Stressor	Infrastructure/ Element	Takeaways/Notes
Connecticut DOT and FHWA	Inland flooding/Extreme rainfall	Bridges and culverts	35% of the 52 structures had inadequate hydraulic design
Pennsylvania DOT	Sea-level rise	Roads and bridges	Developed a tool to be used in resilience decision making
Massachusetts DOT and FHWA	Sea-level rise/flooding	Roads, tunnels, buildings, portals, etc	Developed a tool to be used in resilience decision making and predicted 25 structures and 12 portals by 2030 and 51 structures and 54 portals by 2070 or 2100 will become vulnerable.
Caltrans	Extreme temperature, precipitation, run off, fire, landslide, sea-level rise and storm, dune and cliff erosion	Roads	District 1 indicated the majority of the segments received low vulnerability scores: 95% of segments received lower than 60.
Maryland Department of Transportation State Highway Administration and FHWA	Sea-level rise, storm surge, and precipitation change	bridges	33 bridges are highly vulnerable to sea level change, 172 are highly vulnerable to storm surge, and 102 are highly vulnerable to precipitation change
43 U.S. State DOTs	Precipitation	Stormwater infrastructure	Eight out of 43 states need to revise their standards. All states need to revise their standards under both the RCP 4.5 and RCP 8.5 emissions scenarios for 2050.

The Connecticut DOT, sponsored in part by the FHWA, conducted a systems-level vulnerability assessment of bridge and culvert structures six feet to 20 feet in length from inland flooding associated with extreme rainfall events (CDOT, 2014). The results of their assessment show that of the 52 structures evaluated, 34 (65%) satisfied the design water surface elevation criteria for the specified design frequency discharge based on the current precipitation estimates. However, 13 of these structures may require some corrective action due to scour. Further it was found that 18 of the 52 structures (35%) do not satisfy the hydraulic design criteria and are therefore hydraulically inadequate based on the current precipitation estimates.

Lopez-Cantu and Samaras (2018) evaluated State DOT design manuals for stormwater infrastructure from 43 states in U.S. An index between 0 to 1 for each climate region was developed to assess each state's requirements, called regional index. Higher values represented states that have higher standards (and thus, were more prepared for rare storm events). The percent change between the previous (TP40) and current (Atlas 14) precipitation frequency document were estimated. Using these regional index values, the observed change in precipitation frequency estimates, and whether the design manual standard was published after latest precipitation frequency document, states were assigned priority value of 1 to 4, 1 being the lowest and 4 the highest priority to immediately revise their stormwater standards. Eight out of 43 states were found to have the highest priority, i.e., these states experienced a 10% or greater increase in precipitation between Atlas 14 and TP40, published their current design manual prior to the release of the latest precipitation document, and were estimated to be in the lower half of their regional index for design return period standards. In addition, these states should assess whether existing infrastructure requires additional adaptive capacity to manage observed precipitation increases. The priority increased for all states under both the RCP 4.5 and RCP 8.5 emissions scenarios for 2050.

Some agencies have gone beyond simple vulnerability assessments and developed tools to systematically and quantifiably assess for vulnerabilities across larger sections of their network. To assist in the collection and identification of Pennsylvania DOT transportation assets vulnerable to extreme weather, a web-based survey and data collection tool was established. The PennDOT District and MPO staff mainly contributed to the survey. Using the survey interactive map, over 450 locations vulnerable to flooding, snow, high winds, fires, earthquake, high temperature, and landslides were identified within the state. Flooding considered to be the primary issue in the state. The DOT has developed a tool that dynamically predicts the extent of the flood plain based on increased rainfall scenarios. The DOT uses this tool to assess the inundation of its roads and bridges based on increased stream depths and sea-level rise. This information helps to modify their design approaches to have resilient infrastructure (PennDOT, 2017).

The Massachusetts DOT has developed a tool to conduct a vulnerability assessment on Central Artery/Tunnel (CA/T) system in Boston (located on part of I-93 and I-90) which is one of the most valuable components of Massachusetts' transportation infrastructure (Bosma et al., 2015). The vulnerability assessment tool was developed based on ADvanced CIRCulation model (ADCIRC) coupled with Simulating WAves Nearshore (SWAN) model to predict coastal inundation. This model simulates storm-induced waves in concert with the hydrodynamics. The vulnerability assessment determined the probability of coastal flooding by 2030, 2070, and 2100 under a future sea-level rise scenario in Boston. The results showed that 25 structures and 12 portals by 2030 and 51 structures and 54 portals by 2070 or 2100 will become vulnerable to sea-level rise or flooding.

Caltrans has codified its process for evaluating the vulnerability of transportation assets in District 1 due to various climate change factors into a decision support tool to assess adaptation strategies for vulnerable assets (Cros et al., 2014). The climate stressors including extreme temperature, precipitation, run off, fire, landslide, sea-level rise and storm, dune and cliff erosion were considered in this assessment. In this study, the vulnerability score for each Transportation Concept Report (TCR) road segments was defined as criticality (including socioeconomic, operational, and health and safety importance) score multiplied by potential for impact (including exposure, sensitivity, and adaptive capacity) score. A vulnerability score ranges between 0 to 100. A technical advisory group (TAG) and local stakeholders were engaged throughout the assessment process. The distribution of vulnerability scores for District 1 indicated the majority of the segments received low vulnerability scores: 85% received a score of lower than 50, and 95% received lower than 60. This tool was developed and then tested on the four prototype locations in Del Norte County, Humboldt County, Lake County, and Mendocino County.

In 2017, FHWA presented the Vulnerability Assessment and Adaptation Framework, which is a manual that helps transportation agencies, and their partners assess the vulnerability of transportation infrastructure and systems to extreme weather and climate effects. One of the tools introduced in this manual is Vulnerability Assessment Scoring Tool (VAST) (Filosa et al., 2017). VAST is an Excel-based tool to calculate metric-based vulnerability scores in terms of the three vulnerability components (exposure, sensitivity, and adaptive capacity). The VAST vulnerability scores range from 1 to 4, 1 representing low vulnerability and 4 representing high vulnerability. Based on the scoring scales given for each metric, first, the VAST converts observed values for an asset to its metric-level vulnerability scores, and then calculates weighted averages of metric-level vulnerability scores to obtain the component-level vulnerability scores of the asset. Finally, the tool calculates the overall vulnerability score of an asset by averaging its three component-level vulnerability scores. The overall vulnerability scores of individual transportation assets can be used to prioritize the assets for maintenance, repair or upgrading. VAST also examines how agencies can integrate climate adaptation considerations into transportation decision-making (Filosa et al., 2017). The VAST tool has been used by different agencies including the Kentucky Transportation Cabinet (KYTC) and Maryland Department of Transportation State Highway Administration (MDOT SHA) (Bhat et al., 2019; Blandford et al., 2019). The KYTC conducted vulnerability assessment for bridges and pavement under extreme heat and extreme precipitation and ranked the assets based on their vulnerability. The vulnerability assessment done by MDOT SHA indicated that, of the 8,588 bridges evaluated, 33 are highly vulnerable (i.e., scoring at least 3 out of 4) to sea level change, 172 are highly vulnerable to storm surge, and 102 are highly vulnerable to precipitation change.

Return on Investments

As discussed previously, the process of improving resiliency is more of an iterative process since every framework and design needs to be monitored and modified over time. Each decision to change the framework and design needs to be carefully made since any change in this scale should economically be justified. Therefore, benefit-cost and return on investment analyses are another important element of improving resiliency of the infrastructure. The resiliency of the infrastructure consists of various interdependent elements which makes this type of return-on-investment analysis particularly complicated and for this reason there are a few studies in this area which are in their preliminary stages. Some examples of return on investment (ROI) studies are presented here.

There are tools to evaluate the return on investment on improving the resiliency of the infrastructure. Arizona DOT (ADOT) uses Resilience Investment Economic Analysis (RinVEA) to integrate extreme weather and climate justification into asset management and financial decision making (Olmsted, 2021). A framework presented in NCHRP Report 938 uses the FHWA HEC-17 approach to conduct Cost Benefit Analysis (CBA) of adaptation strategies (Dewberry, 2020; McGinley, 2021). The USDOT has developed its Resilience and Disaster Recovery (RDR) Tool to estimate ROI (Lewis et al., 2021). This tool evaluates resilient infrastructure return-on-investment (ROI), ranks resilience investments by performance, and ROI and/or ranking can be used by analysts as a factor or weighting to inform project prioritization. Also, this tool is location agnostic and geospatially explicit, leverages existing tools available to DOTs and MPOs, and addresses a variety of hazard conditions and is intended to be hazard agnostic. The RDR Tool Suite is being piloted with Hampton Roads Transportation Planning and Virginia DOT.

The Urban Land Institutes *Business Case for Resilience in Southeast Florida* report presents estimates of the economic consequences to coastal counties in the region if local governments and business communities fail to take action to mitigate the impacts from tidal flooding and frequent coastal storms (ULI, 2020). The analysis takes a regional perspective, considering the impacts to the region given the interconnected economies across all four counties. In addition, the study estimates the economic benefits from certain types of adaptation actions designed to mitigate the coastal hazard risks. These adaptation actions could all make a difference, but some of these actions are more suitable for some counties than others and each county may need a customized approach to address its own unique resilience challenges. The adaptation strategies focus on both community-wide initiatives and individual building-level ones as well. In community-wide strategies, beach nourishment, sand dunes, green infrastructure, and seawalls are considered. In building-level strategies structure elevation, permeable surfaces, dry floodproofing, and wet floodproofing are applied. The benefit-cost analysis for these strategies showed that community-wide and building-level strategies result in 2.08 and 3.97 benefit-cost ratio, respectively.

Members of the MPO of Hillsborough County in Florida, as part of their participation in FHWA's 2013–2015 pilot projects, identified cost-effective strategies to mitigate and manage the risks of coastal and inland inundation (Holsinger, 2017). The purpose was to incorporate those strategies into the Hillsborough MPO's 2040 long-range transportation plan and other transportation planning and decision-making processes. The pilot project looked at several critical assets in the region and evaluated mobility and economic impacts if any of those facilities were to be out of service. Gandy Boulevard, part of an important link between Hillsborough and neighboring Pinellas County, was one of the assets evaluated. The pilot project identified a 0.38-mile (0.6-kilometer) segment on Gandy Boulevard between the Selmon elevated expressway and the raised Gandy Bridge as a critical hurricane evacuation route from adjacent Pinellas County. Following the pilot project, the Hillsborough MPO coordinated with the Tampa Hillsborough Expressway Authority, the owner of the facility, to conduct a follow-up study. The study looked at additional risk evaluations specific to the vulnerable segment, refining strategies and providing conceptual designs and pre-engineering cost estimates to offer low-risk, high-benefit solutions for implementation. The follow-up assessment suggests that the approximately $1.9 million adaptation strategies recommended would show a positive return on investment compared to the more than $3million cost to replace the facility.

The Massachusetts DOT (MassDOT) developed a probabilistic flood risk model incorporating sea level rise and storm surge for the Boston Harbor to determine when different levels of strategies would be needed to protect the Central Artery highway tunnels and associated assets (Ongoing FHWA Pilot Project). MassDOT found that at several locations, temporary flood barriers that could be placed in advance of a storm event and removed during normal operations would be sufficient to reduce risk to acceptable levels from now until 2030, at which point permanent flood protection strategies will likely be needed. As such MassDOT is currently working to finalize the design to protect various operational and tunnel assets. Various protection strategies are being considered and a being narrowed to those that are readily deployable such flood planks and those that reduce the number of personnel needed during deployment. Using the USDOT Hazard Mitigation Cost Effectiveness Tool, MassDOT found the benefit cost ratio for the temporary flood barriers is 58:1.

PennDOT will conduct advanced hydrologic and hydraulic analyses at two sites in Allegheny and Delaware Counties (Ongoing FHWA Pilot Project). They will address impacts of future extreme precipitation events by developing viable alternative structure designs and conducting economic analyses to determine the most cost-effective adaptation options.

Utah Department of Transportation (UDOT) will develop, validate, and deploy a method by which risk and resilience findings can be incorporated into existing management programs including statewide, corridor, and project planning (Ongoing FHWA Pilot Project). The pilot will develop a standard method by which quantitative risk data can be normalized across various facility types to allow for informed decision making. In addition, thresholds for risk metrics will be established to allow engineers and planners to identify those areas of high risk as compared to similar facilities across the state system. Ultimately, UDOT intends to incorporate risk and resilience assessment as an additional focus of their overall investment strategy similar to safety, mobility, and preservation.

SUMMARY AND KNOWLEDGE GAPS

In this literature review, the North Carolina DOT drainage and hydraulic design was reviewed and compared against other design guidelines (regionally and nationally). The major components considered in the hydraulic design are peak runoff, annual exceedance probability (AEP), and rainfall intensity. These components are reflected in the selection of a ‘design’ event, the impacts of which are used to select the size and other details of a hydraulic structure. The review has shown that these design events have statistical uncertainties, which should be taken into account. These uncertainties, and the probabilistic implications of the uncertainties, become more pronounced with more extreme events (i.e., lower AEP). Much of the current literature also highlights how climate change adds further uncertainty because it implies a non-stationary effect. Consequently, several cited studies have recommended using the most up-to-date precipitation data and projections to properly identify design intensity. The review also found that FHWA has produced a manual, which explicitly recommends incorporating potential effects of extreme events and climate change because it was established that this approach will enhance the life cycle benefits. In a related effort, NCHRP has sponsored several research projects that have produced guides and comprehensive frameworks for considering and incorporating climate change into the design processes for inland and coastal applications.

The literature review found that NCDOT currently follows FHWA guidelines for hydraulic design. The current practice of NCDOT was described in this literature review and it was compared with other states’ practices. The methods for calculating peak storm discharges and design frequencies were compared between NCDOT practice and other states. The important component that is missing from the design guidelines is that they do not use the most up-to-date design events and there are no dynamic guidelines, i.e., in the case of failure due to extreme events, it is not established that how the design should be improved for future events.

The review of the literature showed that infrastructure resilience is becoming an increasingly critical issue for many agencies and organizations. The literature also demonstrates that resilient infrastructure requires a complex and integrated framework of engineering and institutional management, policy, and decision making. It also requires changing the design standards to be more adaptive to impacts of extreme events, conducting vulnerability assessments to identify locations with highest priority to apply necessary changes, and determining the return on investment when an option or multiple options are considered in decision making. These actions can be interconnected and/or be a component of a resilience framework.

While the leading edge of research has not yet produced specific guidance on how to ensure that infrastructure is resilient to extreme events, it has produced frameworks, strategies, and examples to follow towards that goal. Overwhelmingly, these methods begin with collecting and analyzing performance data in order to develop quantified analyses to support further decision making and improvements in policy and practice. Since the codes and standards as well as policy and socioeconomic conditions vary greatly from one state to the next, this data collection must be done by each agency within their own jurisdictions in order to provide accurate and meaningful insights. In addition, the detailed literature review has confirmed that there does not currently exist national guidance to identify when certain repairs, designs, strategies or other approaches are efficient enough to make a system more resilient in long term. Usually, the last step of any framework is to monitor the applied strategy or design, but there is not enough study to provide long term evaluation of the recommended strategy, repair, or design. In another words, the guidelines are in the stage of “what should be done” or “what has been done,” further research studies are required to reach the stage of “what has been done and how the system performed after certain events.”

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Culvert Identification and Nomenclature

CMP= Corrugated Metal Pipe

- CSP= Corrugated Steel Pipe
- CAP= Corrugated Aluminum Pipe
- CAAP= Corrugated Aluminum Alloy Pipe
- CMPA= Corrugated Metal Pipe Arch
  - CSPA= Corrugated Steel Pipe Arch
  - CAPA= Corrugated Pipe Arch

CMSPPA= Corrugated Metal Structural Plate Pipe Arch

RCPA= Reinforced Concrete Pipe Arch

RCBC= Reinforced Concrete Box Culvert

ABC= Aluminum Box Culvert

HW= Head Wall

APPENDIX B: SUPPLEMENTARY INFORMATION ON VULNERABILITY ASSESSMENT

Mapping Hurricane Intensity

The daily Quantitative Precipitation Estimates (QPE) were extracted from the National Weather Services (NWS) NOAA for the period of October 6 – 10, 2016 (Figure B.1) and September 13 – 19, 2018 (Figure B.2) for Hurricane Matthew and Florence, respectively. The observations showed that the periods in which these two events happened inside the boundaries of North Carolina were October 8 – 9, 2016 and September 14 – 18, 2018 for Hurricane Matthew and Florence, respectively. The shapefile for the daily precipitation for selected dates were obtained from NWS NOAA in the format of NetCDF and imported in ArcGIS Desktop (Figure B.3(a) – (b) and Figure B.4(a) – (e)). The raster data for the duration of each event were cumulated in ArcGIS using Raster Calculator (Spatial Analyst) tool and a new raster data was created for each event (Figure B.3(c) and Figure B.4(f)).

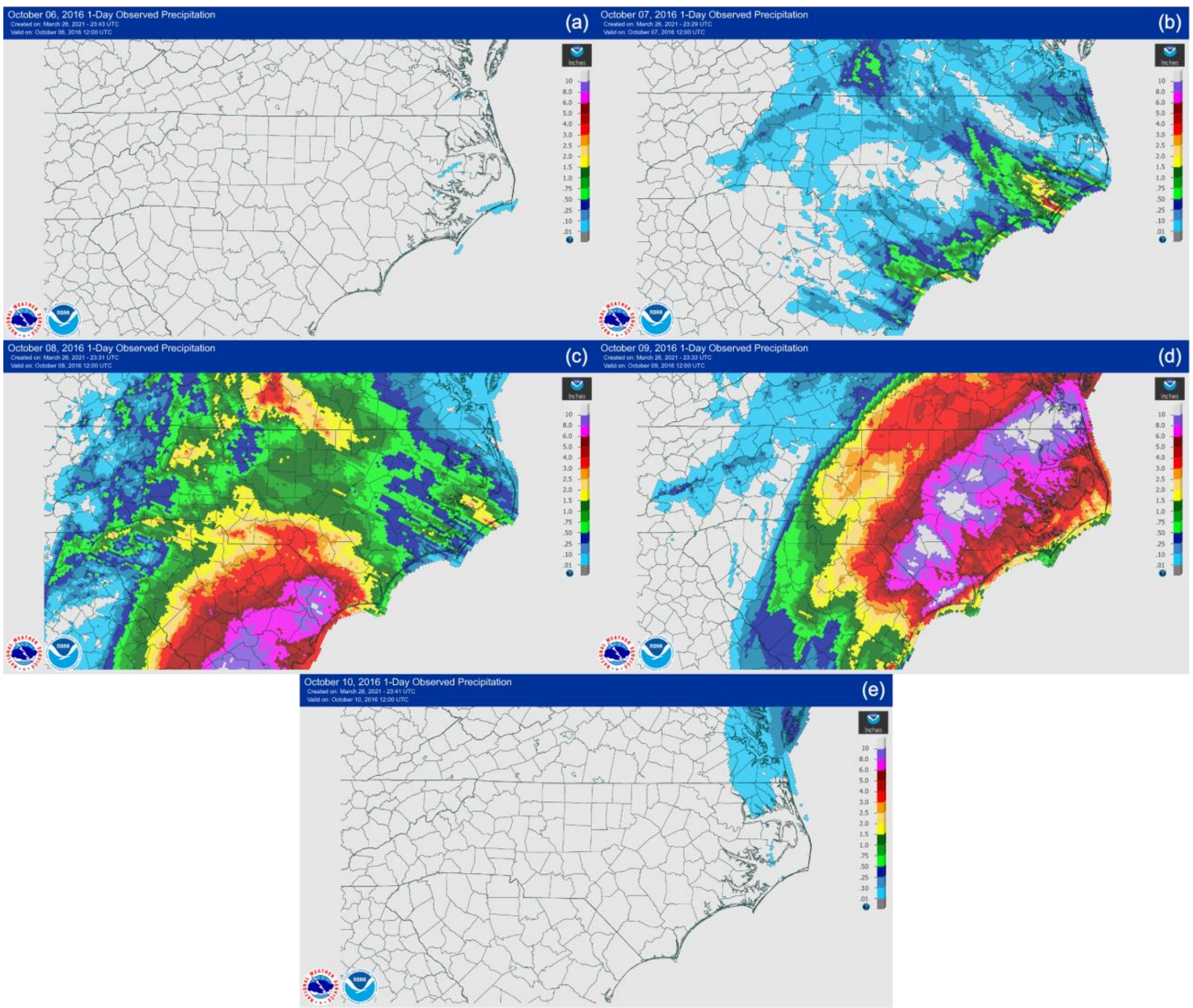


Figure B.1. Daily observed precipitation for: (a) October 6, 2016, (b) October 7, 2016, (c) October 8, 2016, (d) October 9, 2016, and (e) October 10, 2016.

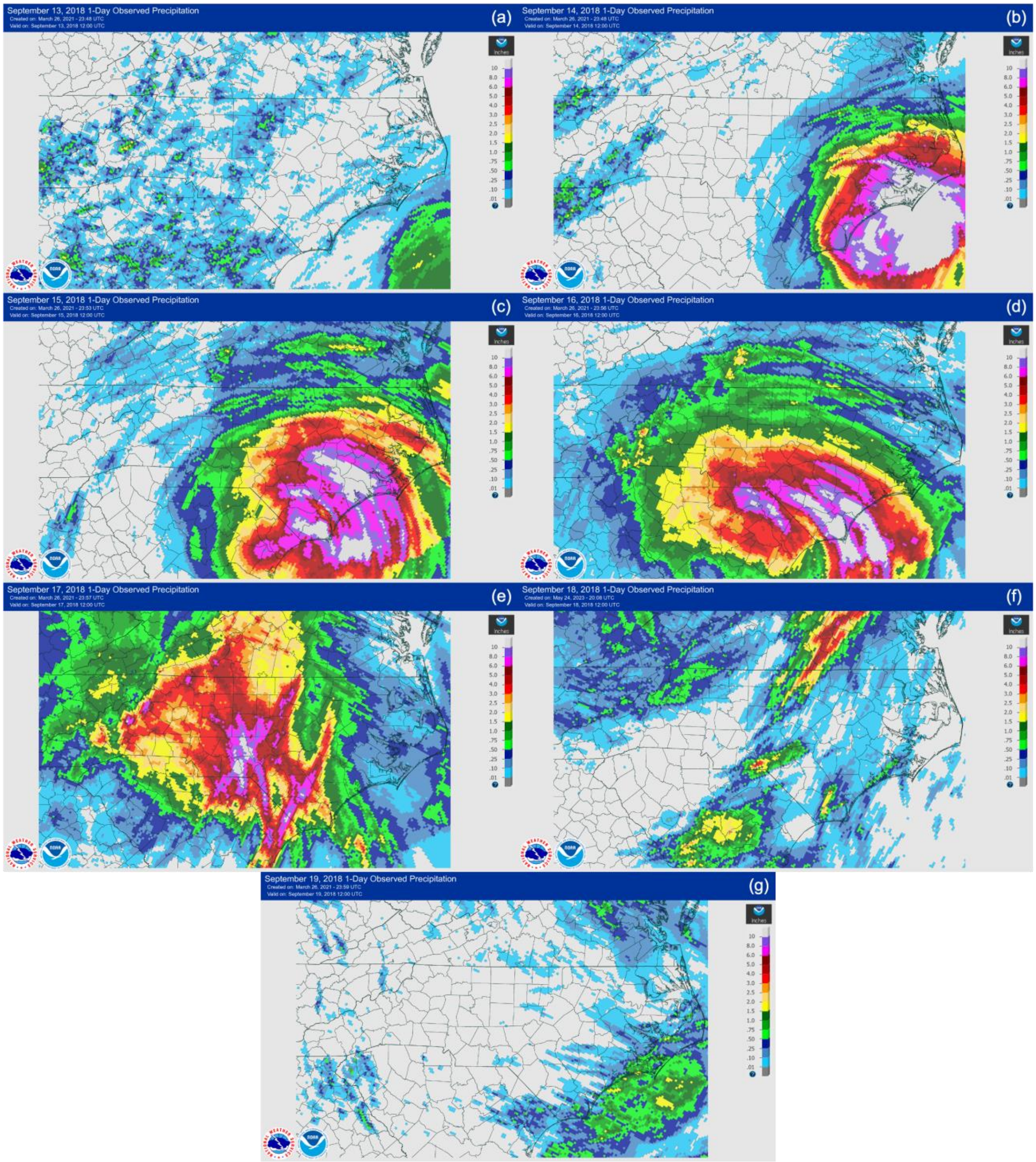


Figure B.2. Daily observed precipitation for: (a) September 13, 2018, (b) September 14, 2018, (c) September 15, 2018, (d) September 16, 2018, (e) September 17, 2018, (f) September 18, 2018, and (g) September 19, 2018.

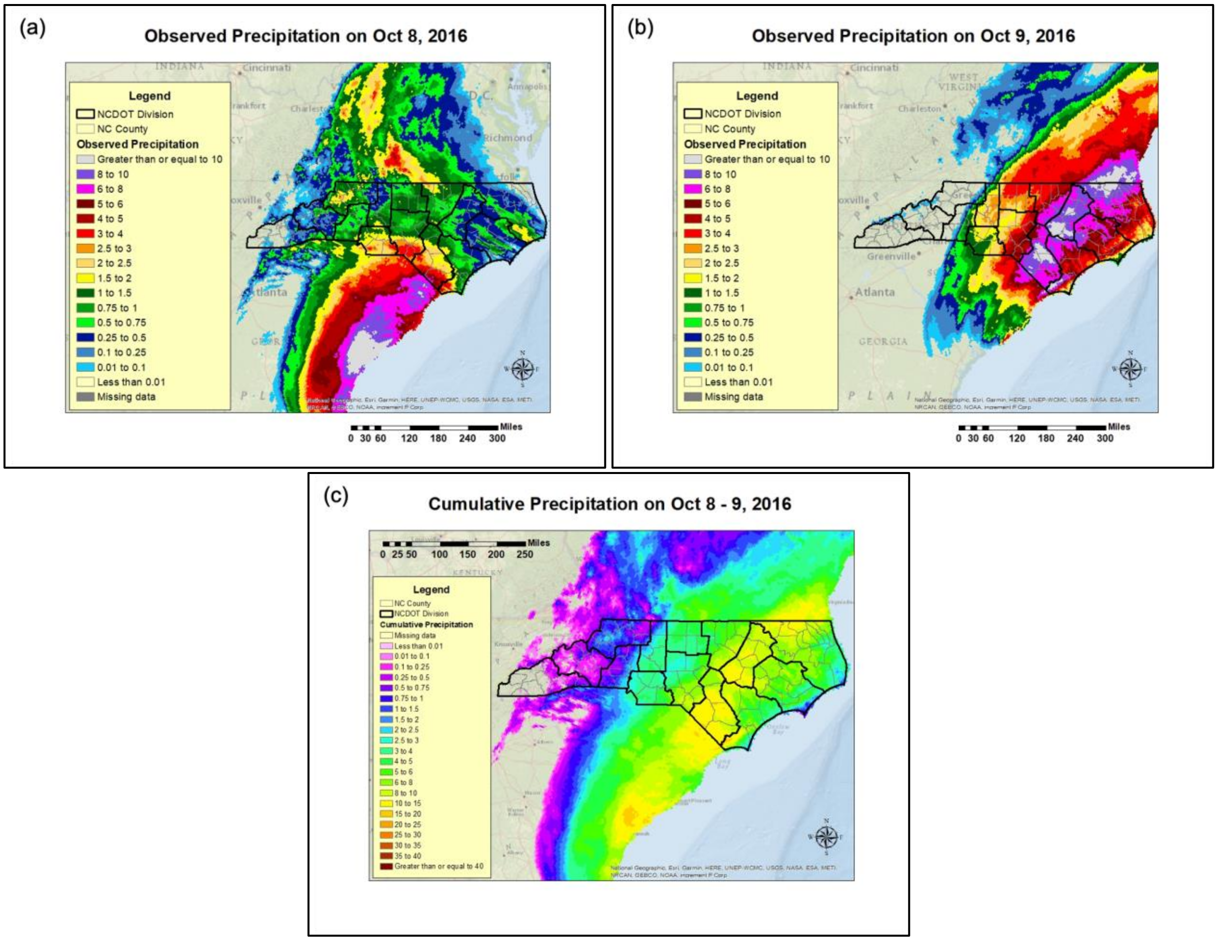
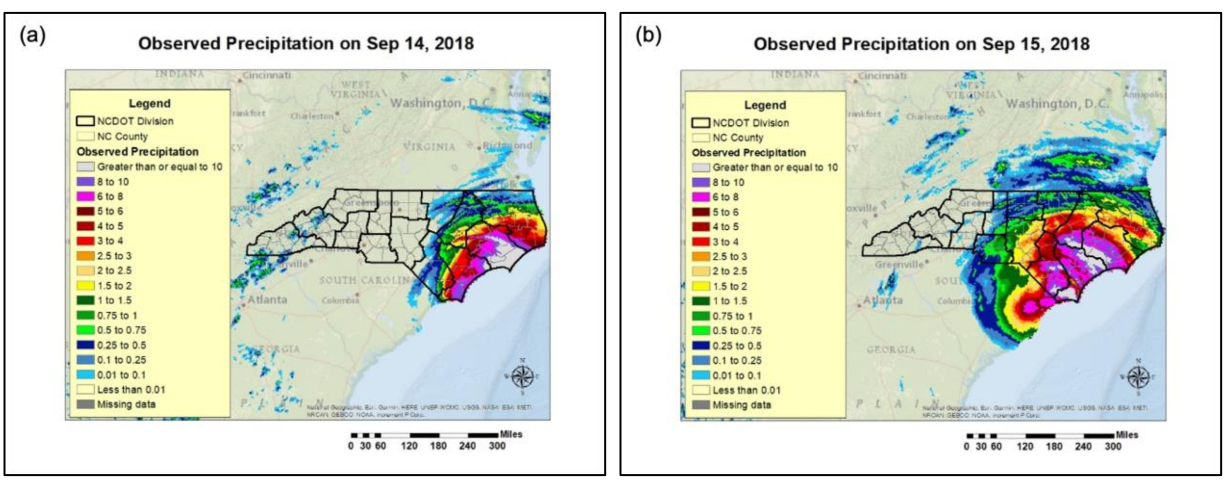


Figure B.3. Rainfall intensity maps during Hurricane Matthew: (a) daily precipitation on Oct 8, 2016, (b) daily precipitation on Oct 9, 2016, and (c) cumulative precipitation for duration of Hurricane Matthew (i.e., Oct 8 – 9, 2016).



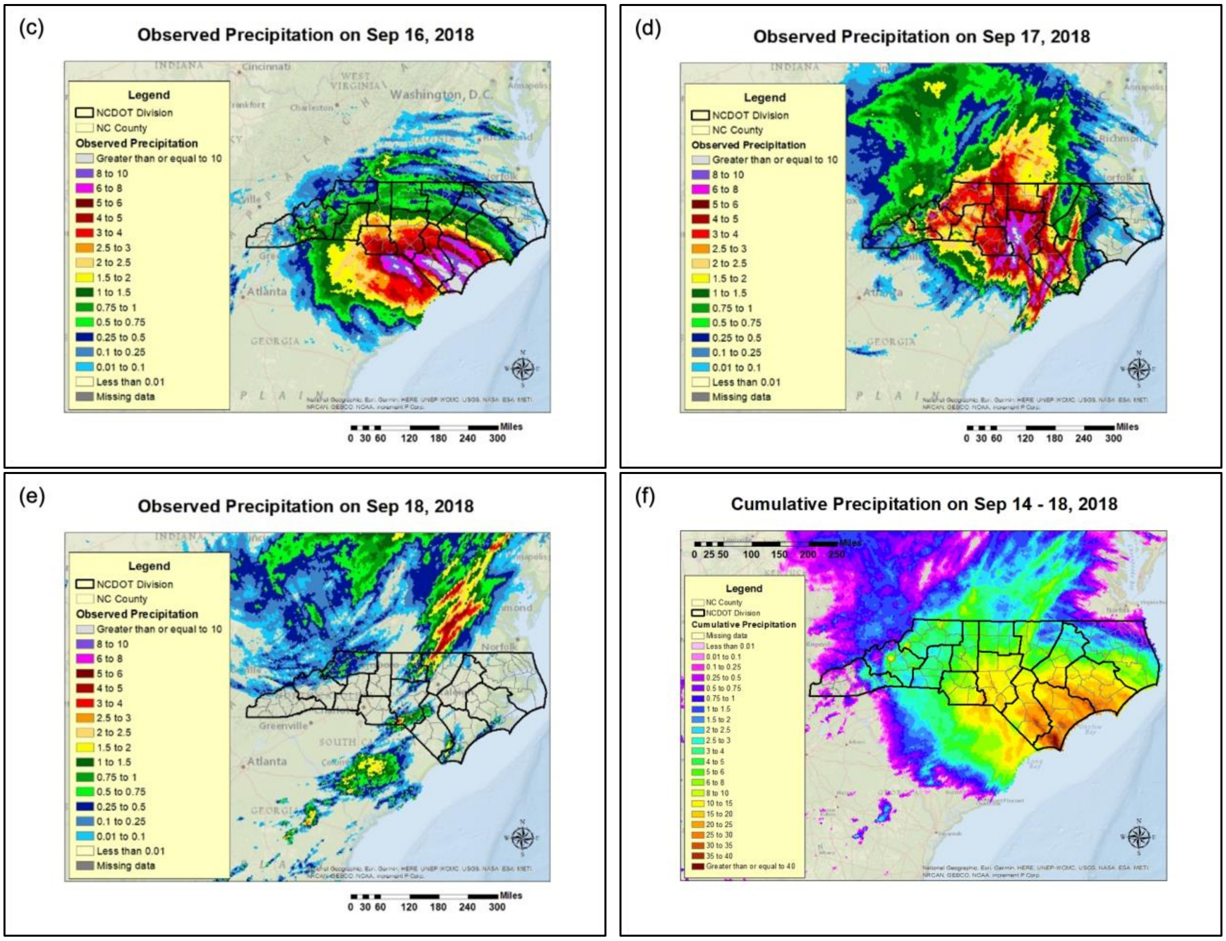


Figure B.4. Rainfall intensity maps during Hurricane Florence: (a) daily precipitation on Sep 14, 2018, (b) daily precipitation on Sep 15, 2018, (c) daily precipitation on Sep 16, 2018, (d) daily precipitation on Sep 17, 2018, (e) daily precipitation on Sep 18, 2018, and (f) cumulative precipitation for duration of Hurricane Florence (i.e., Sep 14 – 18, 2018).

Mapping Damaged Locations

The information and GIS layers for damaged pipes after Hurricane Matthew and Florence were obtained from NCDOT project RP 2021-03,Figure B.5(a). Figure B.5(a) shows that 671 and 449 pipes were damaged after Hurricane Matthew and Florence, respectively. Additional information and GIS layers for damaged locations after Hurricane Florence were obtained from NCDOT Survey 123,Figure B.5(b). Figure B.5(b) shows that 3,727 locations were damaged in pipe, roadway, and bridge area. The focus of this analysis is on locations with damaged pipes, so the locations with damaged pipes from Survey 123 were mapped, Figure B.5(c). Figure B.5(c) indicates that 713 pipes were damaged after Hurricane Florence based on Survey 123. Locations with damaged pipes after Hurricane Florence were combined from NCDOT RP 2021-03 and Survey 123, Figure B.5(d), and duplicated locations were eliminated from GIS layer. As shown in Figure B.5(d), 915 pipes were damaged after Hurricane Florence.

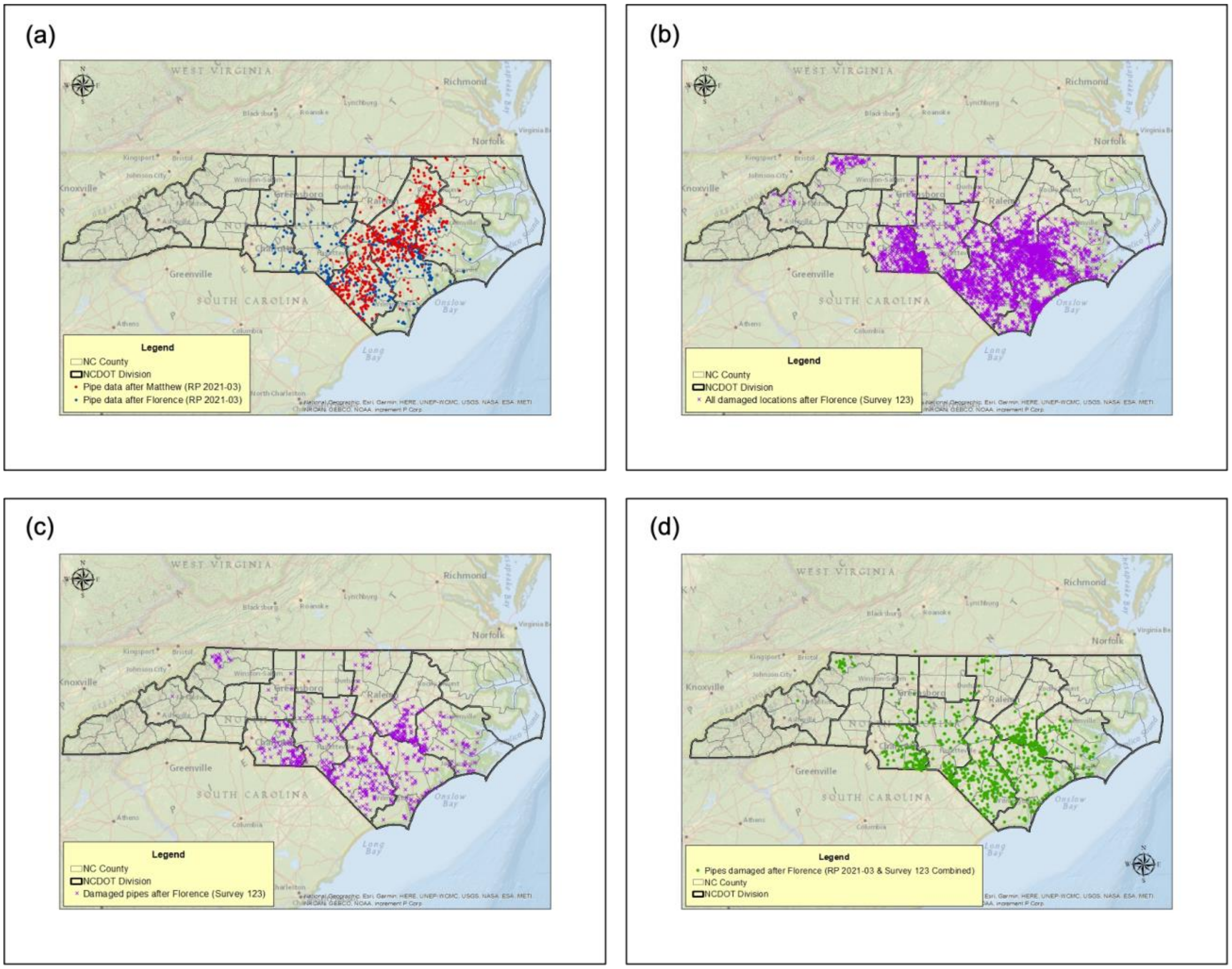


Figure B.5. Map of damaged locations due to Hurricane Matthew and Florence: (a) damaged pipes after Hurricane Matthew and Florence based on NCDOT RP2021-03, (b) damaged locations after Hurricane Florence based on Survey 123, (c) damaged pipes after Hurricane Florence based on Survey 123, and (d) damaged pipes after Hurricane Florence based on Survey 123 and NCDOT RP2021-03 combined.

Mapping Overlapped Locations

As explained in Section 2.2.2, damaged pipe locations were plotted in ArcGIS and using the Buffer (Analysis) tool in ArcGIS a buffer of 500 meters were assigned to the points to account for possible mismatch in GPS coordinates for the same location. Then, using the Intersect (Analysis) tool in ArcGIS, the overlapped locations, i.e., locations in which pipes were damaged after Hurricane Matthew where also damaged after Hurricane Florence within 500 meters. These overlapped locations for each county are presented in Figure B.6(a – h).


(a)
JOHNSTON
Legend
NC Counties
Johnston County
Damaged after Matthew and Florence
(b)
ROBESON
Legend
NC Counties
Robeson County
Damaged after Matthew and Florence
(c)
BLADEN
Legend
NC Counties
Bladen County
Damaged after Matthew and Florence
(d)
WAYNE
Legend
NC Counties
Wayne County
Damaged after Matthew and Florence
(e)
HARNETT
Legend
NC Counties
Harnett County
Damaged after Matthew and Florence
(f)
CUMBERLAND
Legend
NC Counties
Cumberland County
Damaged after Matthew and Florence

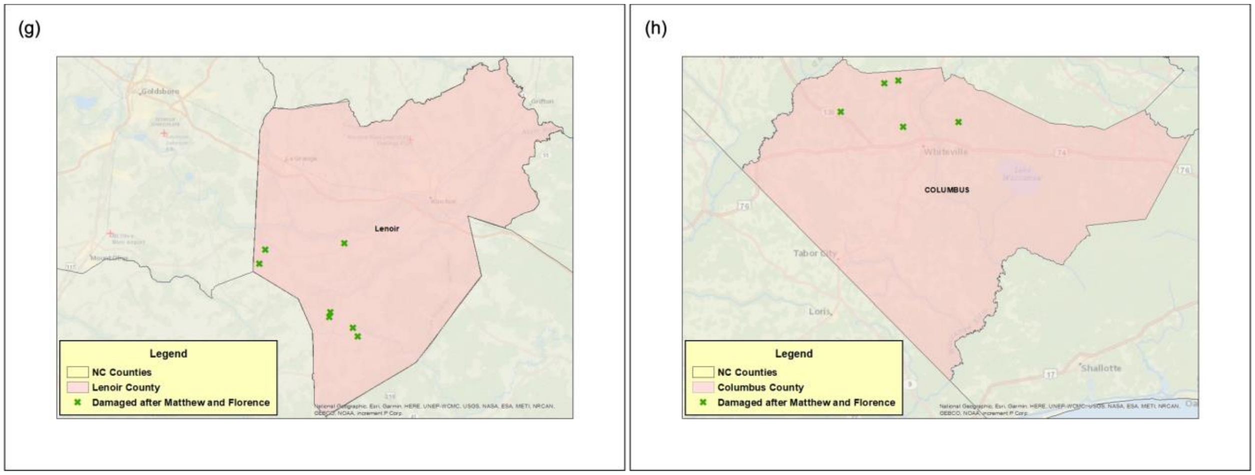


Figure B.6. Locations damaged after both Hurricane Matthew and Florence in selected counties: (a) Johnston County, (b) Robeson County, (c) Bladen County, (d) Wayne County, (e) Harnett County, (f) Cumberland County, (g) Lenoir County, and (h) Columbus County.

Selection of Cases Damaged in Only One Event

In the preliminary case study evaluation, cases that were damaged in only one event in Robeson County was identified. The approach to identify the cases explained below set the stage for other sites identified in other counties. In Robeson County, the case study sites were identified based on similarities or differences in the following parameters:

1- Repair/Cost categories:
   - <$10k (Low cost)
   - $10-50k (Moderate cost 1)
   - $50k-100k (Moderate cost 2)
   - >$100k (High cost)
2- Level of precipitation after Hurricane Matthew and Florence:
   - 6 to 8 in. (referred to as Category 1)
   - 8 to 10 in. (referred to as Category 2)
   - 10 to 15 in. (referred to as Category 3)
   - 15 to 20 in. (referred to as Category 4)
   - 20 to 25 in. (referred to as Category 5)

The maps shown in Figure B.7 and Figure B.8 represents the location of sites that were only damaged after Hurricane Matthew in Robeson County. The labels assigned to each case consists of county code, the location number, M for Matthew or F for Florence (e.g., 77-1-M, 77 is Robeson County code, 1 is location number, M is for Hurricane Matthew). Figure B.7 shows the damaged locations overlaid with a map of precipitation from Hurricane Matthew and Figure B.8 shows the damaged locations overlaid with a map of precipitation from Hurricane Florence. The maps shown in Figure B.9 and Figure B.10 represents the location of case studies that were only damaged after Hurricane Florence in Robeson County. Figure B.9 shows the damaged locations overlaid with a map of precipitation from Hurricane Matthew and Figure B.10 shows the damaged locations overlaid with a map of precipitation from Hurricane Florence. The number associated with these locations designates the type of case study, which is explained in detail below.

The case studies presented in Figure B.7, and Figure B.8 that were only damaged after Matthew are interesting to investigate in comparison with each other as well as stand-alone cases because they help explain some specific situations.

- Case 77-1-M required high-cost category repairs after Matthew, and it was not damaged after Florence (with high level of precipitation) which might be considered as a successful repair.
- Cases 77-2-M and 77-3-M required low-cost category repairs after Matthew, and they were not damaged after Florence (with high precipitation).
  - Comparing case 77-1-M, 77-2-M and 77-3-M might answer the following question: "why both locations were not damaged after Florence considering they were under similar conditions in terms of precipitation intensity?"
- Cases 77-4-M and 77-5-M required low-cost category repairs after Matthew, and they were not damaged after Florence.
- Case 77-6-M and 77-7-M required moderate-cost category repairs after Matthew, and it was not damaged after Florence.
- Cases 77-8-M, 77-9-M, 77-10-M, and 77-11-M required moderate-cost category repairs after Matthew, and they were not damaged after Florence.
  - Comparing cases 77-4-M and 77-5-M – 77-8-M, 77-9-M, 77-10-M, and 77-11-M might explain why these cases that were under the same level of precipitations and received different types of repairs, performed successfully after Florence.
- Case 77-12-M required high-cost category repairs after Matthew, and it was not damaged after Florence.
  - Comparing cases 77-8-M, 77-9-M, 77-10-M, and 77-11-M and 77-12-M might explain the following statement: Case (7) was under lower precipitation in Matthew comparing to case (6), but it was decided to do the same level of repairs on both locations and both locations performed successfully after Florence.
- Cases 77-13-M and 77-14-M was under relatively high precipitation after Matthew, and it only needed low-cost category repairs. What was the specification of this successful design that after Matthew only low-cost category repairs were required and it was not damaged at all after Florence?
- Cases 77-15-M and 77-16-M represent cases that under the same level of precipitation in Matthew and Florence it was not damaged after Florence, but it was damaged after Matthew, and it required high-cost category repairs. What is the reason? What changed between two events?

The case studies presented in Figure B.9, and Figure B.10 that were only damaged after Matthew are interesting to investigate in comparison with each other as well as stand-alone cases because they help explain some specific situations.

- Comparing cases 77-1-F and 77-2-F show cases that were not damaged under the same levels of precipitation after Matthew, but they were damaged after Florence under similar level of precipitation (both in Matthew and Florence) and different type of repairs were required (low-cost and high-cost category repairs).

- Comparing cases 77-3-F and 77-4-F show cases that were not damaged under same level of precipitation after Matthew, but they were damaged after Florence under similar level of precipitation and different type of repairs were required (moderate-cost and high-cost category repairs)
- Cases 77-5-F and 77-6-F represent cases that were not damaged after Matthew under considerable level of precipitation, and after a relatively high precipitation in Florence only low-cost category repair was required. What was the design that performed so successfully?
- Case 77-7-F represents a case that under the same level of precipitation in Matthew and Florence it was not damaged after Matthew, but it was damaged after Florence, and it required moderate-cost category repairs. What is the reason? What changed between two events?
- Cases 77-8-F and 77-9-F represent cases that under the same level of precipitation in Matthew and Florence it was not damaged after Matthew, but it was damaged after Florence, and it required high-cost category repairs. What is the reason? What changed between two events?

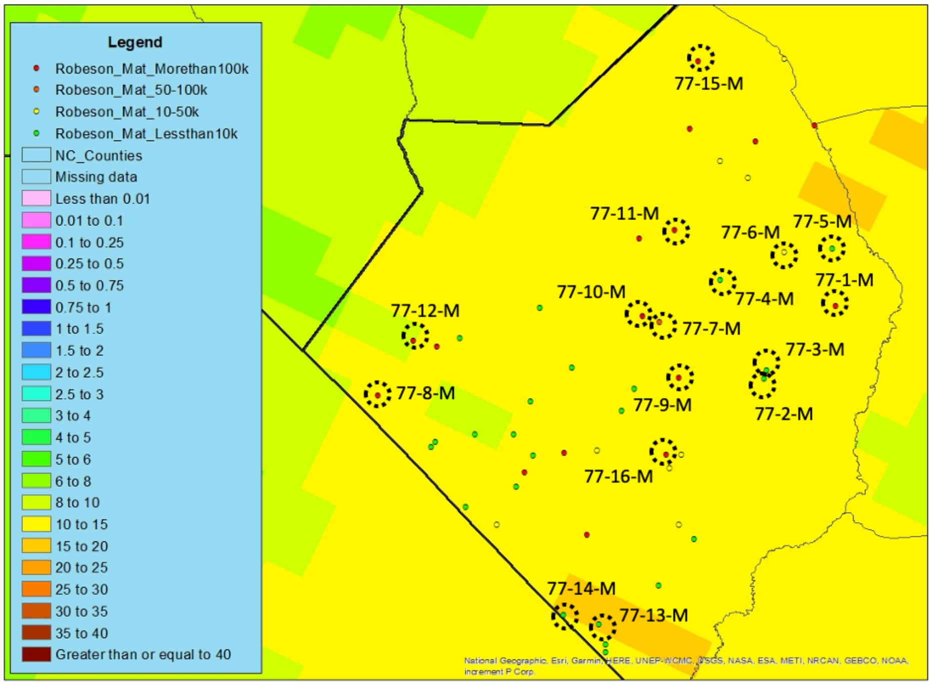


Figure B.7. Damaged locations after Hurricane Matthew overlaid on Hurricane Matthew precipitation map.

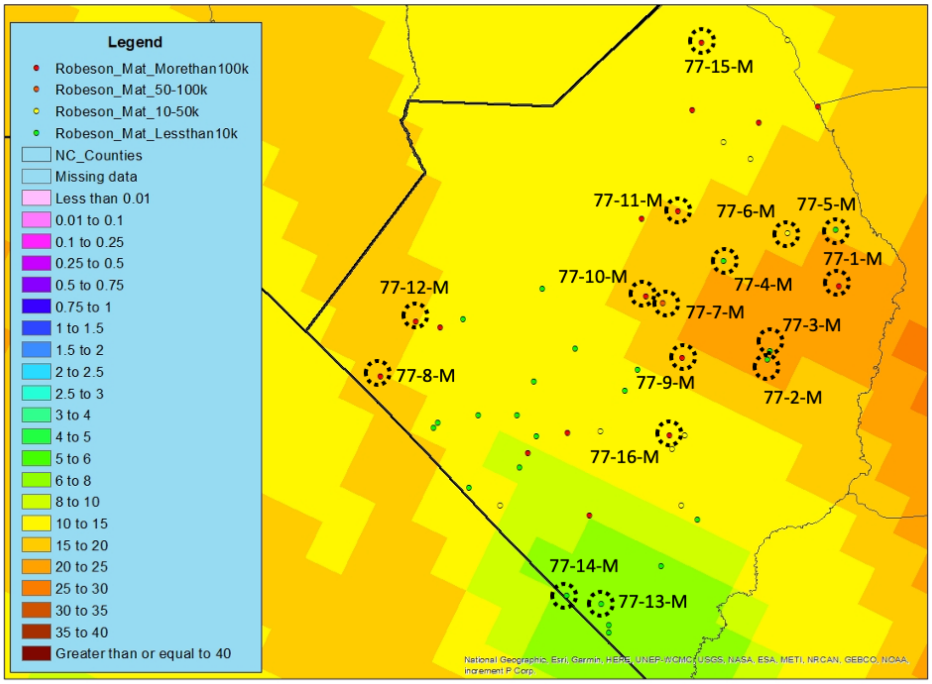


Figure B.8. Damaged locations after Hurricane Matthew overlaid on Hurricane Florence precipitation map.

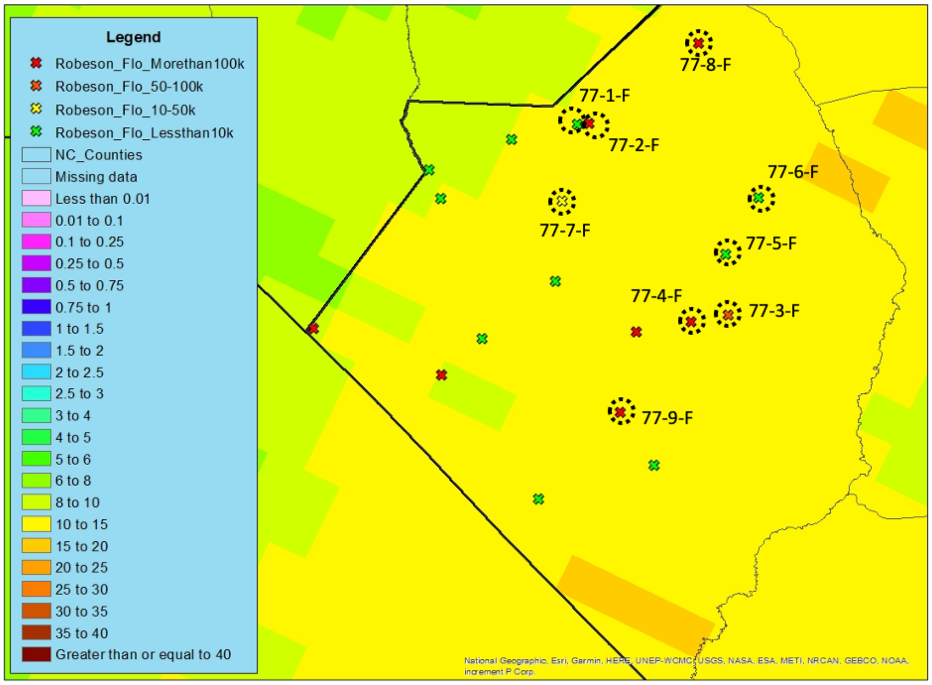


Figure B.9. Damaged locations after Hurricane Florence overlaid on Hurricane Matthew precipitation map.

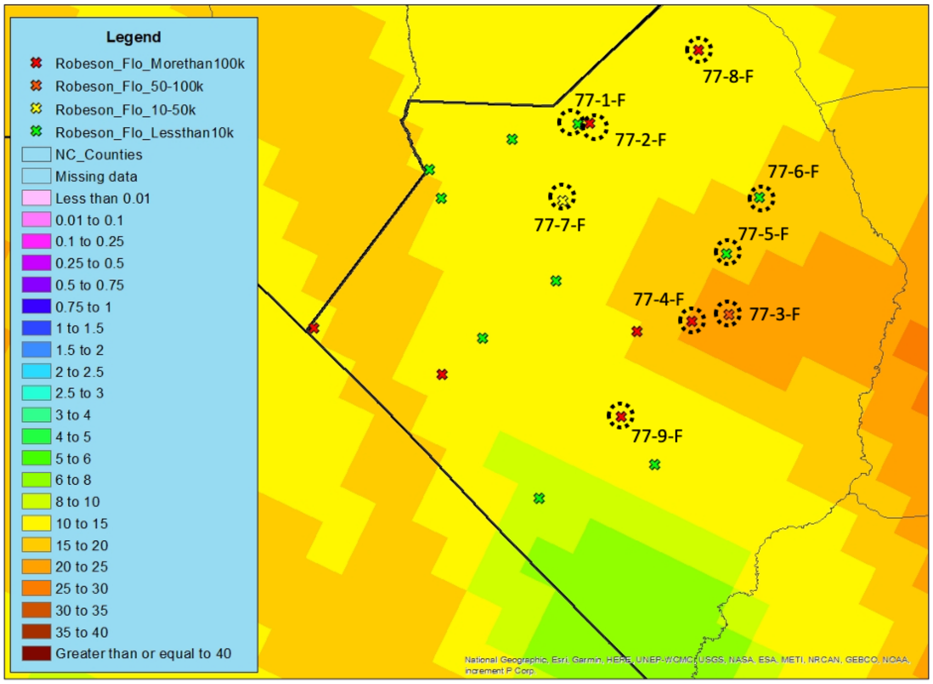


Figure B.10. Damaged locations after Hurricane Florence overlaid on Hurricane Florence precipitation map.

APPENDIX C: SELECTED SITES AND FIELD VISITS

Sites

Based on vulnerability assessments 138 sites were selected for this research study. The information on these sites is summarized in Table C.1.

Table C.1. List of all selected sites.

Site Number	County	Road Name	Lat	Long	Visited?
51-1-MF	Johnston	Newton Grove Road	35.29	-78.32	✓
51-2-MF	Johnston	Woodard Road	35.41	-78.21	✓
51-3-MF	Johnston	Woods Crossroads Road	35.33	-78.49	✓
51-4-MF	Johnston	Massey Holt Road	35.43	-78.16	✓
77-1-MF	Robeson	Smith Mill Road	34.69	-78.86	✓
77-2-MF	Robeson	Dallas Road	34.57	-79.10	✓
77-3-MF	Robeson	Dogwood Road	34.42	-79.06	✓
77-4-MF	Robeson	Ashpole Church Road	34.55	-79.32	✓
77-5-MF	Robeson	Kitchen St Road	34.56	-79.35	✓
77-6-MF	Robeson	Persimmon Road	34.58	-79.35	✓
77-7-MF	Robeson	Fairley Road	34.60	-79.39	✓
77-1-M	Robeson	Howell Road	34.67	-78.88	×
77-2-M	Robeson	Snake Road	34.60	-78.96	✓
77-3-M	Robeson	Cedar Grove Road	34.61	-78.95	✓
77-4-M	Robeson	Mt. Moriah Church Road	34.69	-79.00	✓
77-5-M	Robeson	Turnpike Road	34.72	-78.88	✓
77-6-M	Robeson	Vester Road	34.72	-78.94	×
77-7-M	Robeson	Pinelog Road	34.66	-79.07	×
77-8-M	Robeson	McCrimmon Road	34.59	-79.38	×
77-9-M	Robeson	K B Road	34.61	-79.05	×
77-10-M	Robeson	Oakgrove Church Road	34.66	-79.09	✓
77-11-M	Robeson	McDuffie Crossing Road	34.74	-79.05	×
77-12-M	Robeson	Midway Road	34.64	-79.34	×
77-13-M	Robeson	Marietta Road	34.38	-79.14	✓
77-14-M	Robeson	Cowpen Swamp Road	34.39	-79.18	✓
77-15-M	Robeson	Carolina Church Road	34.90	-79.02	✓
77-16-M	Robeson	Centerville Church Road	34.55	-79.10	✓
77-1-F	Robeson	Evon Road	34.82	-79.16	✓
77-2-F	Robeson	Pearsall Road	34.82	-79.15	✓
77-3-F	Robeson	Fayetteville Road	34.65	-79.00	×
77-4-F	Robeson	Sanderson Road	34.64	-79.04	×
77-5-F	Robeson	W Powersville Road	34.70	-79.00	✓
77-6-F	Robeson	Townsend Road	34.75	-78.97	✓
77-7-F	Robeson	John French Road	34.75	-79.18	×
77-8-F	Robeson	McIver Road	34.55	-79.10	✓
77-9-F	Robeson	Pleasant Hope Road	34.90	-79.02	✓
95-1-MF	Wayne	Polly Watson Road	35.59	-78.04	✓
95-2-MF	Wayne	Mark Herring Road	35.19	-77.87	✓
95-3-MF	Wayne	Stevens Mill Road	35.34	-78.15	✓
95-4-MF	Wayne	NC 55	35.19	-77.90	✓
95-5-MF	Wayne	Corbett Hill Road	35.27	-78.22	✓
95-6-MF	Wayne	Mark Herring Road	35.18	-77.87	✓
95-1-M	Wayne	Nahunta Road	35.51	-78.00	✓
95-2-M	Wayne	Big Daddys Road	35.49	-77.90	✓
95-3-M	Wayne	Wayne Memorial Drive	35.46	-77.84	✓
95-1-F	Wayne	Hooks Road	35.52	-78.04	✓

Site Number	County	Road Name	Lat	Long	Visited?
95-2-F	Wayne	North Washington Street	35.51	-77.99	✓
95-3-F	Wayne	Pinkney Road	35.52	-78.09	✓
53-1-MF	Lenoir	Gray Branch Church Road	35.12	-77.72	✓
53-2-MF	Lenoir	Davis Mill Road	35.11	-77.69	✓
53-3-MF	Lenoir	N Croom Bland Road	35.21	-77.70	✓
53-4-MF	Lenoir	Dalys Chapel Road	35.19	-77.82	✓
53-5-MF	Lenoir	Eric Sparrow Road	35.13	-77.72	✓
53-6-MF	Lenoir	NC 903	35.20	-77.82	✓
42-1-MF	Harnett	Hodges Chapel Road	35.36	-78.57	✓
25-1-MF	Cumberland	Tabor Church Road	34.88	-78.79	✓
8-1-MF	Bladen	Twisted Hickory Road	34.54	-78.69	✓
8-2-MF	Bladen	Sweet Home Church Road	34.70	-78.57	✓
8-3-MF	Bladen	Brown Creek Church Road	34.63	-78.67	✓
23-1-MF	Columbus	Union Valley Road	34.37	-78.75	✓
23-2-MF	Columbus	Sikes Road	34.38	-78.63	✓
23-3-MF	Columbus	Old US 74	34.40	-78.87	✓
23-4-MF	Columbus	Greens Mill Road	34.44	-78.78	✓
23-5-MF	Columbus	Jordan Road	34.45	-78.75	✓
95-4-M	Wayne	James Hinson Road	35.33	-78.12	✓
95-5-M	Wayne	Sheridan Forest Rd (Old 111 Hwy)	35.32	-77.94	✓
95-6-M	Wayne	NC 55	35.20	-78.01	✓
95-7-M	Wayne	Overman Road	35.31	-78.10	✓
95-8-M	Wayne	Raynor Mill Road	35.28	-78.21	✓
95-9-M	Wayne	North Center Street	35.24	-78.04	✓
95-4-F	Wayne	US 117	35.28	-78.05	✓
95-5-F	Wayne	Old Harvey Sutton Road	35.23	-78.15	✓
95-6-F	Wayne	Spring Bank Road	35.30	-77.92	✓
95-7-F	Wayne	Mark Herring Road	35.21	-77.86	✓
53-1-M	Lenoir	Dunn Road	35.28	-77.54	×
53-2-M	Lenoir	Tulls Mill Road	35.15	-77.72	×
53-3-M	Lenoir	NC 55	35.20	-77.79	✓
53-4-M	Lenoir	J Kenneth Hall Road	35.38	-77.56	×
53-5-M	Lenoir	Old Pink Hill Road	35.13	-77.73	✓
53-6-M	Lenoir	Falling Creek Road	35.29	-77.69	✓
53-1-F	Lenoir	Liddell Shortcut Road	35.18	-77.81	✓
53-2-F	Lenoir	W Pleasant Hill Road	35.07	-77.65	✓
53-3-F	Lenoir	Joe Murphy Road	35.02	-77.73	×
53-4-F	Lenoir	Tulls Mill Road	35.15	-77.72	×
53-5-F	Lenoir	Davis Mill Road	35.10	-77.67	×
42-1-M	Harnett	Thompson Road	35.33	-78.84	×
42-2-M	Harnett	Tilghman Road	35.39	-78.61	×
42-3-M	Harnett	Brick Mill Road	35.39	-78.71	✓
42-1-F	Harnett	Wire Road	35.30	-78.80	✓
42-2-F	Harnett	Carson Gregory Road	35.45	-78.67	×
25-1-M	Cumberland	Pleasant View Drive	35.05	-78.79	×
25-2-M	Cumberland	Yarborough Road	34.87	-78.86	✓
25-1-F	Cumberland	Johnson Road	34.89	-78.76	×
25-2-F	Cumberland	L A Dunham Road	35.02	-78.84	✓
8-1-M	Bladen	NC 211	34.46	-78.57	✓
8-2-M	Bladen	Britt Road	34.56	-78.65	×
8-3-M	Bladen	Old Hwy 41	34.65	-78.77	×
8-4-M	Bladen	Old Abbottsburg Road	34.52	-78.74	✓
8-1-F	Bladen	NC 210	34.71	-78.38	✓
8-2-F	Bladen	Everette Byrd Road	34.45	-78.56	✓

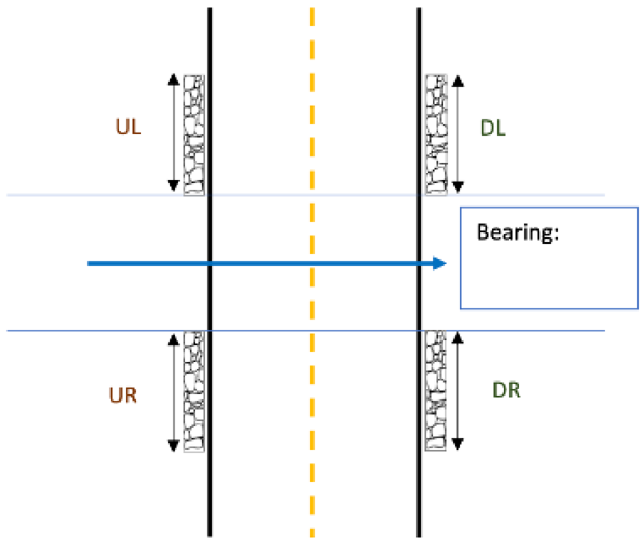
Site Number	County	Road Name	Lat	Long	Visited?
8-3-F	Bladen	Burney Road	34.79	-78.77	×
8-4-F	Bladen	Bivens Bridge Road	34.57	-78.40	×
8-5-F	Bladen	Coley Road	34.58	-78.55	×
8-6-F	Bladen	Lisbon Road	34.52	-78.56	✓
8-7-F	Bladen	Allen Priest Road	34.54	-78.50	×
23-1-M	Columbus	Norris Road	34.13	-78.84	×
23-2-M	Columbus	Bitmore Road	34.30	-78.71	×
23-3-M	Columbus	Kit Horne Road	34.31	-78.75	×
23-4-M	Columbus	Paul Willoughby Road	34.46	-78.89	×
23-5-M	Columbus	Mill Pond Road	34.30	-78.69	✓
23-1-F	Columbus	Old Northeast Road	34.38	-78.57	×
23-2-F	Columbus	Reaves Ferry Road	34.08	-78.62	×
23-3-F	Columbus	Jack Hayes Road	34.40	-78.75	×
23-4-F	Columbus	Peacock Road	34.17	-78.78	✓
95-8-F	Wayne	James Price Road	35.19	-77.87	✓
95-9-F	Wayne	South Jordan's Chapel Road	35.27	-78.26	✓
95-10-F	Wayne	NC 55	35.19	-77.90	×
95-11-F	Wayne	NC 581	35.55	-78.05	✓
95-12-F	Wayne	Westbrook Dairy Road	35.30	-78.28	×
53-7-M	Lenoir	NC 903	35.24	-77.82	×
53-8-M	Lenoir	NC 55	35.21	-77.81	×
53-9-M	Lenoir	Davis Mill Road	35.10	-77.69	✓
53-6-F	Lenoir	Hardy Mill Road	35.21	-77.80	×
53-7-F	Lenoir	NC 903	35.20	-77.82	×
42-4-M	Harnett	NC 55	35.30	-78.58	×
42-5-M	Harnett	Johnsonville School Road	35.31	-79.11	×
42-3-F	Harnett	NC 82	35.28	-78.67	×
25-3-M	Cumberland	NC 53	34.86	-78.73	×
25-4-M	Cumberland	River Road	35.18	-78.78	×
25-3-F	Cumberland	Wade Stedman Road	35.04	-78.69	×
25-4-F	Cumberland	Cypress Lakes Road	34.92	-78.88	×
8-5-M	Bladen	NC 53	34.74	-78.71	×
8-6-M	Bladen	Owen Hill Road	34.66	-78.68	×
8-7-M	Bladen	NC 242	34.68	-78.59	✓
8-8-F	Bladen	NC 41	34.65	-78.47	×
23-6-M	Columbus	Carl Mears Road	34.35	-78.98	×
23-5-F	Columbus	Narrow Gap Road	34.38	-78.26	×
23-6-F	Columbus	Golf Course Road	34.37	-78.74	×

Survey Form

Sample of survey form developed for site visits is presented in Figure C.1. A set of guidelines and instructions was provided for the purpose of data collection to be followed by research team and NCDOT summer interns in the site visits. These instructions are provided in this section.

(a)

Site Number ________________ Roadway Name _________________________ Date ________________



Determine Lengths (ft):

UL= Upstream Left Rip-Rap	__________
UR= Upstream Right Rip-Rap	__________
UH = Upstream Headwall	__________
UT = Upstream Total	__________
DL= Downstream Left Rip-Rap	__________
DR= Downstream Right Rip-Rap	__________
DH= Downstream Headwall	__________
DT = Downstream Total	__________

Checklist

- ☐ Fill out header information with site number, road name, and date.
- ☐ Set out cones (front of vehicle and front sides of culvert on each direction of travel at a minimum).
- ☐ Identify downstream and upstream sides of pipe and mark (if needed) using spray paint the right hand side and left hand sides of the site (see drawing).
- ☐ Use the compass to find the bearing of the channel and record the value on Page 1 of the data collection sheet. Reference recordings with respect to Magnetic North and write in degrees (i.e., 23° means 23° East of North).
- ☐ Take photographs using theodolite app showing the downstream channel and upstream channel.
- ☐ Take photographs using theodolite app showing overview of site looking along roadway. If needed take photographs from multiple directions.
- ☐ Take photographs using theodolite app showing inlets and outlets of culvert (as best as possible).
- ☐ Fill in remaining values and provide relevant notes on the items listed in Pages 1 and 2 of the data collection app.
- ☐ Take soil sample from the downstream side shoulder.
- ☐ Transfer data to the spreadsheet
- ☐ Rename photos and transfer to shared folder as described in the briefing document.

Page 1 of 2 RP2021-08, Data Collection form V.3

(b)

Site Number ____________ Roadway Name ______________________ Date _____________

GPS Coordinates at Site		
Pipe Type		
Pipe Size		
Number of Barrels		
Best Estimate Bed to Crown (ft)		
Est. Length of Pipe(s) (ft)		
Evidence of pavement overlay? (Y or N)		
Buried Pipe (Y,N, or Unknown)		
Channel Type (Trapezoidal, Rectangular, Triangular)	U	
	D	
Channel Bottom Width (if applicable)	U	
	D	
Channel Slope (if applicable) (H:V)	UR	
	UL	
	DR	
	DL	
Headwall (Y or N) and type (metal, concrete, etc.)	US	
	DS	
Condition of pipe(s) (new, damaged, evidence of joint separation, etc.)		
Flood Plain Description (wide, narrow, moderate, etc.)		
Vertical Geometry of Roadway (sag or flat)		
Shoulder/Embankment Description (Erosion? Scour holes present?, etc.)		
Other Notes		

Page 2 of 2 RP2021-08, Data Collection form V.3

Figure C.1. Survey form; (a) page 1 of form and (b) page 2 of form.

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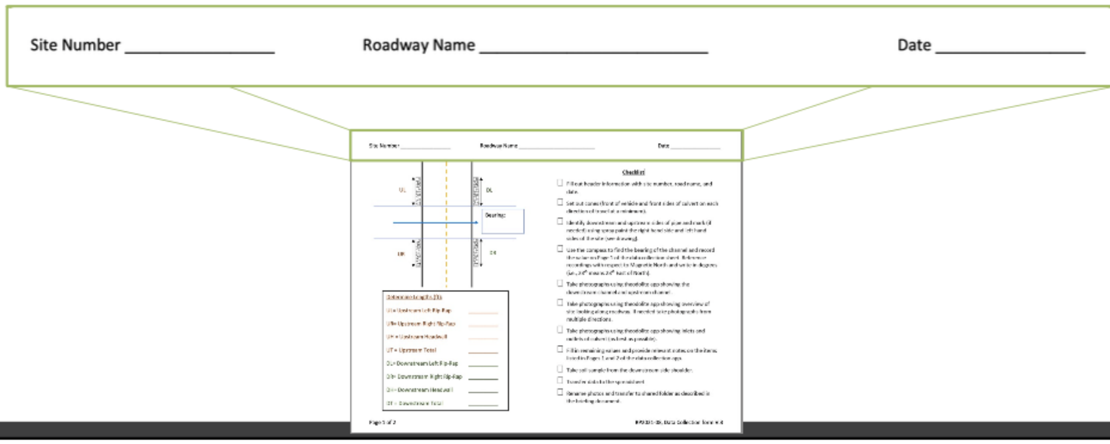
Instructions

1. Fill out header information with site number (County code - case ID, e.g., 078-3-M), road name, and date.
2. Set cones in front of vehicle and front of sides of culvert on each direction of travel at a minimum.



1

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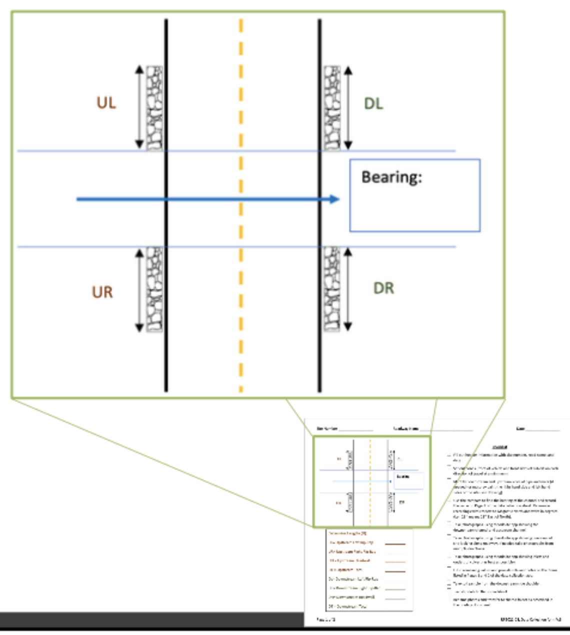
Instructions

3. Identify downstream and upstream sides of pipe. If the stream is not moving, based on elevation difference or evidence of scouring in channel or evidence of erosion on pipe, estimate the direction of stream. Using spray paint or chalk mark the downstream right (DR), downstream left (DL), upstream right (UR), upstream left (UL) of the site.
4. Looking downstream, use the compass to find the bearing of the channel. Record the value on Page 1 of the data collection sheet. Reference recordings with respect to Magnetic North and write in degrees (i.e., 23˚ means 23˚ East of North).



2

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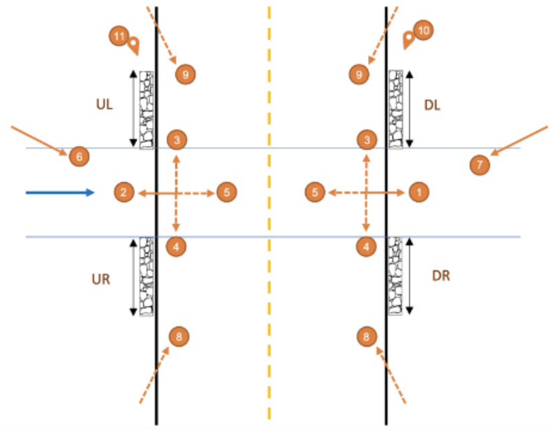
Instructions

5. Take photographs using Theodolite app showing:
   a. the downstream channel and upstream channel (No. 1 and 2)
   b. the pavement in multiple directions either on upstream or downstream (No. 3, 4, and 5)
   c. inlets and outlets of culvert (No. 6 and 7)
   d. overview of site looking along roadway (No. 8 and 9)
   e. any sign of damage or scour hole on the shoulder/embankment, if present (No. 10 and 11)



3

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Instructions

6. On the upstream side, using measuring wheel,
   a. measure rip-rap length on the left side (UL),
   b. measure rip-rap length on the right side (UR),
   c. measure the length of headwall (UH),
   d. measure total length from beginning of rip-rap on one side to the end of rip-rap on the other side (UT). Note that the total length needs to be measured since it might not be equal to the sum of UL, UR, and UH.
7. Same measurements on the downstream side.

Determine Lengths (ft):

UL= Upstream Left Rip-Rap ________

UR= Upstream Right Rip-Rap ________

UH = Upstream Headwall ________

UT = Upstream Total ________

DL= Downstream Left Rip-Rap ________

DR= Downstream Right Rip-Rap ________

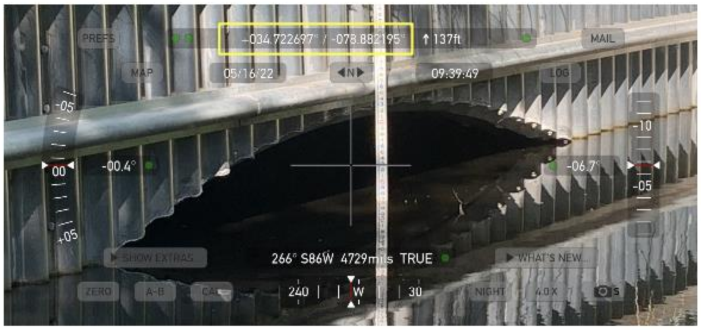
DH= Downstream Headwall ________

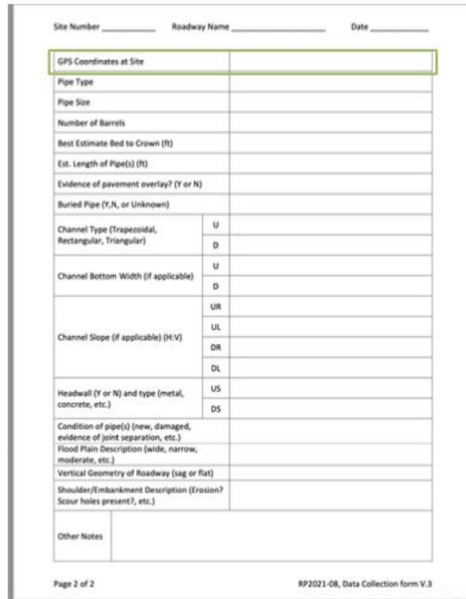
DT = Downstream Total ________

4

Instructions

8. Record GPS coordinates at site of the location as indicated in photos in Theodolite app.





Site Number ________ Roadway Name ______________ Date ________

GPS Coordinates at Site		
Pipe Type		
Pipe Size		
Number of Barrels		
Best Estimate Bed to Crown (ft)		
Est. Length of Pipe(s) (ft)		
Evidence of pavement overlay? (Y or N)		
Buried Pipe (Y,N, or Unknown)		
Channel Type (Trapezoidal, Rectangular, Triangular)	U	
	D	
Channel Bottom Width (if applicable)	U	
	D	
Channel Slope (if applicable) (H:V)	UR	
	UL	
	DR	
	DL	
Headwall (Y or N) and type (metal, concrete, etc.)	US	
	DS	
Condition of pipe(s) (new, damaged, evidence of joint separation, etc.)		
Flood Plain Description (wide, narrow, moderate, etc.)		
Vertical Geometry of Roadway (sag or flat)		
Shoulder/Embankment Description (Erosion? Scour holes present?, etc.)		
Other Notes		

Page 2 of 2 RP2021-08, Data Collection form V.3

5

5

Instructions

9. Identify type of pipe as indicated in photos below.


CMP


CMPA


RCP


RCPA


RCBC


ABC

6

6

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Instructions

10. Measure pipe(s) size using tape measure or leveling rod.
11. Determine number of barrels.
12. Measure bed to crown as follows:
    a. Place the leveling rod at the bottom of the channel (A) and level it.
    b. Read the height using handheld sight level.
    c. Place the leveling rod on the middle of roadway if possible. If not, place the leveling rod on the edge of pavement (B).
    d. Read the height using handheld sight level standing on the same location.
    e. Calculate the difference between two heights.

7

7

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Instructions

13. Measure length of pipe(s) using measuring wheel as follows:
    a. Start on top of the beginning of the pipe on one side (for example upstream).
    b. Move the measuring wheel towards the other side (for example downstream) on top of the end of pipe. If the beginning or end of the pipe is not accessible, measure that part using tape measure or estimate the length and add to the length measured with measuring wheel.
14. Determine whether the pavement on the location has been overlaid.
15. Determine whether the bottom of pipe is buried. If the bottom of the pipe is not visible, try poking the bottom of pipe with a stick or leveling rod and estimate whether it is buried or not.

8

8

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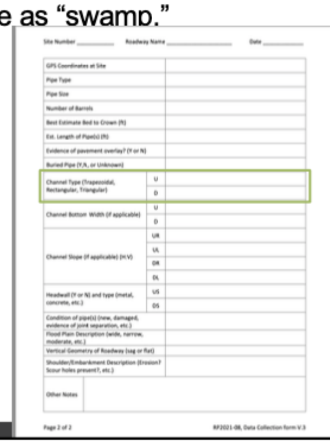
Instructions

16. Determine the shape of the channel on both upstream and downstream sides. Typical shapes of channels are trapezoidal, rectangular, and triangular. If the shape of the channel changes further away from the pipe, indicate the shape of the channel near the pipe and in the “Other notes” section explain the shape of channel is changed further away from the pipe and determine the shape. If the stream is connected to a swamp on any side, describe the channel shape on that side as “swamp.”



Site Number ______ Roadway Name ______ Date ______

GPS Coordinates at Site		
Pipe Type		
Pipe Size		
Number of Barrels		
Best Estimate Bed to Crown (ft)		
Est. Length of Pipe(s) (ft)		
Evidence of pavement overlay? (Y or N)		
Buried Pipe (Y,N, or Unknown)		
Channel Type (Trapezoidal, Rectangular, Triangular)	U	
	D	
Channel Bottom Width (if applicable)	U	
	D	
Channel Slope (if applicable) (H:V)	UR	
	UL	
	DR	
	DL	
Headwall (Y or N) and type (metal, concrete, etc.)	US	
	DS	
Condition of pipe(s) (new, damaged, evidence of joint separation, etc.)		
Flood Plain Description (wide, narrow, moderate, etc.)		
Vertical Geometry of Roadway (sag or flat)		
Shoulder/Embankment Description (Erosion? Scour holes present?, etc.)		
Other Notes		

Page 2 of 2 RP2021-08, Data Collection form V.3

9

9

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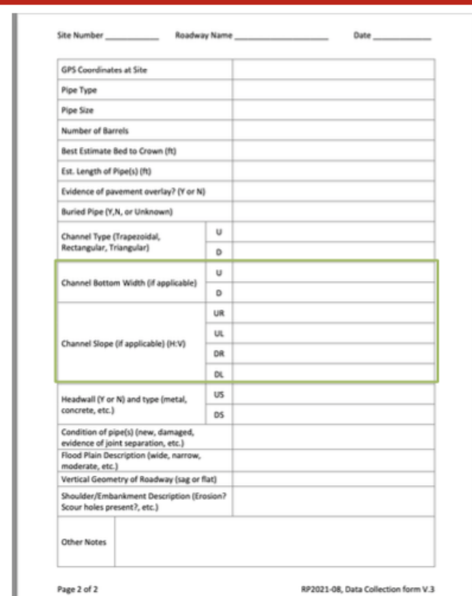
Instructions

17. Measure the channel bottom width on both upstream and downstream. If the channel is accessible use tape measure to determine the width of the channel bottom. If it is not accessible mark approximate location of sides of the channel on the shoulder or headwall and measure the width using tape measure or measuring wheel.
18. Measure the slope (V:H) of the channel on upstream right and left and downstream right and left as follows:
    a. Place the leveling rod on the bottom of the channel.
    b. Measure the vertical length from bottom to top of the channel.
    c. Measure the horizontal length from top of the channel to the leveling rod.
    d. Repeat this measurement for each side.



Site Number ______ Roadway Name ______ Date ______

GPS Coordinates at Site		
Pipe Type		
Pipe Size		
Number of Barrels		
Best Estimate Bed to Crown (ft)		
Est. Length of Pipe(s) (ft)		
Evidence of pavement overlay? (Y or N)		
Buried Pipe (Y,N, or Unknown)		
Channel Type (Trapezoidal, Rectangular, Triangular)	U	
	D	
Channel Bottom Width (if applicable)	U	
	D	
Channel Slope (if applicable) (H:V)	UR	
	UL	
	DR	
	DL	
Headwall (Y or N) and type (metal, concrete, etc.)	US	
	DS	
Condition of pipe(s) (new, damaged, evidence of joint separation, etc.)		
Flood Plain Description (wide, narrow, moderate, etc.)		
Vertical Geometry of Roadway (sag or flat)		
Shoulder/Embankment Description (Erosion? Scour holes present?, etc.)		
Other Notes		

Page 2 of 2 RP2021-08, Data Collection form V.3

10

10

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Instructions

19. Determine the existence of headwall and type of headwall material. Typical material types are concrete, metal, wood, and stones or extended rip-rap.




20. Explain the condition of the pipe(s) on both upstream and downstream. If no visible damage can be seen determine as new or no damage. If it is damaged explain whether it is broken, joint separated, squashed, etc. There are some cases where scour holes are visible on the shoulder and the separation of pipe or sign of breakage can be seen through the scour holes.


11

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Instructions

21. Describe the flood plain. Compare the width of the total structure (pipe/headwall + rip raps) with the channel. If the channel is wider than the total length, the flood plain is described as wide. If the stream is connected to a swamp, it is also considered as wide. If the channel is narrower than the total length, the flood plain is described as narrow. The case between wide and narrow is considered moderate.

22. Determine the vertical geometry of roadway at the location of pipe as follows:
    a. Stand on the roadway on the location of pipe.
    b. Compare the elevation of the roadway from where you stand with the elevation on both right and left side (far away from the location of pipe).
    c. If the elevation of the roadway on right and left sides are higher, the geometry is described as sag. If the elevation of the roadway on right and left sides are the same level as where you stand, the geometry is described as flat. In rare cases the elevation of the roadway on right and left sides are lower which the geometry is described as crest.


12

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Instructions

23. Describe the condition of shoulder/embankment on both upstream and downstream. Describe if there is any evidence of scour holes or erosion or other visible damage on the shoulder or embankment.
24. Explain any other features, signs of damage, and possible contributing factors based on your engineering judgement. For example, one of important contributing factors is existence of beaver dams near the channel.

13

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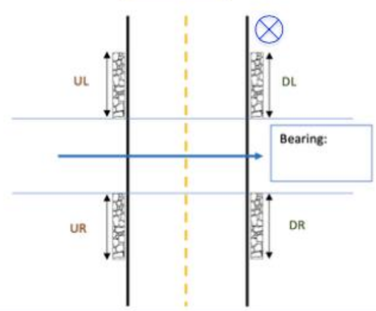
Instructions

24. Take soil sample from the downstream side shoulder as follows:
    a. Take the soil sample away from top of the pipe.
    b. Throw away the surface layer (about 3 in.) with grass and other things.
    c. Take the sample from lower layers.
    d. Label the sample bag with site number and date.
    e. Mark the approximate location where soil sample was taken on illustration in Page 1 of the data collection form.
25. Transfer data to provided google form.
26. Upload a scan of paper form to provided google form.
27. Rename photographs as described in handout and transfer to provided shared folder.



14

Site Evaluations

Table C.2. Summary of site evaluations.

Site Number	Complete Washout	Possible RCP Issues	Washaway Behind Headwall	Inconsistencies with Survey 123	Beaver Dam	Evidence of Ongoing Erosion	Bottom of Channel Erosion	Rip rap Lost/ Moved
51-1-MF	×	×	×	×	×	×	×	×
51-2-MF	×	×	×	×	×	×	×	×
51-3-MF	×	×	×	×	×	×	×	×
51-4-MF	×	×	×	×	×	✓	✓	✓
77-1-MF	✓	×	×	×	×	×	×	×
77-2-MF	×	×	×	×	×	×	×	×
77-3-MF	×	×	×	✓	×	✓	×	×
77-4-MF	×	✓	×	✓	×	✓	×	×
77-5-MF	×	×	×	×	×	×	×	×
77-6-MF	×	×	×	×	×	×	×	×
77-7-MF	✓	×	×	×	×	×	×	×
77-1-M	✓	×	×	×	×	×	×	×
77-2-M	×	✓	×	×	×	✓	×	×
77-3-M	×	×	×	×	×	✓	×	×
77-4-M	×	×	×	×	×	✓	×	×
77-5-M	×	×	×	×	✓	×	×	×
77-6-M	×	×	×	×	×	×	×	×
77-7-M	×	×	×	×	×	×	×	×
77-8-M	×	×	×	×	×	×	×	×
77-9-M	×	×	×	×	×	×	×	×
77-10-M	×	×	×	×	✓	✓	×	×
77-11-M	✓	×	×	×	×	×	×	×
77-12-M	✓	×	×	×	×	×	×	×
77-13-M	×	✓	×	×	×	✓	×	×
77-14-M	×	×	×	×	×	×	×	×
77-15-M	✓	×	×	×	×	×	×	×
77-16-M	×	×	×	×	×	×	×	×
77-1-F	×	×	×	×	×	×	×	×
77-2-F	×	×	×	✓	×	×	×	×
77-3-F	×	×	×	×	×	×	×	×
77-4-F	×	×	×	×	×	×	×	×
77-5-F	×	×	×	×	×	✓	×	×
77-6-F	×	✓	×	×	×	✓	×	×
77-7-F	×	×	×	×	×	×	×	×
77-8-F	×	×	×	×	×	×	×	×
77-9-F	×	×	×	×	×	×	×	×
95-1-MF	×	✓	✓	×	×	✓	×	×
95-2-MF	×	✓	×	✓	×	✓	×	✓
95-3-MF	×	×	×	✓	×	×	×	×
95-4-MF	✓	×	✓	×	×	×	×	×
95-5-MF	×	×	✓	×	×	×	×	×
95-6-MF	×	✓	✓	×	×	✓	×	×
95-1-M	×	×	×	×	×	×	×	×
95-2-M	×	×	×	×	×	×	×	×
95-3-M	×	×	×	×	×	✓	×	✓
95-1-F	×	×	×	×	×	✓	×	✓
95-2-F	×	×	×	×	×	✓	×	×
95-3-F	×	✓	×	×	×	×	×	×
53-1-MF	×	×	×	×	×	✓	×	✓

Site Number	Complete Washout	Possible RCP Issues	Washaway Behind Headwall	Inconsistencies with Survey 123	Beaver Dam	Evidence of Ongoing Erosion	Bottom of Channel Erosion	Rip rap Lost/ Moved
53-2-MF	×	×	×	×	×	×	×	×
53-3-MF	×	✓	×	×	×	✓	×	×
53-4-MF	×	×	×	✓	×	✓	✓	✓
53-5-MF	×	×	×	✓	×	✓	✓	×
53-6-MF	✓	×	×	✓	×	×	×	×
42-1-MF	×	×	×	✓	×	×	×	✓
25-1-MF	×	×	×	×	✓	✓	×	×
8-1-MF	×	×	×	✓	×	×	×	×
8-2-MF	×	✓	×	✓	×	×	×	×
8-3-MF	×	✓	×	×	×	×	×	×
23-1-MF	×	×	×	×	×	✓	×	×
23-2-MF	×	×	×	×	×	✓	×	✓
23-3-MF	×	✓	×	×	×	×	×	×
23-4-MF	×	×	×	×	×	×	×	×
23-5-MF	×	×	×	×	×	✓	×	×
95-4-M	×	×	×	×	×	×	×	×
95-5-M	×	×	×	×	×	×	×	×
95-6-M	×	×	✓	×	✓	✓	×	×
95-7-M	×	×	×	×	×	✓	×	×
95-8-M	✓	×	×	×	×	×	×	×
95-9-M	✓	×	×	×	×	✓	×	×
95-4-F	×	×	×	✓	×	✓	×	×
95-5-F	×	×	×	✓	×	×	×	×
95-6-F	×	×	×	✓	×	×	×	×
95-7-F	×	×	×	✓	×	✓	×	×
53-1-M	×	×	×	×	×	×	×	×
53-2-M	×	×	×	×	×	×	×	×
53-3-M	×	×	×	×	×	✓	×	×
53-4-M	×	×	×	×	×	×	×	×
53-5-M	×	×	×	×	×	✓	×	×
53-6-M	✓	×	×	×	×	✓	✓	×
53-1-F	×	×	×	×	×	×	×	×
53-2-F	×	×	×	×	×	✓	×	×
53-3-F	×	×	×	×	×	×	×	×
53-4-F	×	×	×	×	×	×	×	×
53-5-F	×	×	×	×	×	×	×	×
42-1-M	×	×	×	×	×	×	×	×
42-2-M	×	×	×	×	×	×	×	×
42-3-M	✓	×	×	×	×	×	×	×
42-1-F	×	×	×	×	×	✓	×	×
42-2-F	×	×	×	×	×	×	×	×
25-1-M	×	×	×	×	×	×	×	×
25-2-M	×	×	×	×	×	✓	×	×
25-1-F	×	×	×	×	×	×	×	×
25-2-F	×	×	×	✓	×	×	×	×
8-1-M	×	×	×	×	×	×	×	×
8-2-M	×	×	×	×	×	×	×	×
8-3-M	×	×	×	×	×	×	×	×
8-4-M	×	×	×	×	×	✓	×	×
8-1-F	×	×	×	×	×	×	×	×
8-2-F	×	×	×	✓	×	×	×	×
8-3-F	×	×	×	×	×	×	×	×

Site Number	Complete Washout	Possible RCP Issues	Washaway Behind Headwall	Inconsistencies with Survey 123	Beaver Dam	Evidence of Ongoing Erosion	Bottom of Channel Erosion	Rip rap Lost/ Moved
8-4-F	×	×	×	×	×	×	×	×
8-5-F	×	×	×	×	×	×	×	×
8-6-F	×	×	×	✓	×	×	×	×
8-7-F	×	×	×	×	×	×	×	×
23-1-M	×	×	×	×	×	×	×	×
23-2-M	×	×	×	×	×	×	×	×
23-3-M	×	×	×	×	×	×	×	×
23-4-M	×	×	×	×	×	×	×	×
23-5-M	×	×	×	×	×	✓	×	×
23-1-F	×	×	×	×	×	×	×	×
23-2-F	×	×	×	×	×	×	×	×
23-3-F	×	×	×	×	×	×	×	×
23-4-F	×	×	×	✓	×	✓	×	×
95-8-F	×	×	×	×	×	×	×	×
95-9-F	×	×	×	✓	×	×	×	×
95-10-F	×	×	×	×	×	×	×	×
95-11-F	×	×	×	×	×	✓	✓	×
95-12-F	×	×	×	×	×	×	×	×
53-7-M	×	×	×	×	×	×	×	×
53-8-M	×	×	×	×	×	×	×	×
53-9-M	×	×	×	×	×	✓	×	×
53-6-F	×	×	×	×	×	×	×	×
53-7-F	×	×	×	×	×	×	×	×
42-4-M	×	×	×	×	×	×	×	×
42-5-M	×	×	×	×	×	×	×	×
42-3-F	×	×	×	×	×	×	×	×
25-3-M	×	×	×	×	×	×	×	×
25-4-M	×	×	×	×	×	×	×	×
25-3-F	×	×	×	×	×	×	×	×
25-4-F	×	×	×	×	×	×	×	×
8-5-M	×	×	×	×	×	×	×	×
8-6-M	×	×	×	×	×	×	×	×
8-7-M	×	×	×	×	×	×	×	×
8-8-F	×	×	×	×	×	×	×	×
23-6-M	×	×	×	×	×	×	×	×
23-5-F	×	×	×	×	×	×	×	×
23-6-F	×	×	×	×	×	×	×	×

Table C.3. Summary of damage assessment of study sites.

Site Number	Level of Damage				
	Shoulder	Pavement	Repair	Pipe	Total
8-1-MF (F)	3	1	3	2	9
8-1-MF (M)	3	1	3	1	8
8-2-F	1	0	0	1	2
8-2-MF (F)	3	0	2	2	7
8-2-MF (M)	2	1	1	2	6
8-3-MF (F)	3	1	2	1	7
8-3-MF (M)	3	1	2	0	6
8-4-M	1	0	0	2	3
8-6-F	3	1	1	2	7
8-7-M	2	0	2	2	6
23-1-MF (F)	2	0	0	0	2
23-1-MF (M)	1	0	0	0	1
23-2-MF (F)	3	1	1	0	5
23-2-MF (M)	1	0	0	0	1
23-4-F	2	0	0	2	4
23-5-M	2	0	1	0	3
23-5-MF (F)	2	1	2	2	7
23-5-MF (M)	2	0	0	0	2
25-1-MF (F)	1	0	0	1	2
25-1-MF (M)	1	0	1	0	2
25-2-F	1	1	2	2	6
25-2-M	3	1	3	2	9
42-1-F	0	1	2	2	5
42-1-MF (F)	3	1	2	2	8
42-1-MF (M)	3	1	2	1	7
42-3-M	3	3	3	3	12
53-1-MF (F)	3	1	2	2	8
53-1-MF (M)	3	1	2	1	7
53-2-F	2	0	0	1	3
53-2-MF (F)	2	1	2	2	7
53-2-MF (M)	3	1	2	0	6
53-3-M	3	1	2	0	6
53-3-MF (F)	3	1	3	2	9
53-3-MF (M)	3	2	3	3	11
53-4-MF (F)	3	1	2	2	8
53-4-MF (M)	2	1	2	0	5
53-5-M	3	0	1	2	6
53-5-MF (F)	3	1	2	2	8
53-5-MF (M)	3	1	2	0	6
53-6-M	3	3	3	3	12
53-6-MF (F)	3	0	1	2	6
53-6-MF (M)	3	3	3	3	12
53-9-M	3	1	2	2	8
95-11-F	2	0	0	2	4
95-1-F	1	0	0	0	1
95-1-M	1	1	1	0	3
95-1-MF (F)	1	0	0	0	1
95-1-MF (M)	1	0	0	1	2
95-2-F	1	0	0	2	3
95-2-M	1	1	1	2	5
95-2-MF (F)	3	1	2	2	8
95-2-MF (M)	1	1	1	0	3

Site Number	Level of Damage				
	Shoulder	Pavement	Repair	Pipe	Total
95-3-F	0	1	0	2	3
95-3-M	1	1	1	2	5
95-3-MF (F)	1	0	0	0	1
95-3-MF (M)	1	1	1	2	5
95-4-F	1	0	0	2	3
95-4-M	1	0	0	0	1
95-4-MF (F)	2	0	0	0	2
95-4-MF (M)	3	3	3	3	12
95-5-F	1	1	1	2	5
95-5-M	1	1	1	0	3
95-5-MF (F)	2	0	0	0	2
95-5-MF (M)	3	2	1	3	9
95-6-F	2	0	0	2	4
95-6-M	1	0	0	1	2
95-6-MF (F)	2	0	0	0	2
95-6-MF (M)	2	1	1	2	6
95-7-F	1	0	0	2	3
95-7-M	2	2	2	2	8
95-8-F	2	0	0	0	2
95-8-M	3	3	3	3	12
95-9-F	2	0	0	2	4
95-9-M	3	3	3	3	12

APPENDIX D: HYDRAULIC ANALYSIS RESULTS

Base Peak Discharge Results

Based on available hydraulic data necessary for conducting hydraulic analysis, the following sites were selected for this research study.

Table D.1. Results of HDS-5 and HY-8 analysis for base peak discharge for all identified sites (red cells have calculated HW/D > 1.2 and green cells have calculated HW/D ≤ 1.2).

Case ID	Street Name	Base Discharge HDS-5			Base Discharge HY-8		
		Base Matthew	Base Florence	Base Current	Base Matthew	Base Florence	Base Current
95-1-MF	Polly Watson Rd	1.04	1.01	1.09	1.8	1.8	1.13
95-2-MF	Mark Herring Rd	1.51	2.17	0.96	1.8	2.03	1.09
95-3-MF	Stevens Mill Rd	2.56	0.8	0.72	2.55	0.86	0.78
95-4-MF	NC 55	1.09	1.93	1.93	1.16	1.77	1.77
95-5-MF	Corbett Hill Rd	0.88	0.53	0.53	0.95	0.6	0.59
95-6-MF	Mark Herring Rd	0.86	0.65	0.65	0.89	0.78	0.78
95-7-MF-M	Overman Rd	5.48	6	6	2.3	1.97	1.97
95-8-MF-M	Raynor Mill Rd	0.56	0.4	0.4	0.59	0.56	0.56
95-8-MF-F	Raynor Mill Rd	-	0.43	0.2	-	0.27	0
95-9-MF-M	North Center St	0.9	0.65	0.65	0.98	0.7	0.7
95-1-M	Nahunta Rd	1.92	2.55	2.55	1.97	1.91	1.91
95-2-M	Big Daddys Rd	2.45	0.83	0.83	2.5	0.93	0.93
95-3-M	Wayne Memorial Drive	0.83	0.68	0.68	0.84	0.77	0.77
95-4-M	James Hinson Rd	1.63	1.44	1.44	1.62	1.46	1.46
95-5-M	Sheridan Forest Rd	0.29	0.29	0.29	0.28	0.28	0.28
95-6-M	NC 55	0.59	0.59	0.59	0.73	0.73	0.73
95-1-F	Hooks Rd	-	16.79	0.88	-	3.48	0.88
95-2-F	North Washington St	-	3.52	1.19	-	4.18	2.37
95-3-F	Pinkney Rd	-	0.96	0.79	-	1.06	0.9
95-4-F	US 117	-	10.57	1.71	-	3.45	1.77
95-5-F	Old Harvey Sutton Rd	-	2.17	0.61	-	2.17	0.7
95-6-F	Spring Bank Rd	-	2.92	3.01	-	2.17	1.75
95-7-F	Mark Herring Rd	-	1.1	0.59	-	1.06	0.67
95-8-F	James Prince Rd	-	0.61	0.94	-	0.69	1.1
95-9-F	South Jordan's Chapel Rd	-	1.08	0.72	-	1.36	0.81
95-11-F	NC 581	-	1.81	0.77	-	1.83	0.84
53-1-MF	Gray Branch Church Rd	9.88	9.88	1.05	2.87	2.87	1.21
53-2-MF	Davis Mill Rd	0.78	0.78	0.58	0.87	0.87	0.62
53-3-MF	N Croom Bland Rd	80.55	46.19	1.18	6.32	5.03	1.28
53-4-MF	Dalys Chapel Rd	1.01	1.01	0.71	1.24	1.24	0.78
53-5-MF	Eric Sparrow Rd	1.98	1.98	1.55	2.17	2.17	1.29
53-6-MF	NC 903	0.65	0.65	1.2	0.31	0.27	1.01
53-7-MF	Davis Mill Rd	2.61	0.76	0.83	2.12	0.83	0.9
53-3-M	NC 55	0.99	1.15	1.15	1.06	1.23	1.23
53-5-M	Old Pink Hill Rd	2.28	1.25	1.25	2.32	1.23	1.23
53-6-M	Falling Creek Rd	1.25	0.57	0.57	1.68	0.13	0.13
53-1-F	Liddell Shortcut Rd	-	0.85	1.24	-	1.04	1.79
53-2-F	W. Pleasant Hill Rd	-	1.48	1.48	-	1.47	1.47
42-1-MF	Harnett	2.5	0.69	1.01	2.68	0.91	0.8
42-3-M	Brick Mill Rd	0.48	0.48	0.73	0.5	0.5	0.87

Case ID	Street Name	Base Discharge HDS-5			Base Discharge HY-8		
		Base Matthew	Base Florence	Base Current	Base Matthew	Base Florence	Base Current
42-1-F	Wiry Rd	-	36.15	2.36	-	2.43	1.64
25-1-MF	Cumberland	-	6.99	3.92	-	0.68	0.57
25-2-M	Yarborough Rd	1.85	0.66	0.66	1.9	0.79	0.79
25-2-F	LA Dunham Rd	-	1.57	1.35	-	1.64	1.09
8-1-MF-M	NC 242	2.26	1.06	1.06	2.07	1.14	1.14
8-2-MF	Brown Creek Church Rd	1.72	1.72	0.37	1.78	1.78	0.57
8-3-MF	Sweet Home Church Rd	7.97	1.75	1.38	4.02	1.81	1.72
8-4-MF	Twisted Hickory Rd	1.92	1.92	1.92	-	3.64	3.64
8-4-M	Old Abbotts burg Rd	17.91	0.77	0.77	3.53	0.86	0.86
8-2-F	Everette Byrd Road	-	34.21	47.8	-	4.36	4.24
8-6-F	Lisbon Road	-	5.89	0.82	-	3.31	0.9
23-1-MF	Union Valley Rd	3.83	3.98	3.98	4.39	2.77	2.77
23-2-MF	Greens Mill Rd	8.74	8.74	8.74	3.13	3.13	3.13
23-3-MF	Sikes Rd	1.23	0.76	0.76	1.39	0.87	0.87
23-5-MF	Old US 74	3.46	3.46	0.85	3.48	3.48	0.93
23-5-M	Mill Pond Rd	71.7	37.1	37.1	3.26	2.32	2.32
23-4-F	Peacock Rd	-	0.63	0.36	-	0.69	0.25
77- 1-MF	Smith Mill	123.80	1.09	1.09	4.61	1.32	1.32
77- 2-MF	Dallas Road	2.25	2.25	1.09	2.13	2.13	1.08
77- 3-MF	Dogwood Rd	2.90	2.90	2.90	2.61	2.61	2.61
77- 4-MF	Ashpole Church Rd	8.09	8.09	8.09	3.44	3.44	3.44
77- 5-MF	Kitchen St Rd	4.05	4.05	1.00	3.33	3.33	1.038
77- 6-MF	Persimmon	43.60	0.77	0.94	3.84	0.78	1.07
77- 7-MF	Fairley Rd	2.50	1.00	1.00	2.32	1.17	1.17
77- 1-M	Howell Rd	1.27	1.05	1.05	1.49	1.07	1.07
77- 2-M	Snake Rd	2.53	1.60	1.60	2.33	1.92	1.92
77- 3-M	Cedar Grove Rd	25.96	2.16	2.16	4.42	2.35	2.35
77- 4-M	Mt. Moriah Church Rd	4.64	0.84	0.84	2.29	0.98	0.98
77- 5-M	Turnpike Rd	2.19	1.40	1.40	1.55	1.39	1.39
77- 6-M	Vester Rd	0.95	0.81	0.81	1.23	0.99	0.99
77- 7-M	Pinelog Rd	5.98	1.99	1.99	5.38	1.99	1.99
77- 8-M	McCrimmon Rd	2.04	0.89	0.89	1.75	1.07	1.07
77- 9-M	K B Rd	1.93	1.25	1.25	1.92	1.34	1.34
77- 10-M	Oakgrove Church Rd	0.77	0.61	0.61	1.04	0.79	0.79
77- 11-M	McDuffie Crossing Rd	1.92	0.94	0.94	1.72	1.08	1.08
77- 12-M	Midway Rd	0.57	0.57	0.57	0.67	0.67	0.67
77- 13-M	Marietta Rd	1.61	0.95	0.95	1.63	1.13	1.13
77- 14-M	Cowpen Swamp Rd	12.94	0.85	0.85	4.27	1.02	1.02
77- 15-M	Carolina Church Rd	0.58	0.58	0.58	0.67	0.67	0.67
77- 16-M	Centerville Church Rd	0.75	0.38	0.38	0.93	0.47	0.47
77- 1-F	Evon Rd	-	4.10	4.10	-	4.07	4.07
77- 2-F	Pearsall Rd	-	42.28	1.57	-	3.86	1.13
77- 3-F	Fayetteville Rd	-	3.61	3.61	-	2.19	2.19
77- 4-F	Sanderson Rd	-	4.79	0.86	-	2.50	0.90
77- 5-F	W Powersville Rd	-	4.56	-	-	2.83	-
77- 6-F	Townsend Rd	-	4.49	-	-	2.83	-
77- 7-F	John French Rd	-	66.47	1.05	-	6.66	2.37
77- 8-F	McIver Rd	-	2.54	0.82	-	2.15	0.98
77- 9-F	Pleasant Hope Rd	-	4.79	0.76	-	3.32	0.93

Adjusted Peak Discharge Results

The examined potential flow scenarios shown in Table D.2 as Ratio 1 and Ratio 2, are the ones calculated using the NOAA definition of 24-hr event and the heaviest 24-hr period respectively.

Table D.2. Results of HDS-5 and HY-8 for peak discharge values adjusted using Ratios 1 and 2. (red cells have calculated HW/D > 1.2 and green cells have calculated HW/D ≤ 1.2).

Case ID	Street Name	Adjusted Discharge HDS-5 HW/D				Adjusted Discharge HY-8 HW/D			
		Matthew R1	Matthew R2	Florence R1	Florence R2	Matthew R1	Matthew R2	Florence R1	Florence R2
95-1-MF	Polly Watson Rd	1.59	1.59	0.94	0.94	2.04	2.04	1.65	1.65
95-2-MF	Mark Herring Rd	1.88	1.88	8.04	8.04	1.91	1.91	2.49	2.49
95-3-MF	Stevens Mill Rd	5.68	5.68	1.02	1.02	3.95	3.95	1.1	1.1
95-4-MF	NC 55	1.24	1.31	6.81	6.81	1.35	1.44	2.17	2.17
95-5-MF	Corbett Hill Rd	1.62	1.61	0.68	0.68	1.43	1.43	0.75	0.75
95-6-MF	Mark Herring Rd	0.97	0.97	1.11	1.11	0.99	0.99	1.3	1.3
95-7-MF-M	Overman Rd	11.67	11.67	16.29	16.29	2.52	2.52	2.25	2.25
95-8-MF-M	Raynor Mill Rd	0.81	0.84	0.52	0.52	0.86	0.89	0.73	0.73
95-8-MF-F	Raynor Mill Rd	-	-	0.53	0.53	-	-	0.57	0.57
95-9-MF-M	North Center St	0.96	1.02	1.09	1.14	1.04	1.11	1.13	1.17
95-1-M	Nahunta Rd	3.13	3.47	2.96	3.36	2.13	2.17	1.95	2
95-2-M	Big Daddys Rd	3.78	4.31	0.88	0.93	3.25	3.29	0.99	1.04
95-3-M	Wayne Memorial Dr	0.88	0.94	0.72	0.8	0.89	0.94	0.82	0.91
95-4-M	James Hinson Rd	3	3.5	2.72	2.72	1.87	1.9	1.83	1.83
95-5-M	Sheridan Forest Rd	0.38	0.38	0.39	0.39	0.36	0.36	0.38	0.38
95-6-M	NC 55	0.73	0.76	0.72	0.76	0.91	0.99	0.9	0.94
95-1-F	Hooks Rd	-	-	20.41	24.02	-	-	3.93	3.96
95-2-F	North Washington St	-	-	4.16	4.8	-	-	4.2	4.23
95-3-F	Pinkney Rd	-	-	0.96	1.1	-	-	1.06	1.2
95-4-F	US 117	-	-	40.7	40.7	-	-	3.93	3.93
95-5-F	Old Harvey Sut, Rd	-	-	4.96	4.96	-	-	3.18	3.18
95-6-F	Spring Bank Rd	-	-	6.71	6.71	-	-	2.39	2.39
95-7-F	Mark Herring Rd	-	-	2.86	2.86	-	-	1.77	1.77
95-8-F	James Prince Rd	-	-	2.14	2.14	-	-	1.68	1.68
95-9-F	S Jordan's Chap. Rd	-	-	1.44	1.44	-	-	1.95	1.95
95-11-F	NC 581	-	-	1.81	2.31	-	-	1.83	2.36
53-1-MF	Gray Branch Ch Rd	11.98	11.98	29.96	29.96	2.92	2.92	3.15	3.15
53-2-MF	Davis Mill Rd	0.83	0.83	1.38	1.38	0.92	0.92	1.48	1.48
53-3-MF	N Croom Bland Rd	116.45	116.45	246.97	246.97	6.52	6.52	5.85	5.85
53-4-MF	Dalys Chapel Rd	1.14	1.14	2.01	2.01	1.43	1.43	2.07	2.07
53-5-MF	Eric Sparrow Rd	2.31	2.31	5.08	5.08	2.22	2.22	2.48	2.48
53-6-MF	NC 903	0.73	0.73	1.08	1.08	1.18	1.18	1.69	1.69
53-7-MF	Davis Mill Rd	3.07	3.07	1.58	1.58	2.16	2.16	1.22	1.22
53-3-M	NC 55	1.18	1.18	2.87	2.87	1.27	1.27	2.55	2.55
53-5-M	Old Pink Hill Rd	2.68	2.68	2.64	2.64	2.58	2.58	1.92	1.92
53-6-M	Falling Creek Rd	1.38	1.51	0.61	0.64	1.9	2.11	0.14	0.15
53-1-F	Liddell Shortcut Rd	-	-	1.71	1.71	-	-	1.63	1.63
53-2-F	W. Pleasant Hill RD	-	-	2.82	3.18	-	-	2.59	2.7
42-1-MF	Harnett	3.37	4.4	0.65	0.65	2.87	3.02	0.85	0.85
42-3-M	Brick Mill Rd	0.48	0.55	0.41	0.56	0.5	0.57	0.43	0.47
42-1-F	Wine Rd	-	-	20.13	36.15	-	-	2.22	2.43
25-1-MF	Cumberland	-	-	6.05	8.41	-	-	0.64	0.72
25-2-M	Yarborough Rd	4.66	6.43	0.61	0.66	3.22	3.31	0.73	0.79
25-2-F	LA Dunham Rd	-	-	0.75	0.89	-	-	1.51	1.64
8-1-MF-M	NC 242	2.62	2.62	1.55	1.8	2.18	2.18	1.56	1.66
8-2-MF	Brown Creek Ch Rd	1.98	3.19	6.38	8.5	2.04	2.69	2.85	2.95
8-3-MF	Sweet Home Church Rd	6.88	9.61	3.81	4.35	3.88	4.12	2.41	2.46

Case ID	Street Name	Adjusted Discharge HDS-5 HW/D				Adjusted Discharge HY-8 HW/D			
		Matthew R1	Matthew R2	Florence R1	Florence R2	Matthew R1	Matthew R2	Florence R1	Florence R2
8-4-MF	Twisted Hickory Rd	-	-	4.36	4.66	-	-	2.19	2.25
8-4-M	Old Abbottsburg Rd	30.08	55.27	1.15	1.3	3.68	3.85	1.21	1.29
8-2-F	Everette Byrd Rd	-	-	257.05	287.21	-	-	6.51	6.75
8-6-F	Lisbon Rd	-	-	31.72	41.32	-	-	3.82	3.82
23-1-MF	Union Valley Rd	4.58	7.95	3.46	3.98	4.5	4.93	2.51	2.77
23-2-MF	Greens Mill Rd	7.51	16.72	14.47	16.72	3.03	3.28	3.24	3.28
23-3-MF	Sikes Rd	1.23	2.15	0.61	0.66	1.39	2.28	0.7	0.76
23-5-MF	Old US 74	4.11	7.1	5.49	5.49	3.53	3.68	3.62	3.62
23-5-M	Mill Pond Rd	71.7	111.73	45.33	57.67	3.26	4.23	2.42	2.42
23-4-F	Peacock Rd	-	-	0.77	0.77	-	-	0.84	0.84
77- 1-MF	Smith Mill	243.9	377.9	1.84	2.59	4.99	5.28	2.4	2.76
77- 2-MF	Dallas Rd	3.98	6.45	2.68	2.79	2.31	2.49	1.49	1.22
77- 3-MF	Dogwood Rd	4.78	9.45	1.5	1.52	2.84	3.13	1.5	1.53
77- 4-MF	Ashpole Church Rd	10.5	20.67	15.54	25.64	3.49	3.65	3.58	3.72
77- 5-MF	Kitchen St Rd	4.1	8.45	4.05	7.08	3.33	3.58	3.33	1.76
77- 6-MF	Persimmon	52.1	107.44	0.8	1.08	3.9	4.14	0.81	1.35
77- 7-MF	Fairley Rd	2.3	4.33	0.99	1.21	2.26	2.69	1.15	1.39
77- 1-M	Howell Rd	1.78	2.61	1.5	1.78	1.76	1.98	1.54	1.86
77- 2-M	Snake Rd	4.65	4.65	3.84	3.52	2.5	2.5	2.25	2.23
77- 3-M	Cedar Grove Rd	54.9	54.9	5.75	5.28	4.61	4.61	2.59	2.57
77- 4-M	Mt. Moriah Ch Rd	6.6	8.73	1.17	1.36	2.38	2.45	1.31	1.41
77- 5-M	Turnpike Rd	3.46	2.99	2.92	3.12	1.73	1.68	1.75	1.77
77- 6-M	Vester Rd	1.45	1.23	1.29	1.26	1.68	1.36	1.55	1.5
77- 7-M	Pinelog Rd	9.92	9.37	4.78	3.42	5.49	5.47	4.91	3.45
77- 8-M	McCrimmon Rd	1.89	1.99	1.18	1.07	1.71	1.74	1.5	1.44
77- 9-M	K B Rd	3.02	2.33	2.46	1.57	2.72	2.32	2.33	1.59
77- 10-M	Oakgrove Ch Rd	0.95	0.92	0.9	0.79	1.35	1.3	1.21	1.06
77- 11-M	McDuffie Cross. Rd	2.85	2.56	2.38	2.19	2	1.95	1.65	1.56
77- 12-M	Midway Rd	0.62	0.64	0.76	0.77	0.74	0.76	0.91	0.92
77- 13-M	Marietta Rd	2.37	1.06	0.99	0.44	2.11	1.1	1.63	0.78
77- 14-M	Cowpen Swamp Rd	17.74	6.34	1.49	0.77	4.34	4.13	1.49	0.93
77- 15-M	Carolina Church Rd	0.65	0.5	0.76	0.61	0.74	0.58	0.87	0.7
77- 16-M	Centerville Ch Rd	0.99	0.8	0.57	0.41	1.19	0.99	0.68	0.5
77- 1-F	Evon Rd	6.44	5.92	10.84	8.3	4.21	4.18	4.37	4.29
77- 2-F	Pearsall Rd	16.54	15.14	1.61	1.46	3.99	3.94	1.88	1.62
77- 3-F	Fayetteville Rd	6.08	8.88	9.77	12.32	2.32	2.42	2.44	2.5
77- 4-F	Sanderson Rd	8.17	7.4	1.72	1.52	2.63	2.61	1.93	1.18
77- 5-F	W Powersville Rd	6.72	8.89	-	-	2.92	2.98	-	-
77- 6-F	Townsend Rd	8.34	7.13	-	-	2.96	2.93	-	-
77- 7-F	John French Rd	1227.33	886.67	12.46	7.51	6.99	6.81	2.98	2.73
77- 8-F	McIver Rd	4.86	2.89	8.51	3.02	2.51	2.2	2.41	1.74
77- 9-F	Pleasant Hope Rd	6.63	3.13	2.31	1.64	3.4	3.13	3.5	2

Design Storm Uncertainty

Design storm uncertainty results derived from the analysis of AEP equivalents to the 25-yr storm are shown in Figure D.1, Figure D.2, Figure D.3 and Figure D.4. Minor variations were observed in the 87.5th percentile, aligning with the 7.5-yr storm for 1 and 5 square miles (Figure D.1, Figure D.3), and the 8-yr storm for 2 and 10 square miles (Figure D.2 and Figure D.4). Similar slight discrepancies are noted in the 25th, 12.5th, and 4th percentiles at different drainage areas. A summary of the results is presented in Figure in Table 21.

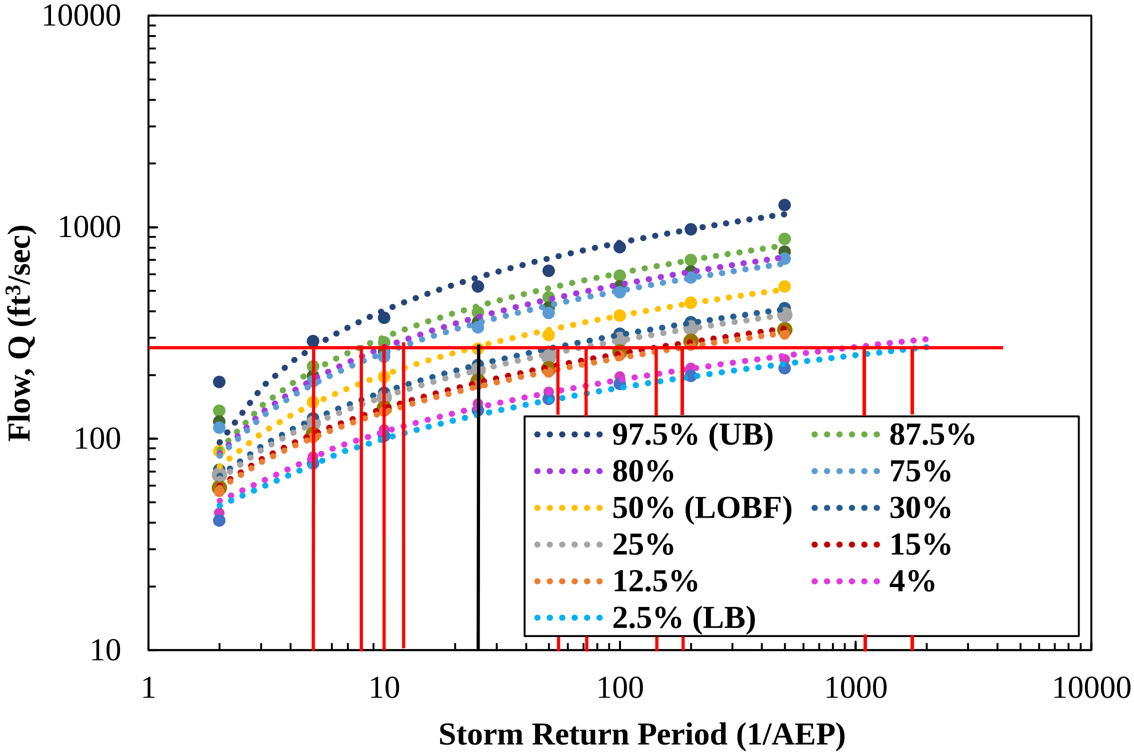


Figure D.1. Relationship between the flow values (Q) for respective percentiles and storm return periods (1/AEP) when drainage area is one (1) square mile.

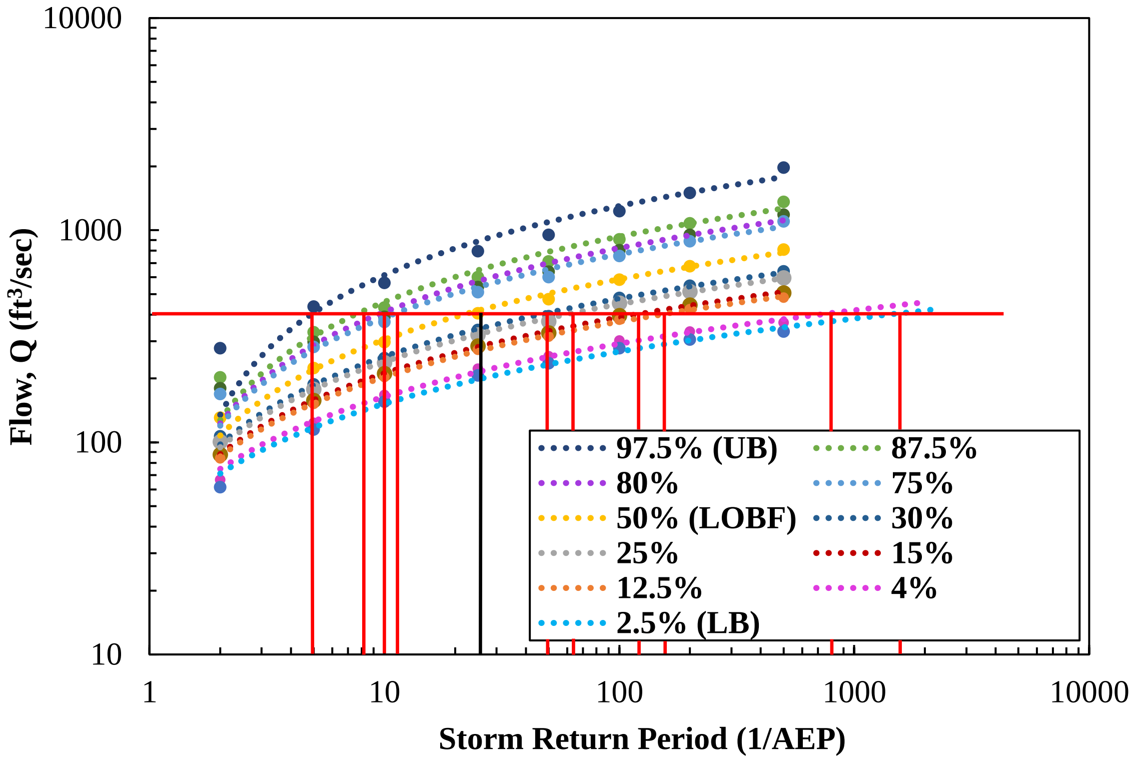


Figure D.2. Relationship between the flow values (Q) for respective percentiles and storm return periods (1/AEP) when drainage area is two (2) square miles.

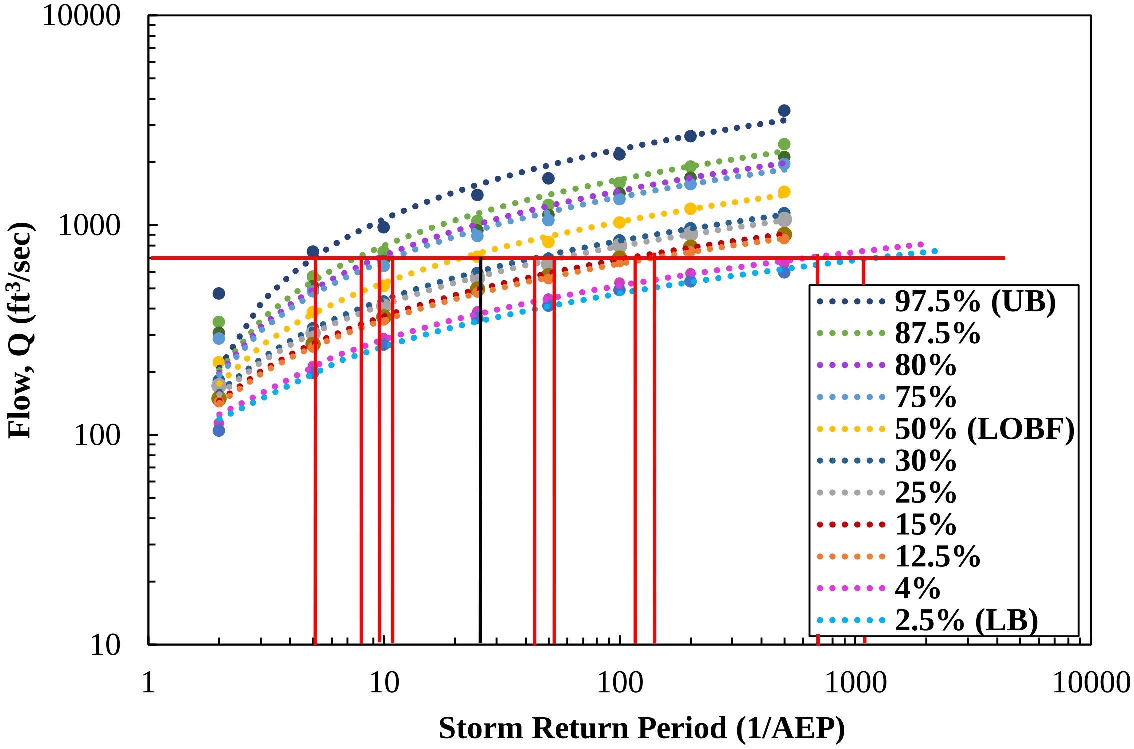


Figure D.3. Relationship between the flow values (Q) for respective percentiles and storm return periods (1/AEP) when drainage area is five (5) square miles.

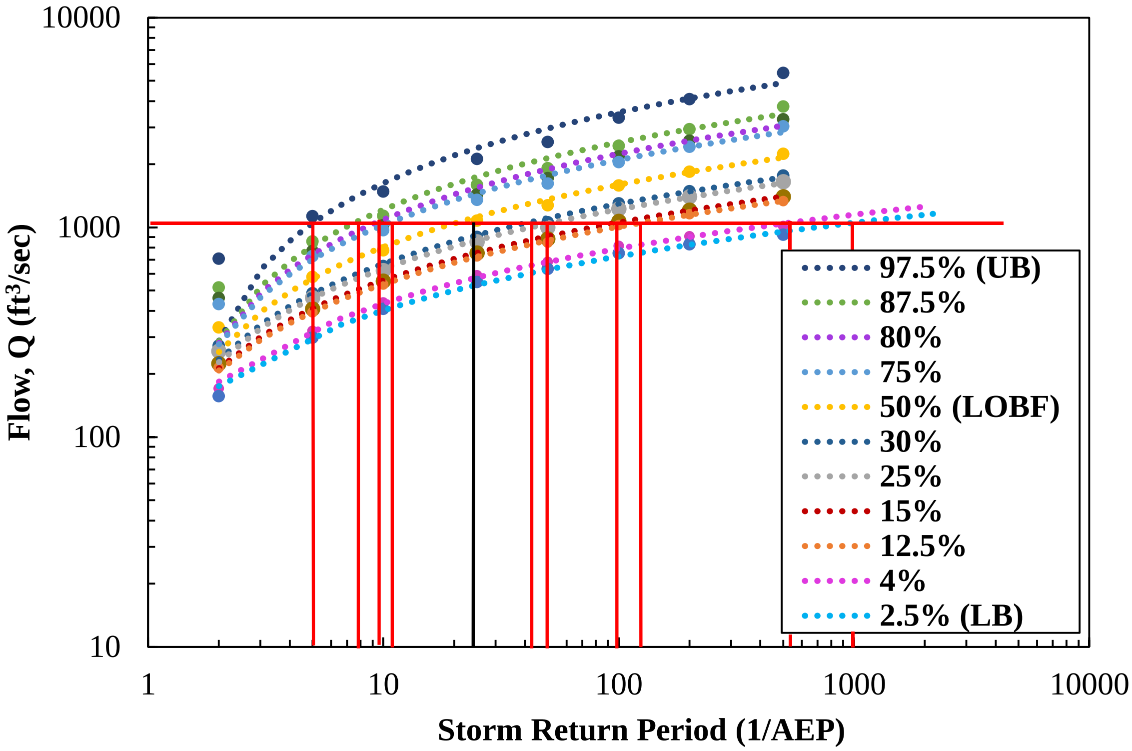


Figure D.4. Relationship between the flow values (Q) for respective percentiles and storm return periods (1/AEP) when drainage area is ten (10) square miles.

Subsequent examination of the relationship between overtopping flow values derived from peak design discharge and corresponding percentiles yielded a range of results. The analysis of eight culvert cases, which experienced damage during Hurricanes Matthew and Florence, had the following conclusions:

- Cases 95-9-MF-M, 95-8-MF-M, and 42-3-M initially had low probabilities of overtopping prior to Hurricane Matthew. However, hydraulic analysis results depicted in Figure D.5, Figure D.6 and Figure D.7 indicated that including headwalls and upsizing culverts improved performance during the design event. Notably, actual culvert damage, documented in hydro reports, revealed complete washouts in these cases. Conversely, Case 42-3-M, which downscaled culvert size (Figure D.7 (b)), exhibited increased post-Hurricane Matthew overtopping probability. This variance in hydraulic headwater values and actual performance challenges conventional overtopping criteria.
- Cases 53-5-M and 25-2-F exhibited a 47% overtopping probability during Hurricane Matthew (Figure D.8 (a) and Figure D.9 (a)), suggesting a recurring trend in the project's investigated cases. After upsizing and/or adding headwalls (Figure D.8 (b) and Figure D.9 (b)), overtopping probabilities decreased to 30% and 21%, respectively.
- Cases 95-2-MF and 42-1-MF had initial overtopping probabilities of 65% and 61%, respectively (Figure D.10 (a) and Figure D.11 (a)). Upsizing these culverts reduced post-Hurricane Florence overtopping probabilities, resulting in 40% for case 95-2-MF and overtopping occurring at the 95th percentile of the 2% AEP for case 42-1-MF (Figure D.10 (b) and Figure D.11 (b)).
- Case 8-4-MF (Figure D.12) exhibited a notably high overtopping probability during Hurricane Florence, with culvert overtopping at the 60th percentile of 50% AEP.

These findings highlight the complexities of culvert performance during storm events, calling for a nuanced evaluation of culvert resilience in relation to hydraulic analysis results and actual damage outcomes.

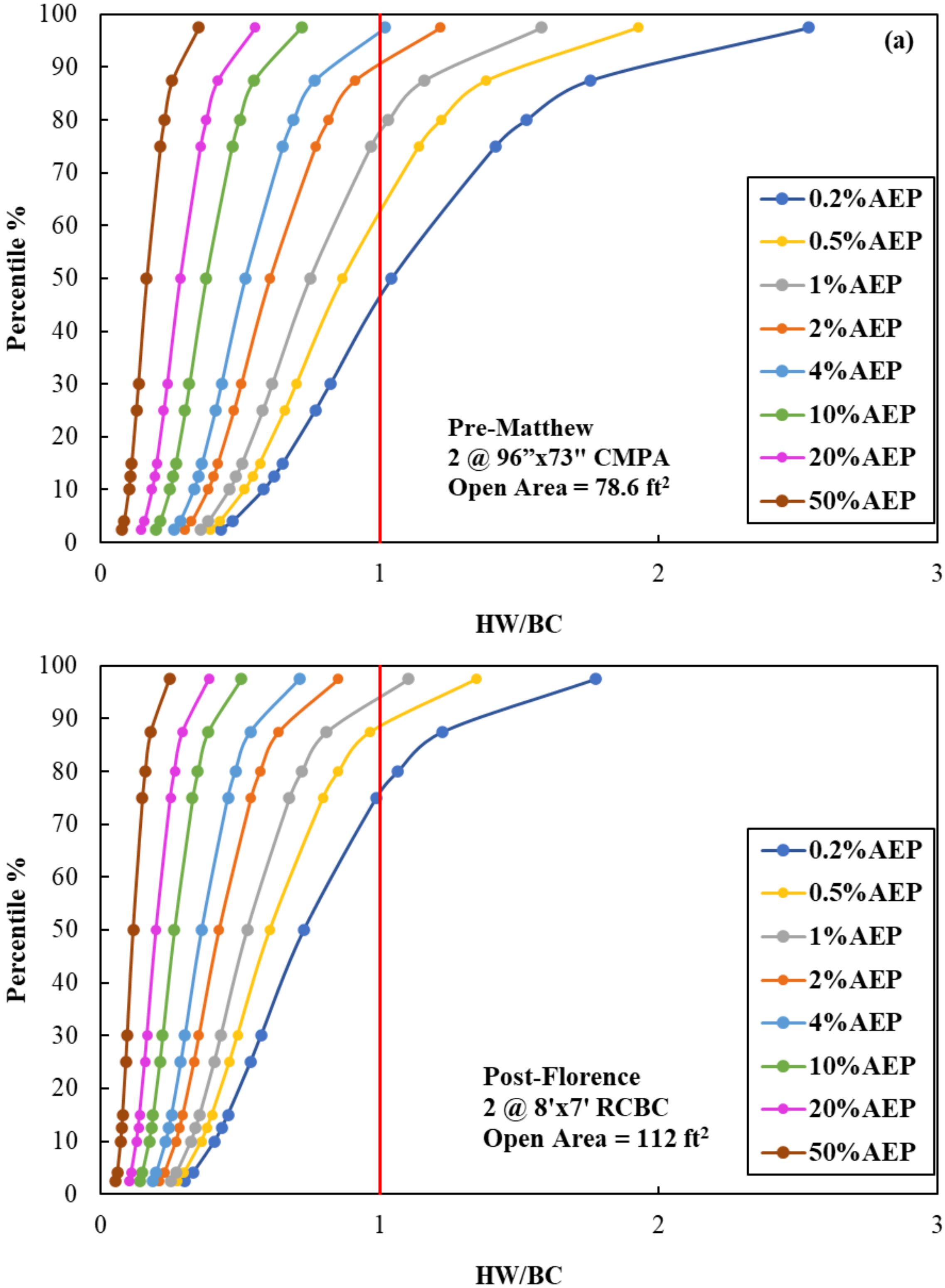


Figure D.5. Probabilistic analysis of overtopping flow values across percentiles for Site 95-9-MF: (a) pre-Matthew structure and (b) post-Matthew structure.

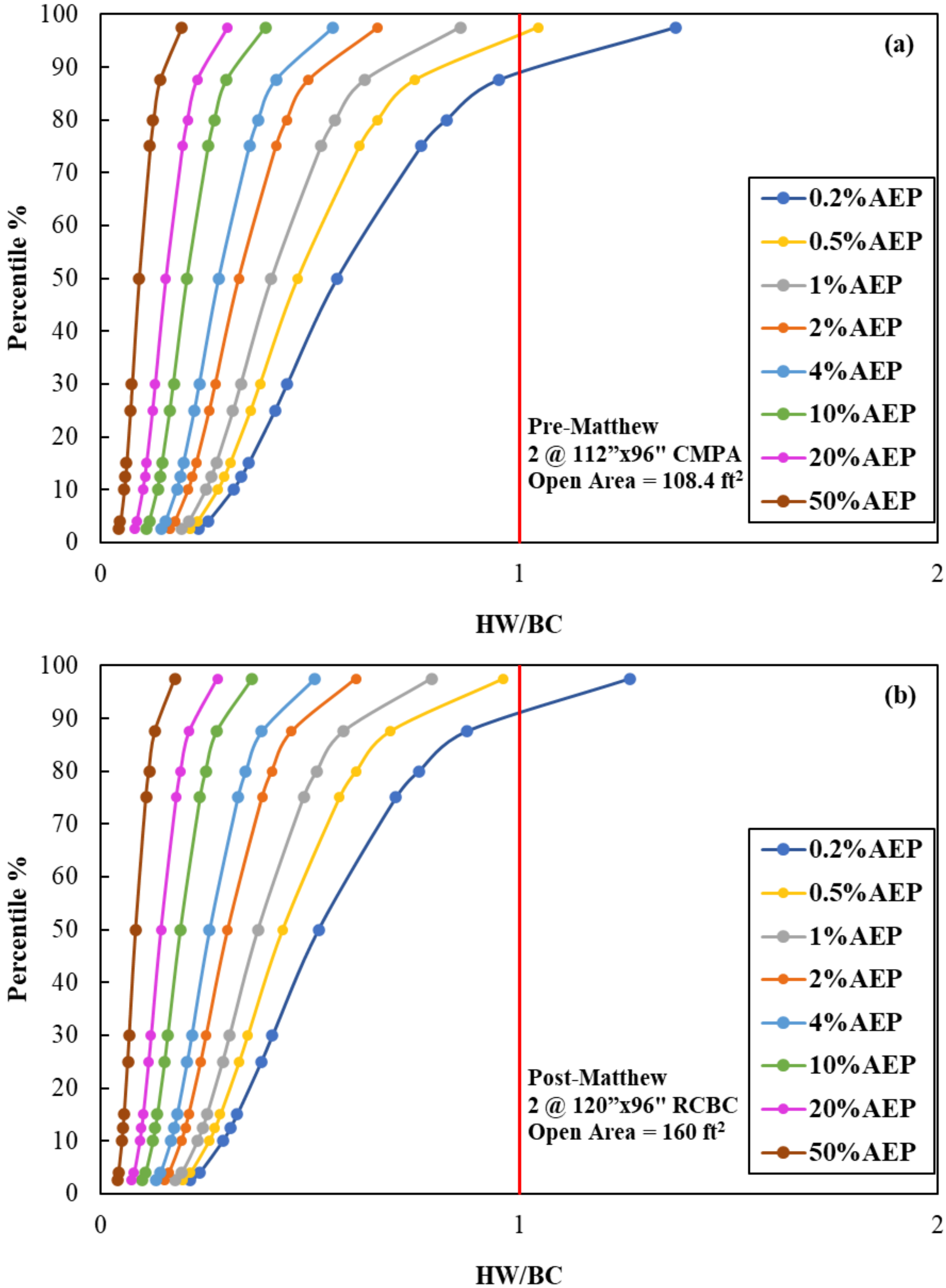


Figure D.6. Probabilistic analysis of overtopping flow values across percentiles for Site 95-8-MF: (a) pre-Matthew structure and (b) post-Matthew structure.

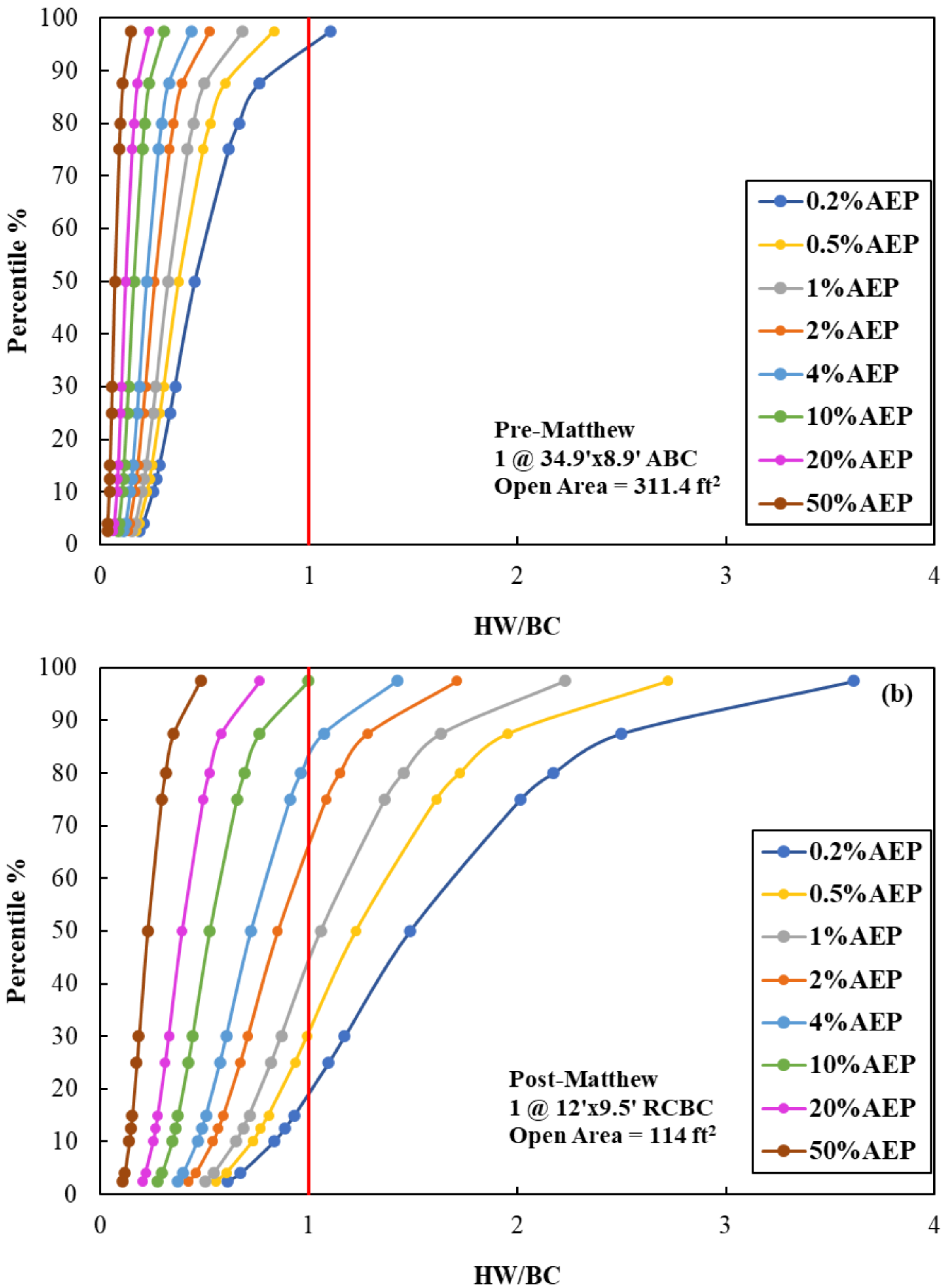
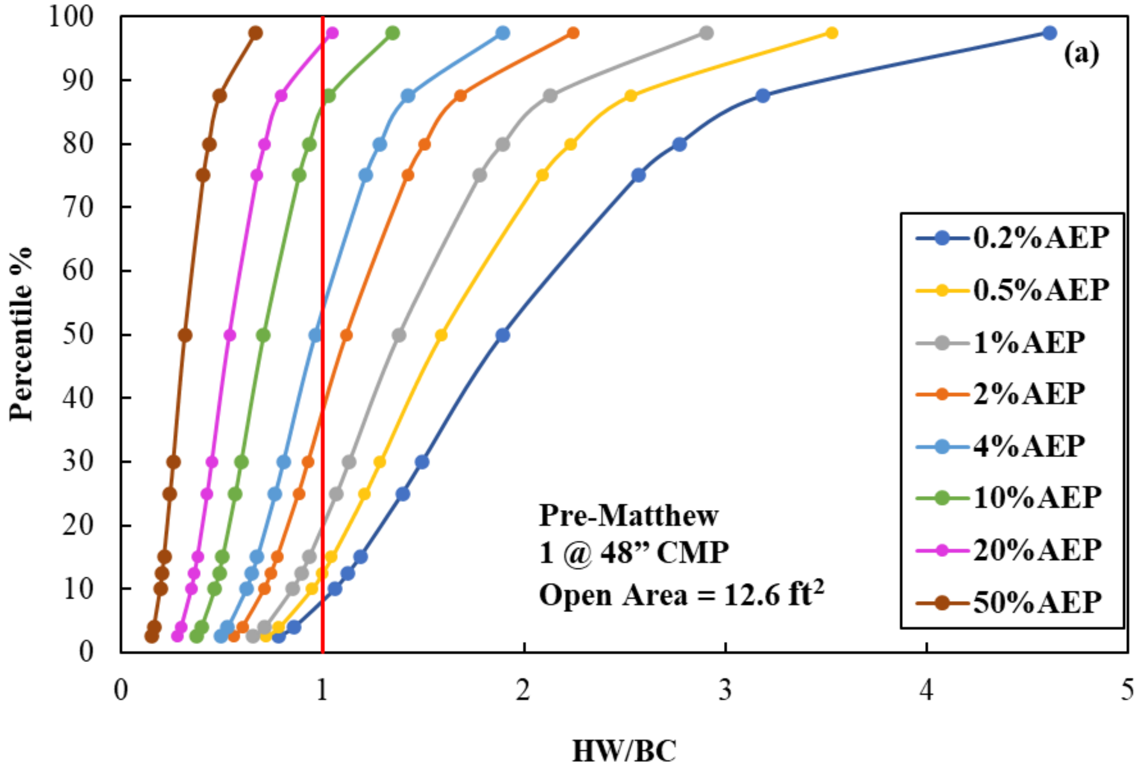


Figure D.7. Probabilistic analysis of overtopping flow values across percentiles for Site 42-3-MF: (a) pre-Matthew structure and (b) post-Matthew structure.



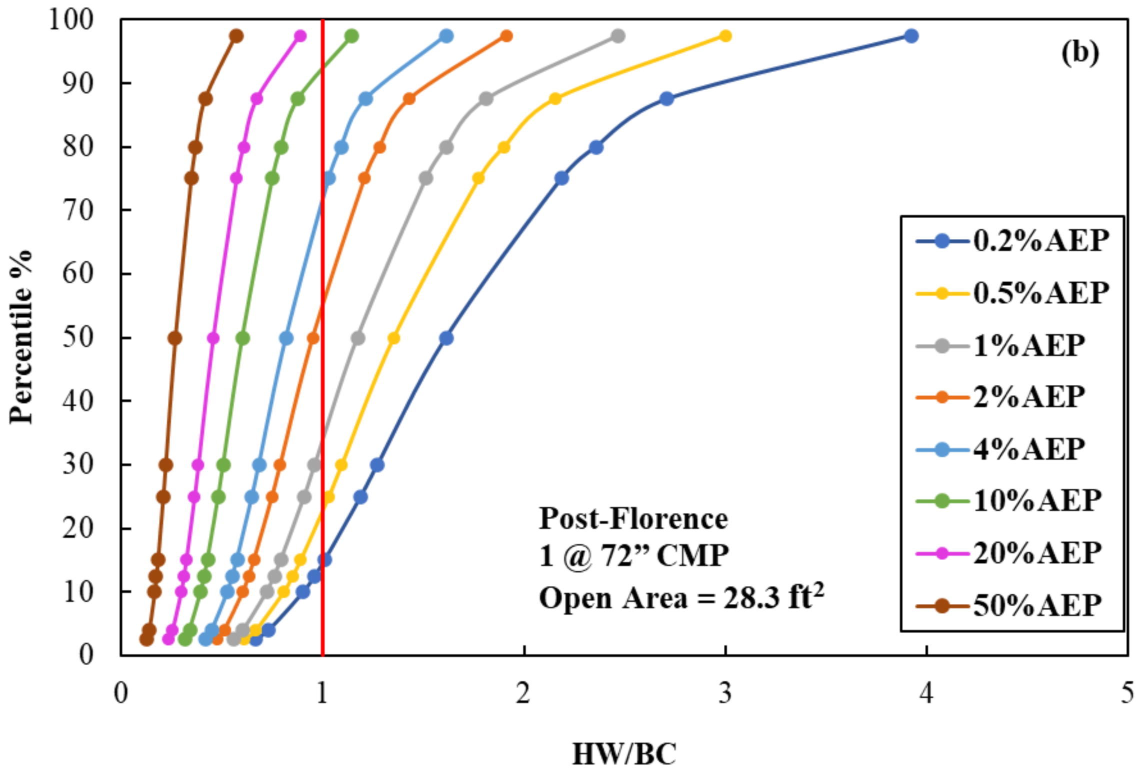


Figure D.8. Probabilistic analysis of overtopping flow values across percentiles for Site 53-5-MF: (a) pre-Matthew structure and (b) post-Florence structure.

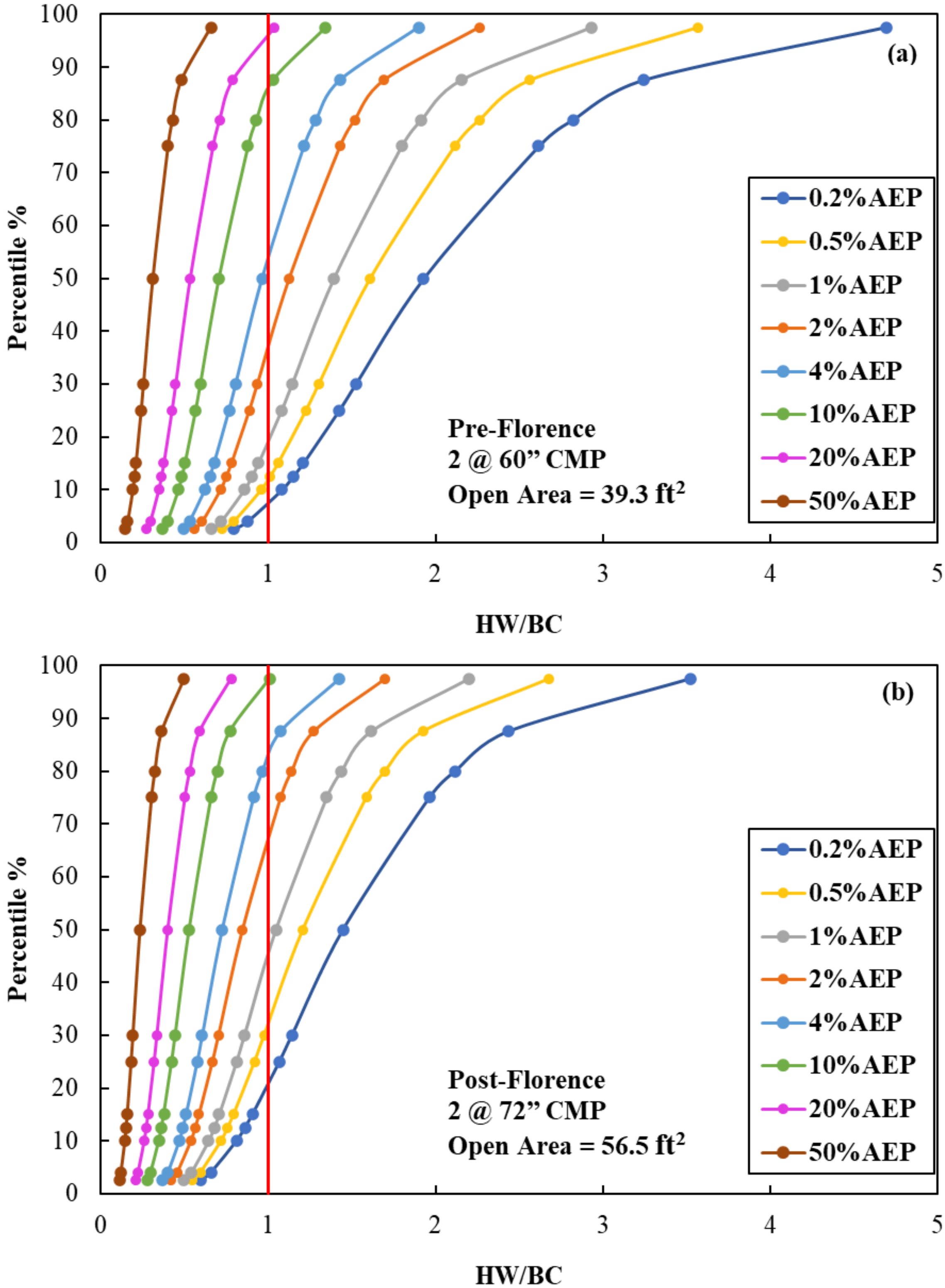


Figure D.9. Probabilistic analysis of overtopping flow values across percentiles for Site 25-2-F: (a) pre-Florence structure and (b) post-Florence structure.

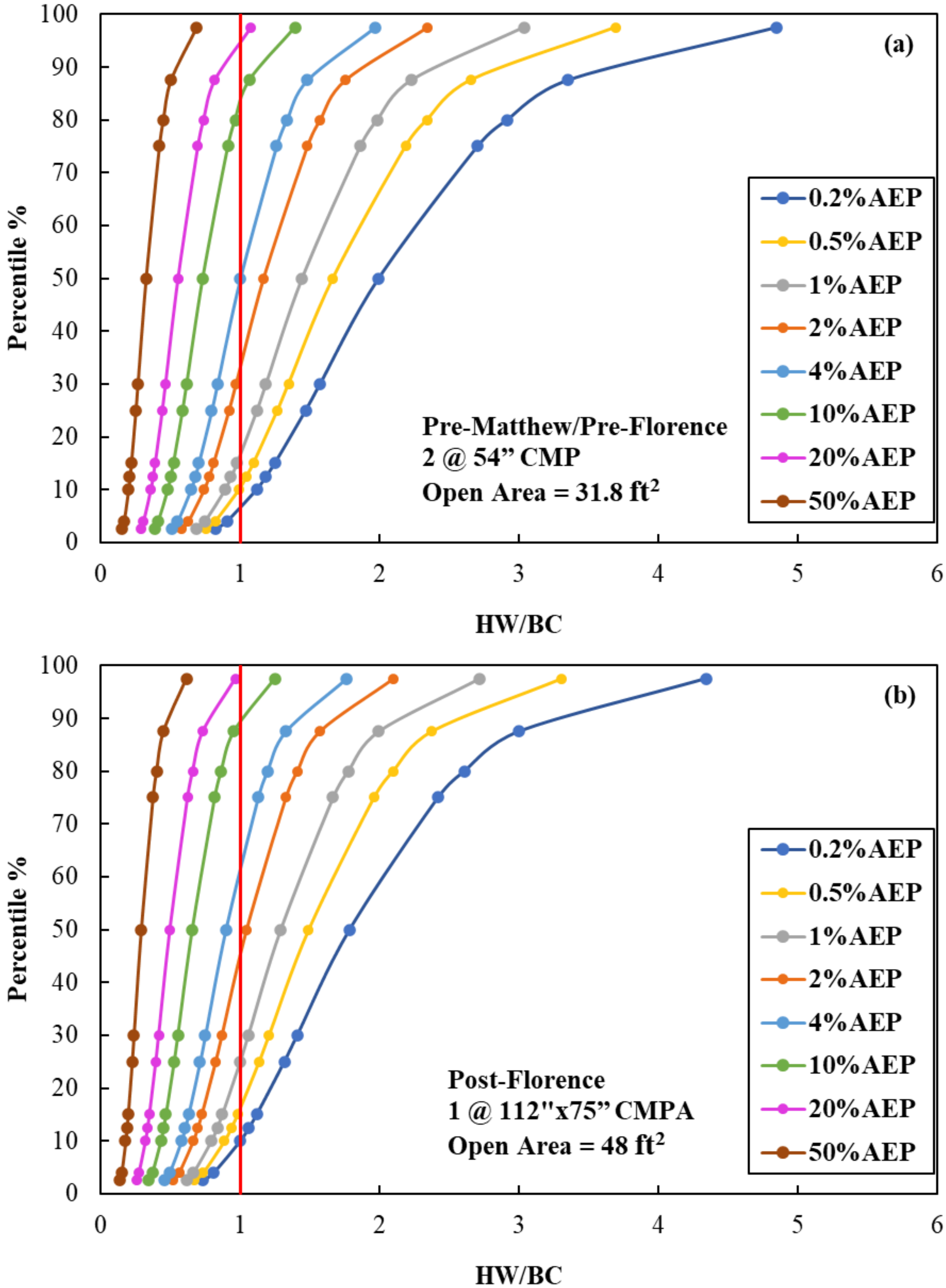


Figure D.10. Probabilistic analysis of overtopping flow values across percentiles for Site 95-2-MF: (a) pre-Matthew/pre-Florence structure and (b) post-Florence structure.

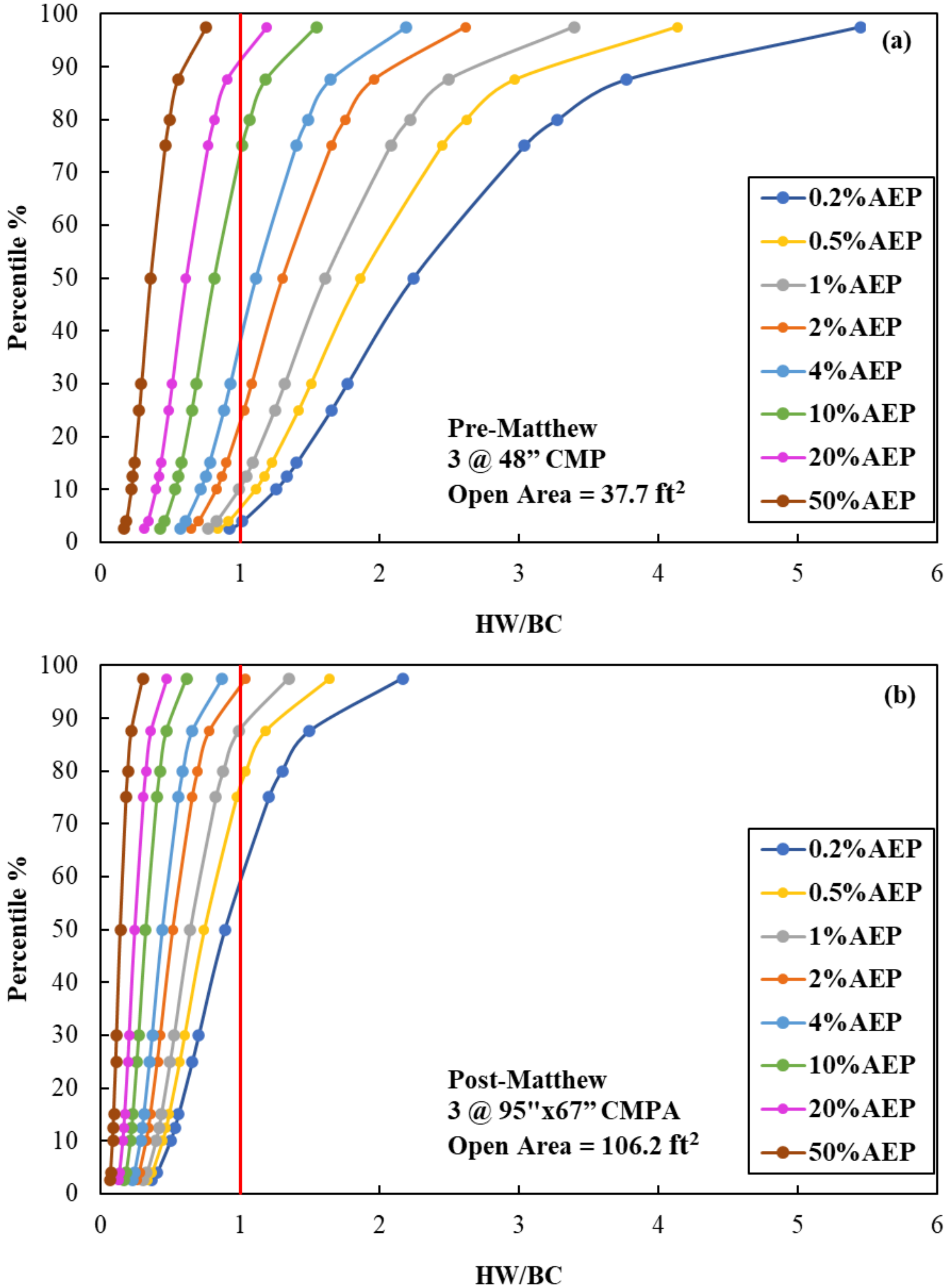


Figure D.11. Probabilistic analysis of overtopping flow values across percentiles for Site 42-1-MF: (a) pre-Matthew structure and (b) post-Matthew structure.

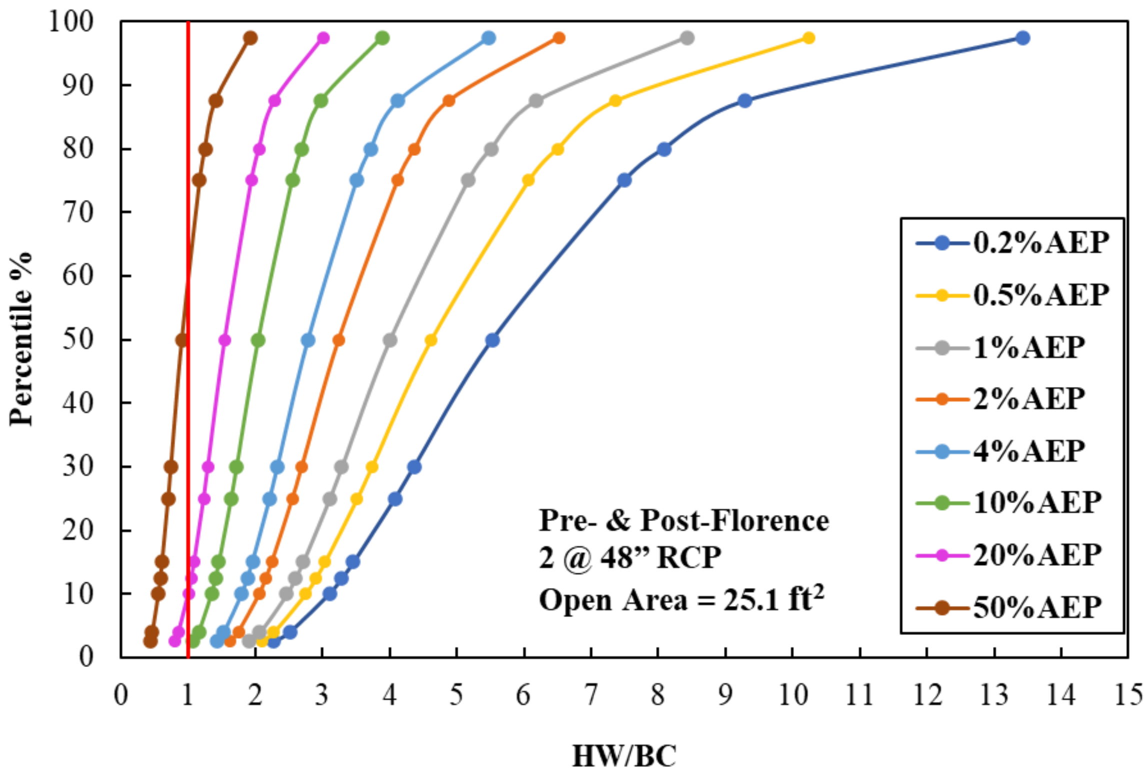


Figure D.12. Probabilistic analysis of overtopping flow values across percentiles for Site 8-4-MF pre and post-Florence.

Damage Level Correlations

Other potential failure pathways explored to enhance the understanding of potential correlations, and hence enable better design and maintenance recommendations to mitigate the risk of failures and optimize culvert performance were expressed in graphical format as shown in the figures below. Recall that the damage level descriptions are given in Table 6. The table also summarizes the proportion of sites experiencing pavement and/or shoulder damage by severity level. The scatter plots represented the culvert types as follows: reinforced concrete pipes (RCP) by blue dots, corrugated metal pipes (CMP) by red circles, corrugated metal pipe arches (CMPA) by green triangles, reinforced concrete box culverts (RCBC) by pink boxes, and High-density polyethylene (HDPE) culverts by blue stars.

Correlation between Damage level and Head Water to Bed-to-Crown ratio (HW/BC)

In 35 of the 73 cases considered, overtopping (HW/BC $\geq$ 1) was estimated to occur during the design event: 12 RCPs, 19 CMPs, three CMPAs, and one HDPE culverts. Figure D.13 shows the relationship of the calculated HW/BC against each damage level category. As discussed earlier, and shown in Table 19, 61% of overtopped cases had a damage level of 2 for pipe damage, 58% had a damage level of 1 for pavement damage, and 40% had a damage level of 3 for shoulder damage. These findings suggest that calculated overtopping potential is a weak indicator of potential damage during a storm, particularly for pipe and pavement damage.

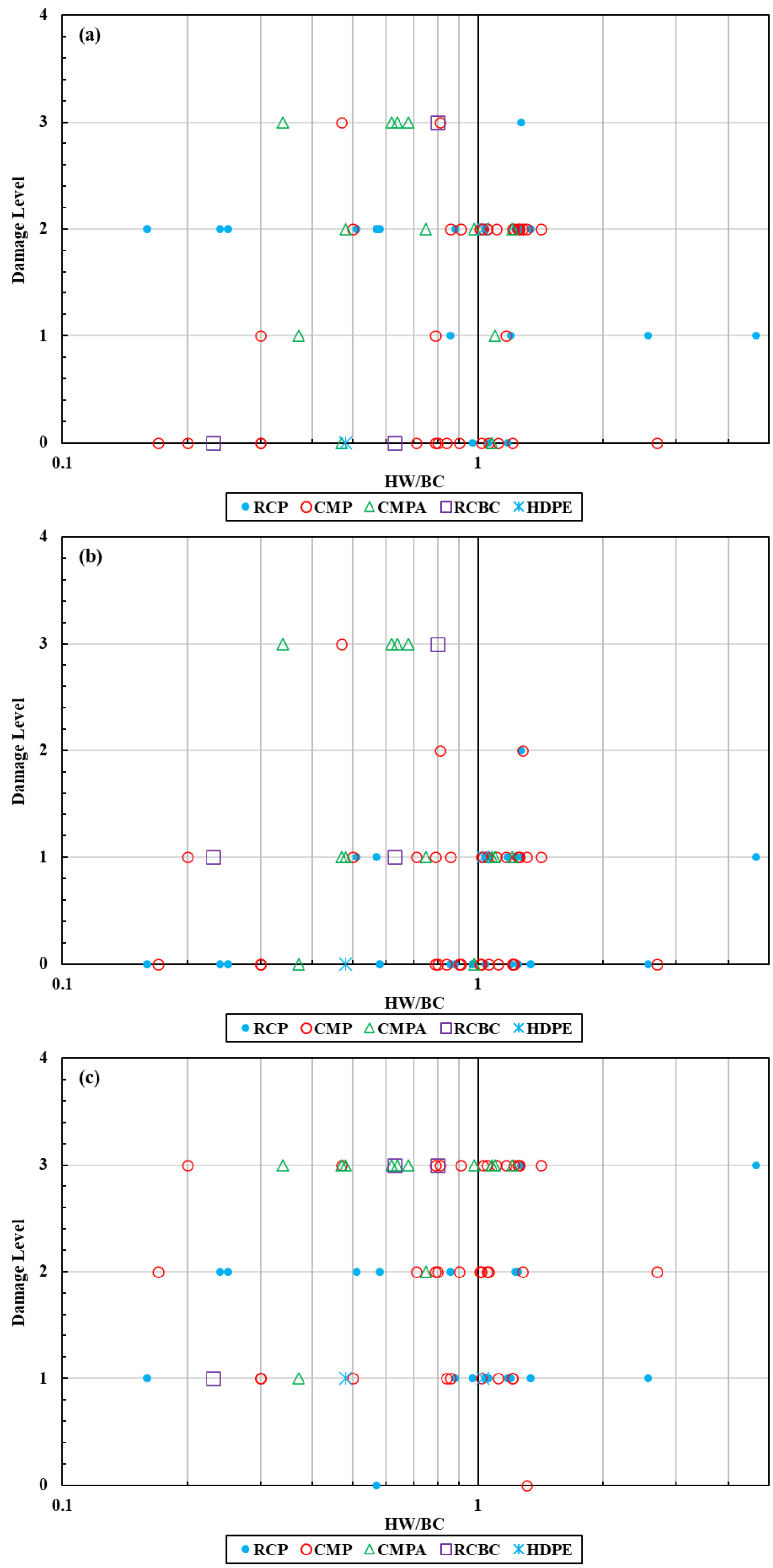


Figure D.13. Correlation between Damage Level and HW/BC for categories: (a) pipe damage, (b) pavement damage and (c) shoulder damage.

Correlation between Damage level and Drainage Area (DA)

To interpret the results of this analysis, it should be recognized that design methods change depending on the drainage area. As earlier stated, for drainage areas of 0 to 0.1 square miles, the Rational Method is used. For drainage areas between 0.1 and 1 square mile, USGS Urban and Small Rural (2014) is used, while all other drainage areas (>1 square mile) use USGS Rural (2009). Figure D.14 shows the results of this analysis and in this figure the delineations between design methods can be easily observed by noting that the black vertical lines along the x-axis are at drainage areas of 0.1 and 1 square miles. For this analysis, the initial expectations were that there would be a potential for higher damage levels with increases in drainage. As shown in Figure D.14 as well as Table 20, results suggest that the relationship between higher levels of damage and increasing drainage areas is a weak indicator of potential damage during a storm for shoulder, pavement, and pipe damage.

Correlation between Damage level and Bed-to-Crown (BC)

Contrary to expectations, the correlation between Damage level and Bed-to-Crown (BC), as depicted in Figure D.15, did not manifest elevated damage levels at shorter BC values. Consequently, the current analysis may not be deemed reliable for the purpose of assessing culvert resilience under storm conditions.

Correlation between Damage level and Backfill (difference between Bed-to-Crown and Pipe Diameter (BC-PD)

Figure D.16 showing the relationship between damage level and height of backfill showed no significant impact of the amount of backfill (difference between Bed-to-Crown and Pipe Diameter, BC-PD) to culvert performance in terms of either pipe, pavement, or shoulder damage.

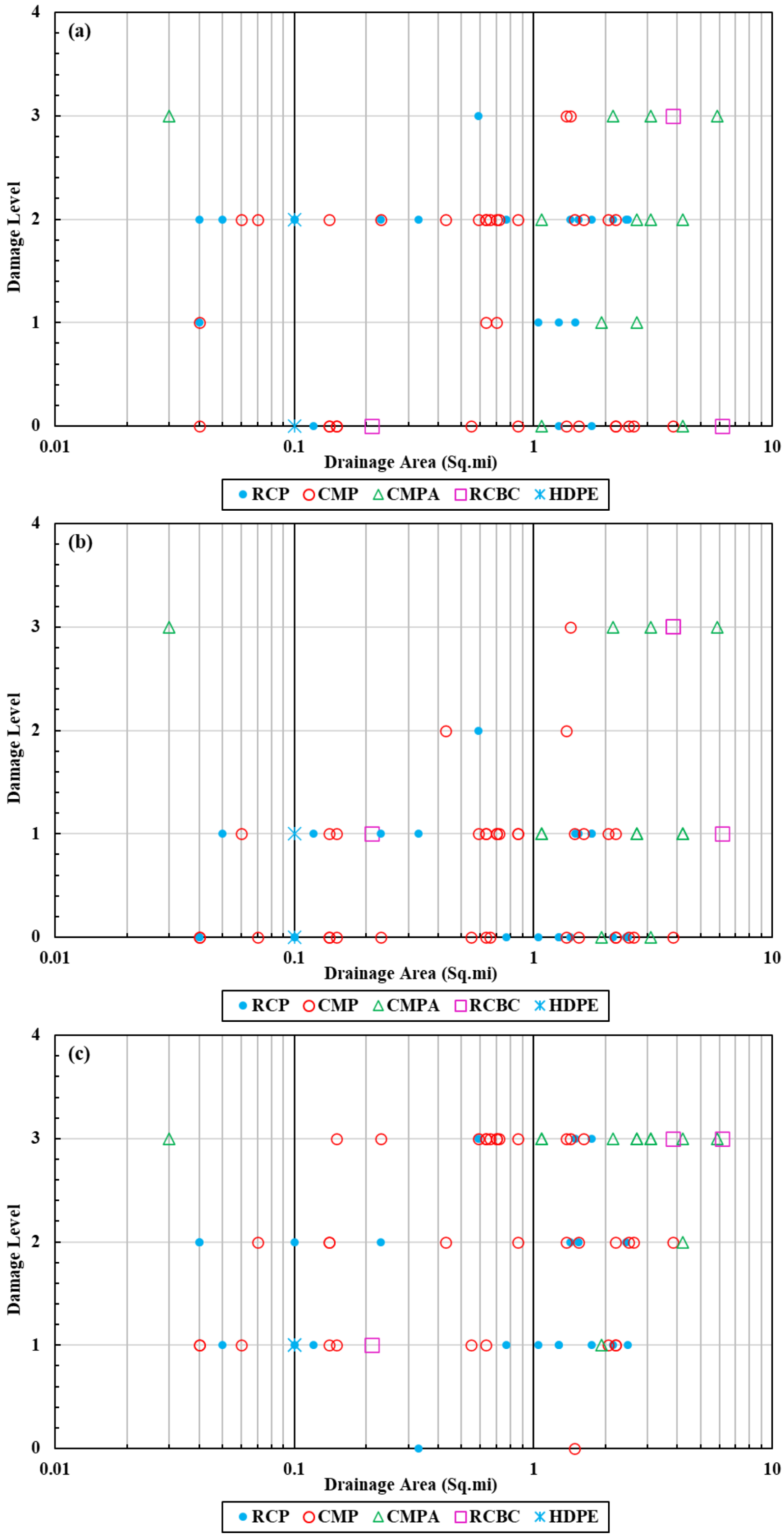


Figure D.14. Correlation between Damage Level and Drainage Area for categories: (a) pipe damage, (b) pavement damage and (c) shoulder damage.

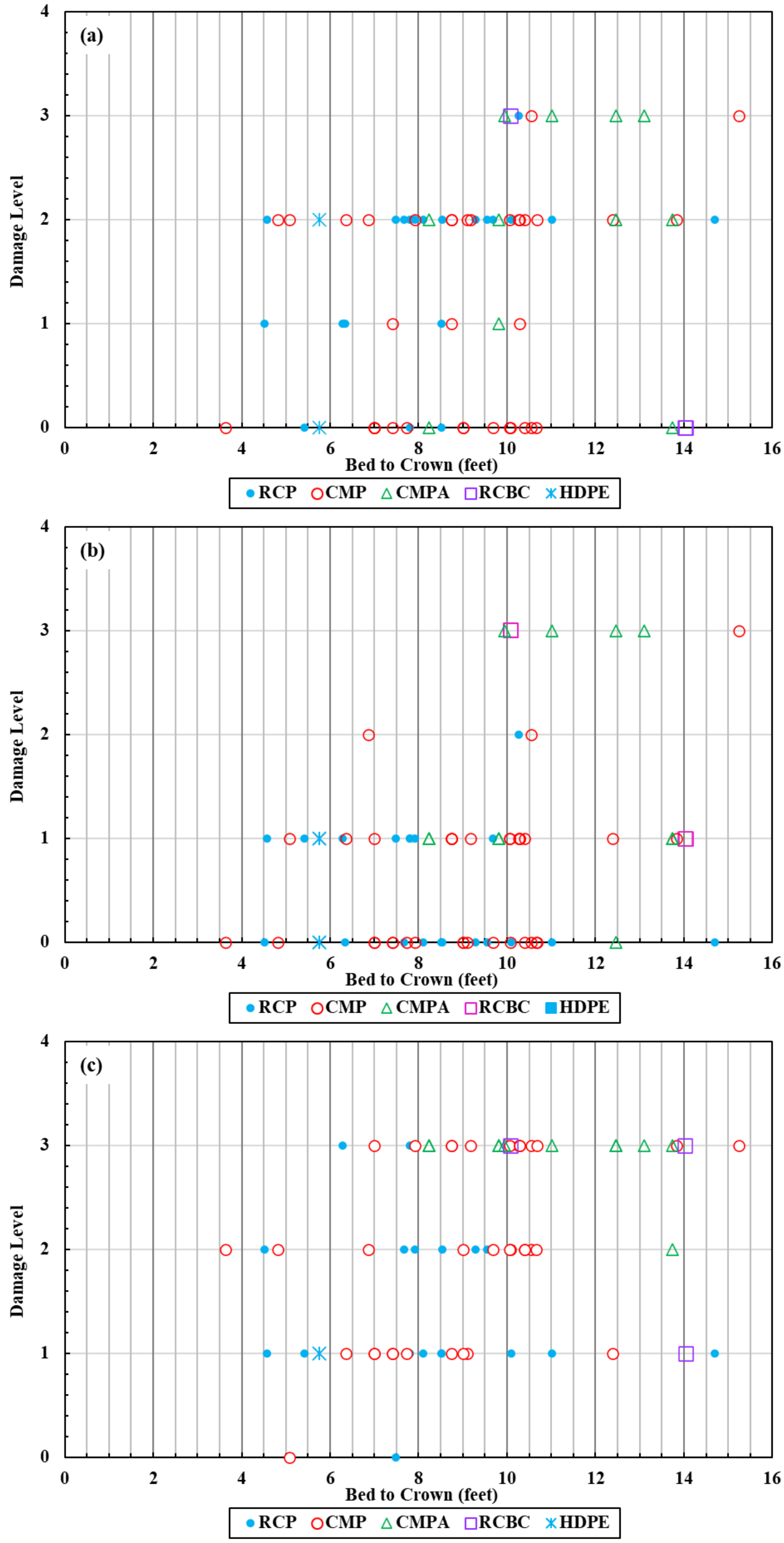


Figure D.15. Correlation between Damage Level and Bed to Crown for categories: (a) pipe damage, (b) pavement damage and (c) shoulder damage.

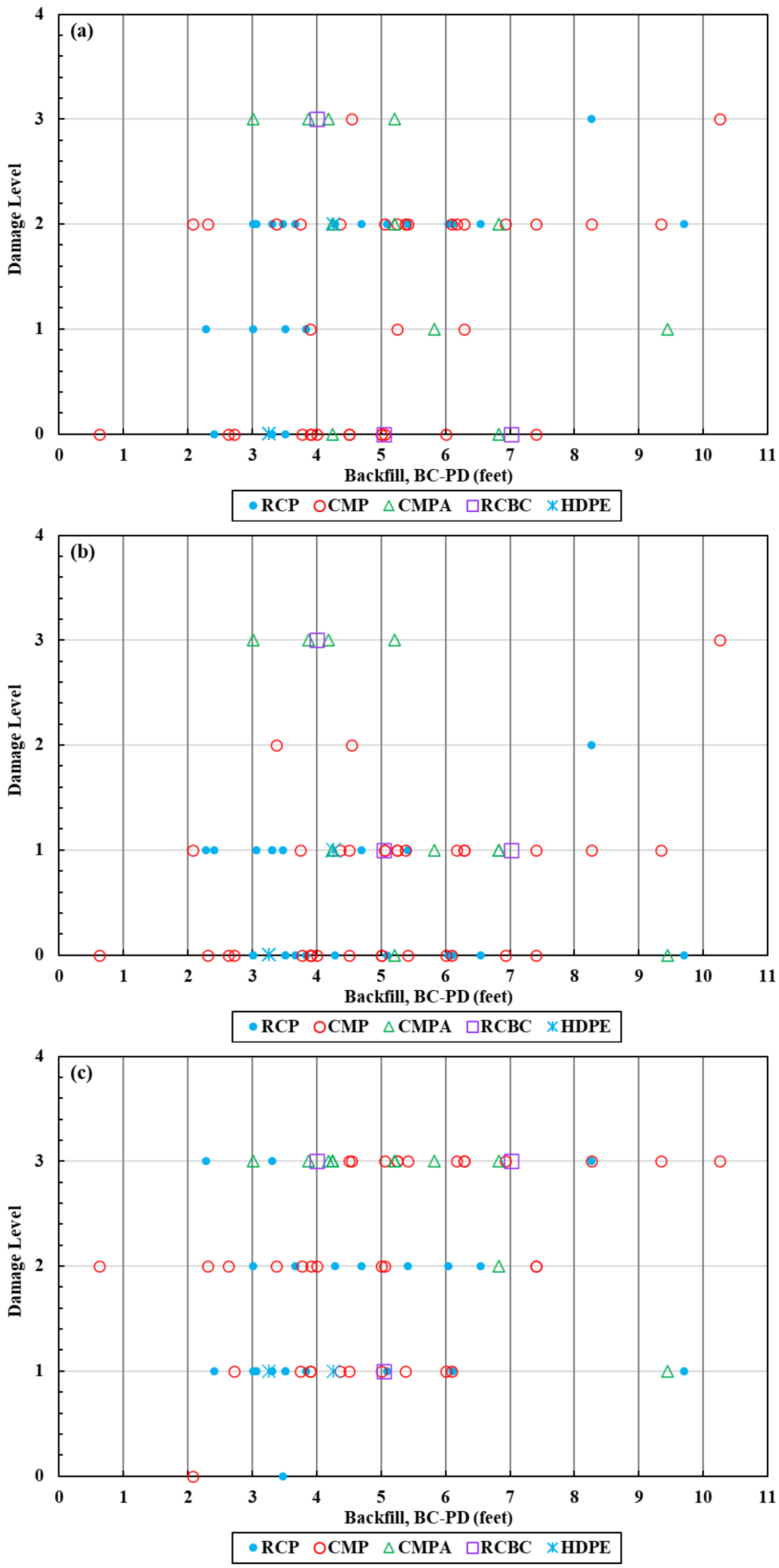


Figure D.16. Correlation between Damage Level and Bed to Crown for categories: (a) pipe damage, (b) pavement damage and (c) shoulder damage.

APPENDIX E: HYDRAULIC ANALYSIS RESULTS

Summary of Findings from Meeting with Division 4 (Wayne County)

In some of photos from damaged locations it was noticed that a layer of aggregate was placed on top of damaged areas. The reason for these practices was asked from division engineers. They responded that it is one of the practices in this division that ABC aggregate layer is placed on the damaged location in order to stabilize the location as a temporary repair until further substantial repairs are done. In some cases, in Wayne County washouts behind the headwall was observed. To better understand this type of failure mechanism common practice for backfilling was discussed. The engineers explained that the common practice for backfilling is that they place No. 57 Stone as bedding up to approximately half the pipe diameter, place ABC on top of the stone, and then place topsoil. The ABC is placed in 6–12-inch lifts and is compacted to 100% relative density. They also clarified that the washout behind the headwall is not a significant concern if the pipe is still in place and intact.

In terms of pipe replacement decisions, in some cases (such as 95-6-MF after Matthew), the shoulder and roadway look intact, but the pipe underneath is separated. The engineers clarified that in these cases, it may be decided that the pipe should be replaced. In other cases, such as 95-6-MF after Florence, even though the area behind the headwall is washed away, it may not be justified to replace the pipe. Due to challenges of installation, CMP pipes are the more preferred option compared to RCP pipes. This preference is especially true for larger RCP pipes. It is getting more difficult and time consuming to acquire RCP pipes. The demand for PE pipes is getting higher.

In many cases, evidence of continued erosion was observed and the possible strategies for mitigating this issue was discussed. The engineers emphasized that the monitoring of the sites needs to be increased to keep track of the damages and erosion to prevent severe damages after hurricanes. There is a new process being implemented where surveys are being done with higher frequencies in order to better monitor the sites. Further, they also explain that one of the contributing factors to erosion is that utility lines may cut over or under existing pipes.

In one of the cases the unusual placement of concrete headwall was discussed. The engineers explained that the concrete headwalls that were not embedded in the soil, usually seen in structures that were placed 40-50 years ago, put a lot of weight on the joints, which can cause gaps.

Summary of Findings from Meeting with Division 2 (Lenoir County)

In terms of pipe replacement decisions, engineers explained that in some cases (e.g., 53-1-MF after Matthew and Florence), even though the shoulder and part of roadway is washed away, it was not justified to replace the pipe. It was explained that if the pipe size is above 48 in. the bridge department is responsible for maintaining it. It is hard to say whether a lot of sites that were damaged in Matthew were also damaged in Florence. However, it can be said that a lot of pipes were upsized after Matthew and there were a handful that blew out again.

The common practice for backfilling in this division was discussed. Engineers explained that the common practice for backfilling since 2000 is that they place No. 57 Stone as bedding. The common practice for backfilling is that they place No. 57 Stone as bedding until the top of the pipe, place a geosynthetic on top of the stone, place ABC on top of the geosynthetic, and then place topsoil. This practice is believed to have been in place since approximately 2006.

In many cases, evidence of continued erosion was observed and the possible strategies for mitigating this issue was discussed. The engineers pointed out that in some cases (e.g., site 53-1-MF); the issue is that the water is coming from side ditches, and it is hard to place a ditch near the blue-line. As a result, the site has multiple shoulder washouts. An erosion or damage cannot be detected until it becomes a problem and affects the road or traffic because preemptive surveys are not commonly done. However, frequent monitoring is needed, and the division is trying to create an inventory for their existing pipes. Usually, the information for damage might come from farmers and others that call and report problems. Depending on the time of year, typically it takes 30 to 90 days for the vegetation to be established after repair. If the vegetation is not established, it makes the site susceptible to washouts. In order to reduce erodibility, compaction, matting and armoring shoulder with Class B stone is useful. One of the contributing factors to erosion is that utility lines cut through the pipes and cause failures that cannot be detected.

The research team asked about the practice on using ABC-M. The engineers explained that in this county, ABC-M is no longer used for shoulder repairs because there were complaints about it wrecking lawn mowers and cars. It was decided not to use this material anymore and instead add organic material to pulled shoulder soil to help with vegetation growth.

In some cases, the proposed/designed pipe were different from the pipes that that were actually in place, and the research team asked about the possible reasons for this inconsistency. The engineers clarified that in some cases, issues with the availability of the pipe, utilities, or the availability of cover might cause restrictions on placing the designed pipe.

Given the sites considered in this research study, it was suggested that the research team investigate one site on NC 903 in more depth.

Summary of Findings from Meeting with Division 6 (Bladen County)

In some cases, complete washouts were observed, and the possible contributing factors were discussed. The engineers explained that one of the contributing features is when the dam breaks on the upstream of the pipe and cause the washout. The common practice for backfilling in this division was discussed. Engineers explained that they place No. 57 Stone as bedding until the top of the pipe, then place ABC on top of the stone, and then place topsoil.

In terms of pipe replacement decisions, engineers explained that due to installation challenges, CMP pipes are preferred when comparing to RCP pipes. Typically, RCP are used for smaller pipes, and CMP are used for bigger pipes because of headwall and cover. Usually, the division uses extended rip rap to protect the inlet side for smaller pipes. For pipes above 48 inches, Aluminum headwalls are used and for pipes below 48 inches, rip rap is used on the inlet, but not the outlet side. After Hurricane Matthew they started to use it on the outlet side too. It is common practice that often when it is necessary to replace a crossline pipe, it is upsized by one increment regardless of the condition.

In many cases, evidence of continued erosion was observed and the possible strategies for mitigating this issue was discussed. The engineers pointed out that if the slope is not stabilized after repair, then the site is more susceptible to shoulder washouts. In some cases (e.g., 8-1-F) the damage was due to erosion not the direct impact of the channel flow. In some cases, the cause of erosion is that the pipe is too short (i.e., the end terminates close to the edge of the pavement) and the roadway embankment is not stabilized. The solution is to expand the pipe, but sometimes it is not possible due to environmental and right of way reasons. When the top layer of shoulder is eroded, the underlying stone stays in place so soil can be placed on top in relatively short order. Thus, these types of failures are not a concern. Better matting and establishment of the soil helps mitigate these issues. In order to mitigate the erosion of the edge of pavement due to joint issues, the pipe joints need to be repositioned so that they do not align with the edge of the pavement. In some cases, the issue with the loss of rip rap is due to the steep slopes. Extended pipe or placing a headwall can be helpful.

The possible contributing factors in the structure failures/damages were discussed. It was pointed out that usually hurricanes do not impact smaller pipes because they are not located in a big watershed. Medium to large pipes usually get damaged. If the toe wall of the headwall is not deep enough, it might cause undermining and settlement of the structure.

In one of the cases, only a pothole on the asphalt surface was observed in the damage photos. This type of failure was discussed with the engineers. They explained that a pothole appearing on the asphalt surface is evidence of issues with the pipe joints.

The research team asked about the impact of using No. 57 Stone. The engineers explained that it is fair to assume that using No. 57 Stone can prevent continued erosion into the broken joints since it would be larger than the gap in the joints. Since 2005 they had started using No. 57 Stone, but it is hard to confirm whether use of No. 57 Stone mitigates the damage.

Based on the engineers' experiences, CMP pipes had issues with acidic soil (or water), which causes rusting problems with the CMP band. In this case, it was decided to use less CMP and use more stainless Aluminum pipes since (approximately) 2004. It was also mentioned that there should be no CMP pipes east of I-95 because of these issues.

With respect to monitoring sites, engineers used to monitor weekly after a repair. They tend to monitor hurricane sites (repaired under FEMA or under contracts) more frequently than other sites. However, in all cases the belief is that more frequent monitoring is needed. An erosion or damage cannot be detected until it becomes a problem and affects the road or traffic because preemptive surveys are not commonly done.

Engineers pointed out that there were a lot of locations where the roadside slopes were washed away after Matthew, so it was decided to flatten the slopes (where possible) and/or install gutters to control the floodwater. These strategies seemed to have helped because the division noted no issues after Florence.