PRIORITIZING LEVEE IMPROVEMENTS

A Thesis

Submitted to the Faculty

of

Purdue University

by

Brian J Meunier

In Partial Fulfillment of the

Requirements for the Degree

of

Master of Science in Civil Engineering

December 2011

Purdue University

West Lafayette, Indiana

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

Page

ABSTRACT

Meunier, Brian J. M.S.C.E., Purdue University, December 2011. Prioritizing Levee Improvements. Major Professor: Venkatesh Merwade.

Levees exist all over the United States, which protect land and property from devastating floods. Many of these levees are more than half of a century old, and were initially intended to serve as protection for farmland; however, increases in development and urban sprawl have caused a rise in the number of homes being sheltered by levees that were not designed with the necessary level of protection. A lack of inclusive record keeping and inspection has left many levees in dire need of costly repairs. This study attempts to define a practical and economical means of prioritizing levee repairs based on the economic risk posed by the breaching of impaired levees and the expected improvement costs for returning the levees to a safer condition. A framework for a simplified breach damage analysis is proposed through a case study of five levees in a flood-prone area in central Indiana. Current analysis methods are examined and compared to the proposed methodology.

Results of the case study provide a means of analytically prioritizing levee repairs, reveal pitfalls of the current standards of practice, and identify future research needs for advancement of the prioritization procedure. The use of an unsteady-flow analysis with storage areas to represent the protected areas is identified as a key component to a realistic characterization of the physical system. Comparisons between breach results, economic costs, and characteristics of the protected areas reveal no apparent correlations, suggesting a need for a ranking parameter. A Priority Ratio is identified in the case study results and suggested for use.

CHAPTER 1 INTRODUCTION

Catastrophes have thrust the topic of flood control infrastructure into the national spotlight in the recent years. Levees in New Orleans were breached during Hurricane Katrina, leaving citizens homeless, bereaved, and helpless in 2005. Midwestern America became a national disaster site in the summer of 2008 as levees and dams were damaged and destroyed by relentless, widespread rainfall.

Levees exist all over the United States, which protect land and property from devastating floods. These levees provide a vital service in the form of preservation of human life as well as maintaining the value of the homes that lie in the protected area. Critical components of infrastructure and industrial sites are also often located adjacent to streams and rivers due a reliance on connectivity to a large source of water, requiring levees to prevent crippling damage to the facilities.

Though this infrastructure goes unnoticed or unrecognized by much of the population, approximately 43 percent of the United States population lives in one of the 692 counties that contain levees. The United States Army Corps of Engineers (USACE) estimates that some 100,000 miles of levee exist in the United States. The vast majority, around 86 percent, is locally owned and maintained (USACE, 2006). Local ownership and maintenance has allowed the condition of private levees to remain unknown by governing bodies. Many of these levees are more than half of a century old and were initially intended to serve as protection for farmland; however, increases in development and urban sprawl have caused a rise in the number of homes being sheltered by levees that were not designed with the necessary level of protection. Many Americans are unaware of the dangers of living in flood protected areas. The absence of mandatory flood insurance seems to convey a sense of safety, or lack of risk; however, surveys of the flood protection infrastructure of the United States have revealed serious flaws in this rationale (USACE, 2006).

A lack of inclusive record keeping and inspection has left many levees in dire need of repairs. As of 2009, only 10 states retained any listing of the levees within their borders. Perhaps more shockingly, a mere 23 states have a form of oversight on levee safety. (ASCE, 2009) The combination of these two factors can allow for the unchecked degradation of these critical components of infrastructure. USACE's current inventory of federally inspected levees states that 9% of the 1,967 levees listed are expected to fail during a significant flooding event (ASCE, 2009). The increased development density behind levees, coupled with declining levee conditions has the potential for devastating loss of human life, destruction of personal and public property, as well as severe damage to other important infrastructure. Levee failures resulting from Hurricane Katrina and the Midwest Flood in 2005 and 2008, respectively, led to 1,834 deaths and an estimated economic damage of more than $200 billion (NCLS, 2009). These levee failures can result from deficient levees; however, the failures can also stem from inadequate design. After the adoption of the National Flood Insurance Program in 1968, many levees were designed to provide adequate protection for the 1% annual chance flooding event to exclude the owners in the protected areas from having to purchase flood insurance. Though the 1% annual chance flood was never intended for use as design criteria (NCLS, 2009), the economical incentives to construct levees to the minimum elevations required to eliminate mandatory flood insurance have forced a Spartan approach to levee construction.

In recognition of the pitfalls in levee safety and oversight, the National Committee on Levee Safety (NCLS) has issued evaluations of the current system. America's levee infrastructure was given a "D-" in the American Society of Civil Engineers' "Report Card for America's Infrastructure," citing that the potential for loss cannot be overlooked (ASCE, 2009). In 2009, the NCLS submitted a report to Congress with numerous recommendations for a National Levee Safety Program. Among the recommendations listed was the establishment of a hazard potential classification system (NCLS, 2009). A set of criteria used for a more holistic assessment of risk, beyond the probability of occurrence, have yet to be formally developed.

The overabundance of levees existing in poor condition creates an economic issue, in addition to the obvious safety concerns. It is not feasible, nor practical, for all levees to be repaired and upgraded to meet the requirements set forth in the National Flood Insurance Program under

Title 44 Code of Federal Regulation 65.10 (44 CFR 65.10). The extent of the deficient structures as well as the expense involved in rehabilitating flood control infrastructure will simply not allow the repair of all structures. As a result, it is necessary to determine which levees to upgrade and maintain in a responsible manner. Currently, there is no generally accepted method for prioritizing levee repair or method for determining which levees should receive no additional attention.

This study attempts to define a practical and economical means of prioritizing levee repairs based on the economic risk posed by the breaching of impaired levees and the expected improvement costs for returning the levees to a safer condition. A framework for a simplified breach damage analysis is proposed through a case study of five levees in a flood-prone area in central Indiana. Suggestions for advancement of the proposed method as well as future research needs are explored.

CHAPTER 2 STUDY AREA BACKGROUND INFORMATION

In the United States, the topic of flooding is most often associated with the Mississippi River and coastal regions subject to hurricane seasons. Flooding is not secluded to these regions. The state of Indiana may not be the first place that comes to many Americans' minds when they think of flooding; however, Indiana has a long history of devastating floods. Even in more recent times, catastrophic floods have created disaster areas out of much of the state. Since 2006, there have been six flooding events severe enough for the government to declare affected areas as federal disaster area. Ninety percent of Indiana's 92 counties were declared federal disaster areas in 2005 after heavy rainfall occurred in saturated watersheds. A total of $7 million in flood insurance claims were paid. Extended periods of significant rainfall culminated in a massive flood in June 2008. Rainfall exceeding the 1%-annual-chance event swept across the state leading to over $175 million of federal disaster assistance (FEMA, 2011). After inspecting the flooding history of Indiana, it is clear that the hazard of flooding is significant; however, the exposure to flooding is equally significant. There are approximately 32,500 flood insurance policies in effect statewide in Indiana, with approximately 22,000 of those policies covering properties in high risk areas (FEMA, 2011). The majority of all major floods within Indiana occur within the White, Wabash, and Ohio River basins. These rivers are relatively low energy rivers, which must swell greatly to convey large amounts of runoff.

The two sites examined in this study are located in Indianapolis, Indiana; a study area location map is shown in Figure 1. Indianapolis is split in half by the West Fork (WFK) White River and has a significant amount of flood control infrastructure to combat the frequent high stages of the river. Indianapolis, much like the rest of the state, is no stranger to flooding.

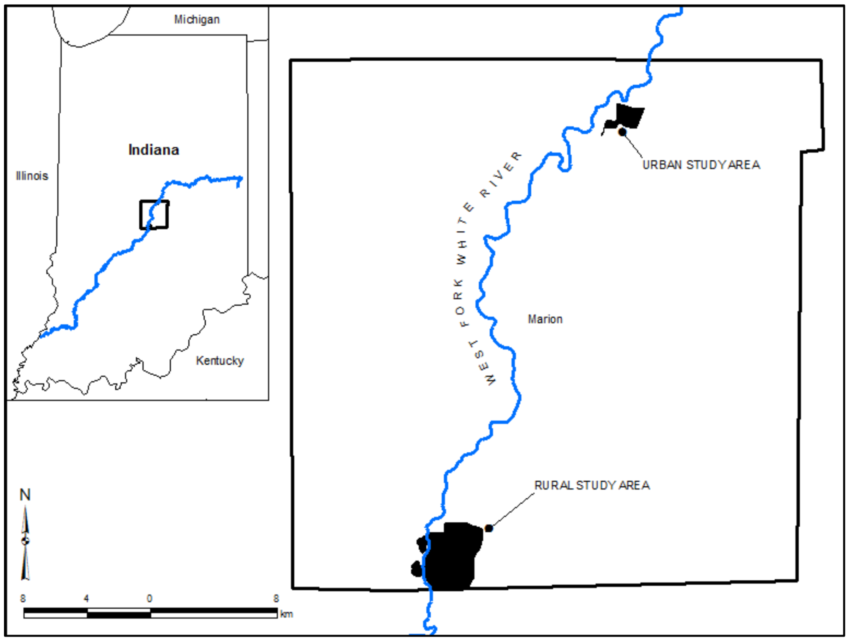

Figure 1: Study Area Location Map

2.1 Indianapolis Flood Control Infrastructure

As a result of previous floods and the obvious exposure to flood hazards, Indianapolis has developed an extensive network of levees and floodwalls to protect itself from the rising waters of the WFK White River and its tributaries. Extensive flood reduction and protection projects began to be constructed in the 1920's, continuing through the 1960's. Reservoirs, major diversions, and detention basins were built to increase storage, and to reduce peak channel flow rates. Earthen levees and floodwalls were also constructed to reduce the remaining risk of flooding (Bodenhamer & Barrows, 1994). The levees protect urban and rural areas in and around the city. Nearly 39 miles of levees exist in the city and surrounding areas in a system of 48 levees; however, the Federal Emergency Management Agency (FEMA) only recognizes 32 of the levees, a total of 29 miles, as providing the 1%-annual-chance level of flood protection. As many of the levees were initially constructed nearly 100 years, the integrity of the structures has deteriorated over time leaving them in a poor condition. Inspections performed by engineering consultants suggest that 13 of the levees are in poor condition and are in need of significant repairs (CBBEL, 2007). Based on the findings of the study, the flood control infrastructure of Indianapolis is showing evidence of significant aging.

2.2 Study Reaches

Two reaches of the WFK White River were selected to serve as study areas. The levees selected are different with respect to type of protection; one study area is primarily urban, while the other study area is dominated by agriculture. The urbanized area has a clear need for flood protection based on the number and types of structures being protected by the levee system. The agriculturally based levee system serves as protection for some households and other structures; however, the majority of the land area encompassed by the levee is open space or farm fields. Finally, the levee systems differ in the apparent necessity for repairs based on a visual inspection of deficiencies. The differing conditions of the protected areas and levee conditions are desirable to convey the variability that may be expected in results and the potential uses for the results of the analysis. In this instance, one would expect a levee that is mildly deficient to be assessed a higher priority status than a levee that is significantly more impaired if the former levee protected a much more populated area. The highly deficient levee with a lower apparent value in terms of protected structures and property was selected to display the need to consider abandonment of levees as opposed to rehabilitation. The areas chosen also afford the opportunity to analyze several levee differing in length, height, and flooding source. A more thorough description of each levee segment follows.

2.2.1 Urban Levee System: WR-C1 and HD-C1

The urban levee system analyzed in this study is slightly north of Broad Ripple Village, a cultural district in the north-central portion of Indianapolis, Indiana. The area is primarily residential and serves as home to one school. A small amount of commercial development also exists in the area. The contributing watershed has a total area of 3,027 square kilometers (km^2) and is primarily agricultural, with 75.0% of the land being used for that purpose. Of the remaining area, 18.3% is urbanized, 5.3% is forested, and 1.4% covered by open water (USGS, 2010). The protected area is exposed to flood hazards from two different sources, WFK White River and

Howland Ditch. Howland Ditch is a tributary to the WFK White River, with the confluence of the streams being slightly south and west of the study area. Each flooding source has a separate levee segment which forms a line of protection. The levee providing protection from WFK White River, WR-C1, is an earthen berm with a paved roadway along the levee crest; the levee segment is approximately 640 meters in length. A second levee segment along Howland Ditch, HD-C1, spans a total of 1,006 meters, with approximately 520 meters of the total length being concrete floodwall. The remaining portion of the levee is earthen embankment. A schematic showing the location and orientation of the levees to the respective flooding sources is shown in Figure 2.

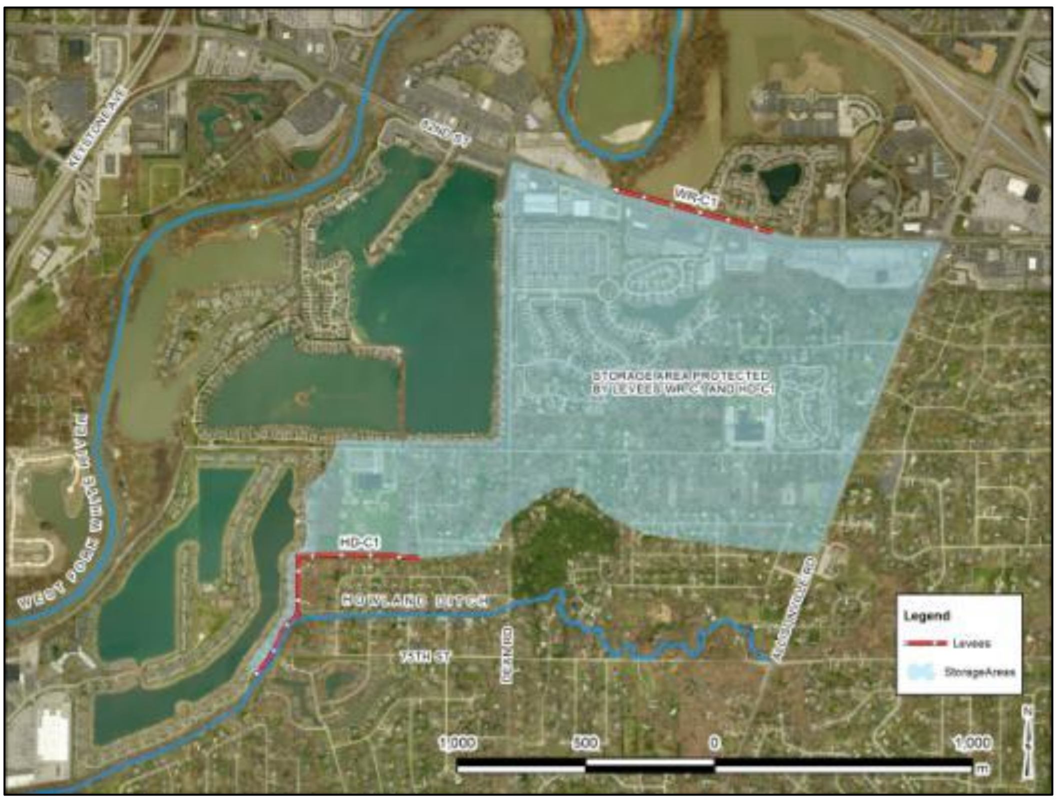

Figure 2: Urban Levee Study Area Located in Washington Township, Indianapolis, IN

2.2.2 Rural Levee System: WR-02, WR-03, and Unnamed Levee

The rural levee system analyzed is located on the southwest perimeter of Indianapolis, Indiana, as shown in Figure 2. The area is much more rural than the former study area; however, there are still several homes and businesses located within the protected area of the Unnamed Levee

(UNL). Levees WR-02 and WR-03 provide protection for a much smaller area which includes a very small number of buildings and a portion of a golf course. The contributing watershed has a total area of 4,885 km^2 and is also primarily agricultural, with 65.0% of the land being used for that purpose. Of the remaining area, 27.7% is urbanized, 5.6% is forested, and 1.6% covered by open water (USGS, 2010). The sole flooding source is WFK White River; WR-02 and WR-03 reside on the west bank of the river and UNL is to the immediate east of the river. All levee segments consist entirely of earthen berm. WR-02 runs along the west bank of the river for approximately 920 meters; WR-03 is slightly north of WR-02 and is nearly 1,000 meters in length. At nearly 21,000 meters in length, UNL is the longest levee. A schematic showing the location and orientation of the levees is shown in Figure 3.

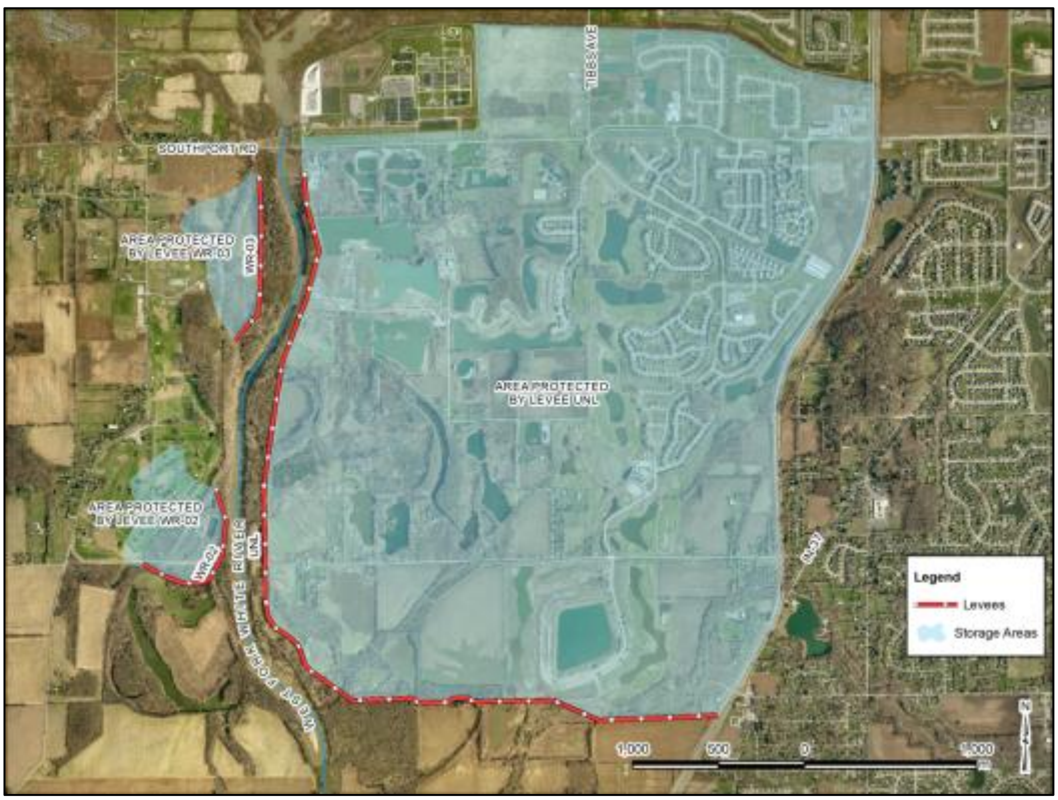

Figure 3: Rural Levee Study Area Located in Perry Township, Indianapolis, IN

CHAPTER 3 STUDY AREA LEVEE PRIORITIZATION METHODOLOGY

3.1 Hydrologic Model Development

Hydrologic models were developed for the urban and rural study reaches using standard hydrologic engineering practices and the United States Army Corps of Engineers' (USACE) HEC-GeoHMS software add-in for ArcGIS. Publicly available data sources were used for all study area datasets. Elevation data was gathered from the United States Geological Survey (USGS) from the Seamless Server (USGS, 2010). A Digital Elevation Model (DEM) having a resolution of one-third arc-second was downloaded for the region which would encompass both the urban and rural study areas. Land use information was also taken from the USGS Seamless Server; the National Land Cover Dataset grid had a cell size of one arc-second (USGS, 2006). Hydrologic soil properties were taken from the 1:24000 SSURGO dataset made available by the Natural Resource Conservation Service (NRCS, formerly the Soil Conservation Service, or SCS) (NRCS, 2010). Design rainfall data was collected from a National Oceanic and Atmospheric Administration (NOAA) Atlas 14 frequency estimate near the centroid of the delineated watershed. Each step in the hydrologic model development is discussed below.

3.1.1 Watershed and Stream Network Delineation

The contributing watersheds and stream networks were delineated for each study area. Once the extent of the entire watershed contributing to the study areas was determined, the stream network was based off of a threshold value of 4% determined by plotting the total stream network length versus the defining watershed area percentage; a plot of stream length versus contributing drainage area is shown in Appendix B. An additional stream branch was created to include Howland Ditch in the stream network delineation. The entire stream network developed for the study areas is shown in Figure 4. Subbasin boundaries were generated based on the stream network delineation, resulting in a total of 17 subbasins. The subbasin representing the drainage area for Howland Ditch was the smallest at 27.5 km^2; the largest subbasin was 827.2 km^2. Times of concentration were determined by using the longest spatial flowpath within each subbasin.

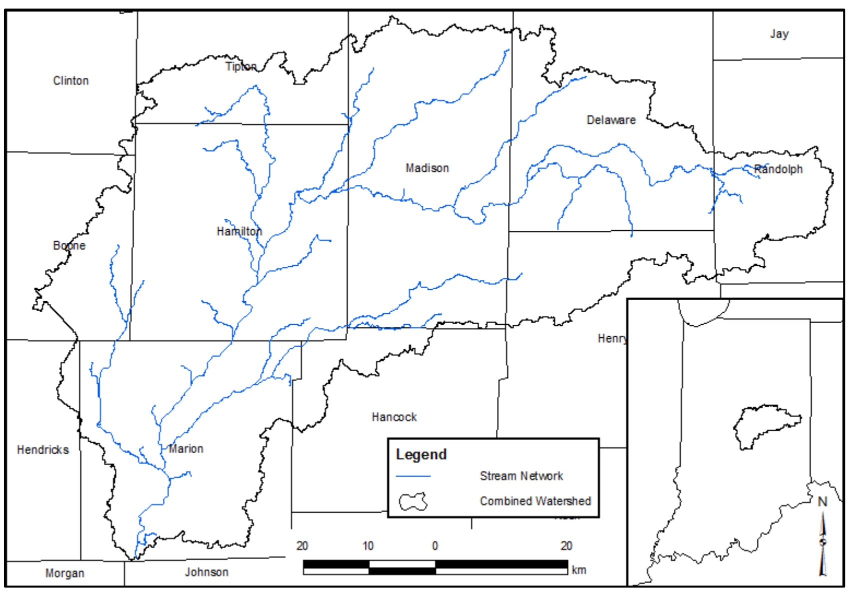

Figure 4: Combined Study Area Watershed and Stream Network

3.1.2 Curve Number Development

Runoff losses were modeled by using the SCS Curve Number Method. Soil type and land use grids were spatially joined to determine the runoff generating capability of each subbasin within the watershed. Hydrologic soil types were classified as A, B, C, or D, with all missing values being assigned hydrologic soil type B. Land use classifications were reclassified into four bins: water, medium residential, forest, and agricultural. Curve Numbers were then defined as suggested by Table 1.

Table 1: Curve Number Matrix (McCuen, 1998)

Gridcode	Land Use Description	Hydrologic Soil Type			
		A	B	C	D
1	Water	100	100	100	100
2	Medium Residential	57	75	81	86
3	Forest	30	58	71	78
4	Agricultural	67	77	83	87

3.1.3 Rainfall Simulation and Model Calibration

The 1%-annual-chance rainfall was simulated by applying the corresponding rainfall depth evenly across the entire watershed area. The corresponding rainfall depth was determined from the NOAA Atlas 14 rainfall frequency estimate for a point at the centroid of the watershed area. Rainfall was distributed temporally by using the SCS Type II rainfall distribution. Hydrologic routing within the Hydraulic Engineering Center Hydrologic Modeling System (HEC-HMS) program was performed using the Muskingum routing method. Typical x-values range between 0.1 and 0.5 (McCuen, 1998); a value of 0.2 was used. K-values were determined during calibration; a typical value of 6 hours was used. The 1%-annual-chance flow rate was determined for the WFK White River by fitting a Log-Pearson Type III distribution to the historical gage data measured by the USGS gaging station on the 82nd Street Bridge crossing near Nora, Indiana (Station 03351000). The initial peak flow estimate determined by the Log-Pearson Type III curve-fitting resulted in a flow rate below the 1913 flood event, which suggested the event exceeded the 1%-annual-chance event. As a result, the peak annual streamflow from 1913 was excluded from the distribution to prevent the calculated flow rate from being positively skewed. Muskingum K-values and SCS Curve Numbers were modified proportionally throughout the watershed to allow the simulated rainfall to produce a peak flow rate of similar magnitude and timing as that measured at the gaging station. After the peak of the hydrograph was properly calibrated, the flow hydrographs from the model nodes associated with the WFK White River near WR-C1, Howland Ditch near HD-C1, and WFK White River near the rural levees were recorded for use in the hydraulic model. Streamflow hydrographs used as input for the hydraulic model as well as a comparison of the observed peak annual streamflow data and the Log-Pearson Type III streamflow determination can be seen in Appendix B.

3.2 Hydraulic Model Development

Urban and rural study reach hydraulic models were developed using standard hydraulic engineering practices in conjunction with USACE's HEC-GeoRAS software add-in for ArcGIS. HEC-GeoRAS was used to create the base model for the HEC River Analysis System (HEC-RAS) program. As with the hydrologic model development process for the study areas, publicly available data sources were used. High resolution elevation data was gathered from the Marion County Light Distance and Ranging (LIDAR) elevation DEM (Marion County, Indiana, 2010). The LiDAR data has a finer resolution, a three-foot cell size, which allows for a more precise characterization of the physical channel banks and overbank areas. Aerial photography (Marion County, 2010) for both study areas was supplemented by the NLCD information to estimate surface roughness properties. Hydraulic structure parameters were adapted from a field investigation carried out by Christopher B. Burke Engineering, Ltd (CBBEL).

3.2.1 Channel Geometry

Modeled channel properties were produced by digitizing the key components of the physical system with HEC-GeoRAS. Aerial photographs and the Marion County LiDAR DEM were used to assist in properly locating the stream features. Channel banks, cross-sections, levees, lateral structures, storage areas, as well as bridges and culverts were mapped, with the LiDAR DEM being used to provide the necessary elevation information. Cross-sections were placed perpendicular to the expected flowpath, and were allowed to extend across the entire floodplain, where possible. Channel and overbank roughness coefficients were determined using guidance from Ven Te Chow's Open-Channel Hydraulics (Chow, 1959). Channel roughness values were established by considering the effects of the presumed bed material, degree of channel bed irregularity, variations in channel cross-section, the relative effect of obstructions, vegetation, and the degree of channel meandering. Overbank and floodplain roughness values were determined from land use type and the suggested range of values presented by Chow. Table 2 contains the ranges of Manning roughness values used in the hydraulic analyses.

Table 2: Manning Roughness Ranges for Study Areas

Land Use Type	Urban Levee Study Area Manning's n-value Range	Rural Levee Study Area Manning's n-value Range
Athletic Fields / Open Space	0.040 - 0.045	0.055
Water	0.035	0.050
Commercial	0.060 - 0.080	-
Forested Area	0.055	0.060
Heavy Residential	0.080	0.080
Medium Residential	0.060	-
Light Residential	0.050	0.070
Farm Field	-	0.055

3.2.2 Initial and Boundary Conditions

Initial and boundary conditions were established for the models based on the results of the hydrologic modeling and the hydraulic structures specific to each study reach. Bridge and culvert geometry was input into the model from the CBBEL study survey information. Hydraulic rating curves were generated for each internal boundary such that the rating curve would extend beyond the greatest depth and flow rate experienced by the system. Areas behind levees were modeled as storage areas. Initial storage area stages were set to the minimum ground surface elevation within the respective storage areas. Boundary conditions for the model were determined by the physical extent of each model and the results of the hydrologic simulation. All models utilized a downstream boundary condition of normal depth. Care was taken to terminate each model where normal depth was likely to be established. Flow hydrographs for the respective stream segments served as the upstream boundary conditions.

3.2.3 Levee Breaching

Levee breach parameters were assigned to each of the respective study reaches upstream-most levee segment using guidance from the Indiana Department of Natural Resources' (IDNR) suggested breach parameters (IDNR, 2001), as shown in Table 3.

Table 3: IDNR Suggested Breach Parameters

Type of Dam	Avg. Breach Width	Breach Side Slope (H:V)	Time to Failure (hrs)
Masonry; Gravity	Monolith Width	Vertical	0.1 to 0.3
Rockfill	HD	-	-
Timber Crib	HD	Vertical	0.1
Slag; Refuse	80% of W	1.0 to 2.0	0.1 to 1.0
Earthen "non-engineered"	2HD to 5HD	0.0 to 1.0	0.1
Earthen "engineered)	2HD to 5HD	0.0 to 1.0	0.5 to 1.0

HD - Height of Dam
W - Crest Width

Average levee heights were used to determine the size and shape of all modeled levee breaches. Preliminary modeled water surface profiles were generated assuming that no levees breached. Hypothetical breach causes were determined by a comparison of the levee crest elevation and the adjacent modeled flood elevation. Levees failures were modeled as overtopping for situations where the modeled flood elevation was greater than the levee crest elevation. Piping failures were modeled for all levees that had sufficient height to prevent overtopping from occurring. Levees being overtopped were breached immediately after water surface elevations reached the levee crest elevation; piping failures were initiated when the river stage reached a peak value. Table 4 contains a summary of the modeled levee breaches.

Table 4: Modeled Levee Breach Parameters

Breach Characteristic	Study Area				
	WR-02^R	WR-03^R	UNL^R	HD-C1^R	WR-C1^R
Bottom Width (m)	9.1	21.3	18.3	10.7	10.7
Side Slopes (H:V)	1	1	1	1	1
Top Width (m)	12.8	29.9	25.6	14.9	14.9
Total Breach Area (m^2)	20.1	109.3	80.3	27.3	27.3
Breach Invert (m)	199.7	198.2	199.4	222.0	223.5
Breach Cause	Piping	OT$^+$	Piping	Piping	Piping
HW US Peak Time^{++} (m/d/yy hh:mm)	1/5/10 18:00	1/5/10 18:00	1/5/10 13:00	1/1/10 20:00	1/5/10 0:00
Breach Time (m/d/yy hh:mm)	1/5/10 18:00	1/3/10 16:30	1/5/10 13:00	1/1/10 20:00	1/5/10 0:00

^{U} Urban Levee Segment
^{R} Rural Levee Segment
$^+$ Overtopping
$^{++}$ Riverward peak water surface elevation at the upstream end of the breach

All levee segments were modeled as lateral structures to allow flow to enter the protected area, should the levee height not be sufficient. Storage areas were used to represent the interior areas behind the levees and were connected to the lateral structures and breach segments.

3.2.4 Unsteady-Flow Simulation

Unsteady-flow simulations were performed on each study reach. Maximum water surface and flow rate changes were set to increase the resolution of the results, in addition to promoting model stability. Computation and output intervals were set to aid in model stability and generate high resolution output. Modeled water surface profiles were examined for irregularities and erroneous results; when found, unsteady-flow modeling parameters were adjusted.

3.3 Breach Damage Estimation

Following completion of each unsteady-flow modeling scenario, maximum water surface profile information was exported to ArcGIS in order to prepare floodplain maps and inundation depth grids. Storage area inundation depth grids were prepared for use in FEMA's HAZUS program. Study areas were developed within the HAZUS program by selecting the census tracts and census blocks associated with each floodplain area developed by the respective levee breaches.

Default building stocks and infrastructure data were used for each study area to maintain the comparability of the levee segments. Local, spatially-referenced infrastructure information was not available for all of the study areas; therefore, no user-defined infrastructure components were added to any of the study areas due to a lack of comprehensive data.

Analyses were carried out to assess the amount of damage and economic loss expected for each of the protected areas. Loss estimates were developed for building, agricultural, transportation, utility, and vehicle losses. Income and inventory losses were included in the building loss estimate for each study area.

3.4 Improvement Cost Assessment

The high resolution DEM data, aerial photography, and site inspections were used to aid in the development of expected levee improvement costs. Deficiencies were identified by using the required qualities of a certified levee based on 44 CFR 65.10. The requirements of this federal regulation are summarized in Table 5 (FEMA, 2008).

Table 5: Requirements of 44 CFR 65.10

44 CFR 65.10 Criteria	Design Requirements
Freeboard	Levee must be constructed with a minimum of 3 - 4.5 ft of freeboard above the effective base flood profile (1%-annual-chance flood water surface profile)
Penetrations	Provide positive backflow prevention in the form of sluice gates and/or flap gates/check valves/bolt-down lids on all storm and sanitary sewers penetrating through or under the levee to prevent flooding of interior areas.
Stability	Provide a stable foundation for all levees/floodwalls. Remove all material which may compromise long term stability. Install foundation drains to prevent piping failure.
Settlement	Construct levee to a height such that any anticipated settlement over time will not result in freeboard below the minimum requirements.
Interior Drainage	Provide for interior drainage by using gravity sewers and/or pump stations such that interior ponding areas do not develop during coincident rainfall and flooding events.

The Flood Insurance Study flood profile for the WFK White River and Howland Ditch were used to develop the minimum required levee crest elevations for each segment. Cross-sections of the existing levee were generated from the LiDAR DEM. Typical levee cross-sections were projected onto the existing cross-sections to determine the quantity of cut and fill, as well as the extent of surface disturbance and surface restoration. The typical levee cross-section used in the analyses is shown in Figure 5. The USACE requires that no woody vegetation be present within fifteen feet of the levee toe (USACE, 2000). The extent of tree removal was determined by a combination of aerial photography and the levee cross-sections generated from the LiDAR DEM.

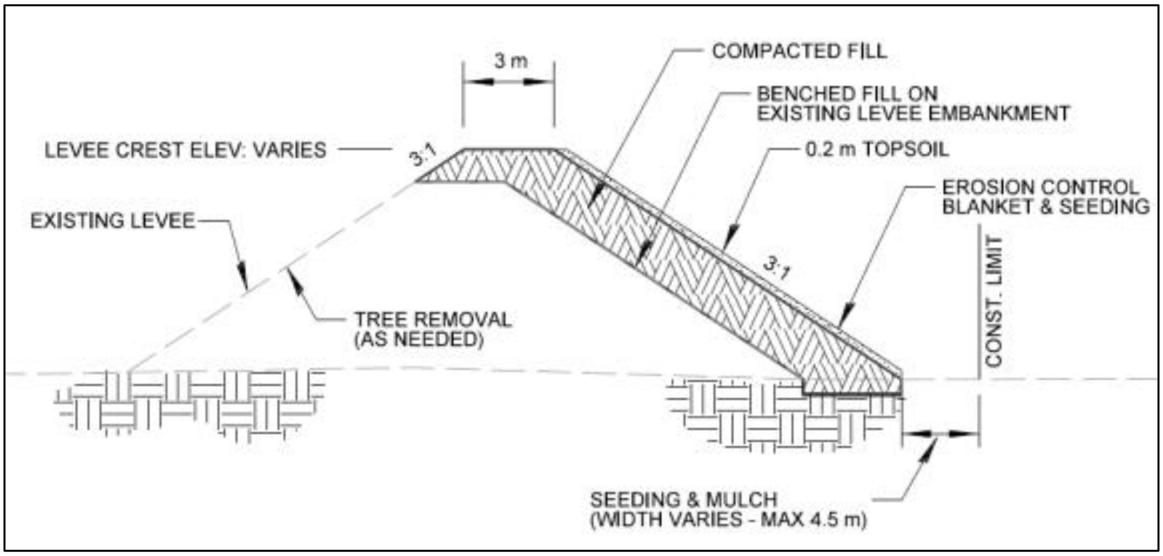

Figure 5: Typical Levee Cross-section

Other critical aspects of determining the repair cost of a levee include the number of storm and sanitary sewer penetrations, the condition of the underlying soil material, the existence and condition of interior drainage systems, and the number and size of necessary openings in the line of protection. Detailed topographic and utility surveys are necessary to develop the number of underground utilities passing under, or through, the levee; this work is often time consuming and expensive for large areas. As a result, underground utilities were not considered in the improvement estimates for the study area levee segments. The number and condition of visible backflow prevention devices needed for culverts and pipes was determined during site inspections. The physical properties of the levee's parent materials can only be determined by a physical investigation including soil borings in the vicinity of the levee. The geotechnical investigation required to assess the condition of base soil materials is similarly expensive and time consuming; therefore, this consideration was not included in this study. Current local unit cost estimates were used (CBBEL, 2011) to convert the necessary improvements to dollars of expected construction cost. Engineering judgment and industry standards were used to determine the approximate site survey, geotechnical, and design fees for each levee segment.

CHAPTER 4 STUDY AREA RESULTS

4.1 Hydraulic Results for Levee Breaches

Due to differences in protected area size, topography, flooding source, as well as levee characteristics, the resulting breach floods were wide-ranging. Flooding duration was accounted for by the length of time during which water was flowing into, or out of the storage area in appreciable amounts. By using this method of determining flooding duration, the duration required to dewater the interior area below the bottom of the levee breach is not considered. The data requirements necessary to adequately describe the dewatering time is beyond the necessary scope for the intended purpose of this analysis. A summary of the breach results is shown in Table 6.

Table 6: Levee Breach and Storage Area Results

Storage Area	Max Stage (m)	Max Inflow (m^3/s)	Peak Storage (Mm^3)	Flood Area (km^2)	Flooding Duration (days)
WR-C1[U]	224.12	24.9	1.2	0.8	6.0
HD-C1[U]	222.78	20.1	0.4	0.4	5.0
UNL[R]	201.03	159.9	8.7	6.4	8.8
WR-02[R]	201.15	25.4	0.2	0.1	4.1
WR-03[R]	202.76	150.9	0.3	0.1	6.1

[U] Urban Levee Segment
[R] Rural Levee Segment

4.1.1 Storage Area Stage and Flow Hydrograph Analysis

Unsteady-flow modeling using storage areas provides the opportunity to assess the details of the system response to the levee breach. The resulting stage and flow hydrographs can be examined to extract critical information regarding the nature of the flooding behind the levee as well as the key factors which caused the flooding to occur in the manner predicted by the model.

The results of an unsteady-flow analysis of a levee breach in WR-C1, shown in Figure 6, reveals a relatively moderate peak breach inflow. The stage hydrographs proximate to the levee breach suggest that the peak water surface elevation achieved in the storage area is significantly less than the stage in the main river channel. A reduction in water surface elevation is created by the lag between the peak of the river hydrograph and the peak in the storage area hydrograph. The unsteady-flow modeling allows for an approximation of the time required for the interior and exterior stages to equalize. Based on the model results, the flood stage behind the breached levee will not decrease as fast as the exterior river stage which is due to the small breach size relative to the size of the WR-C1 storage area. The duration of the flooding behind the levee is expected to be lengthened by this phenomenon.

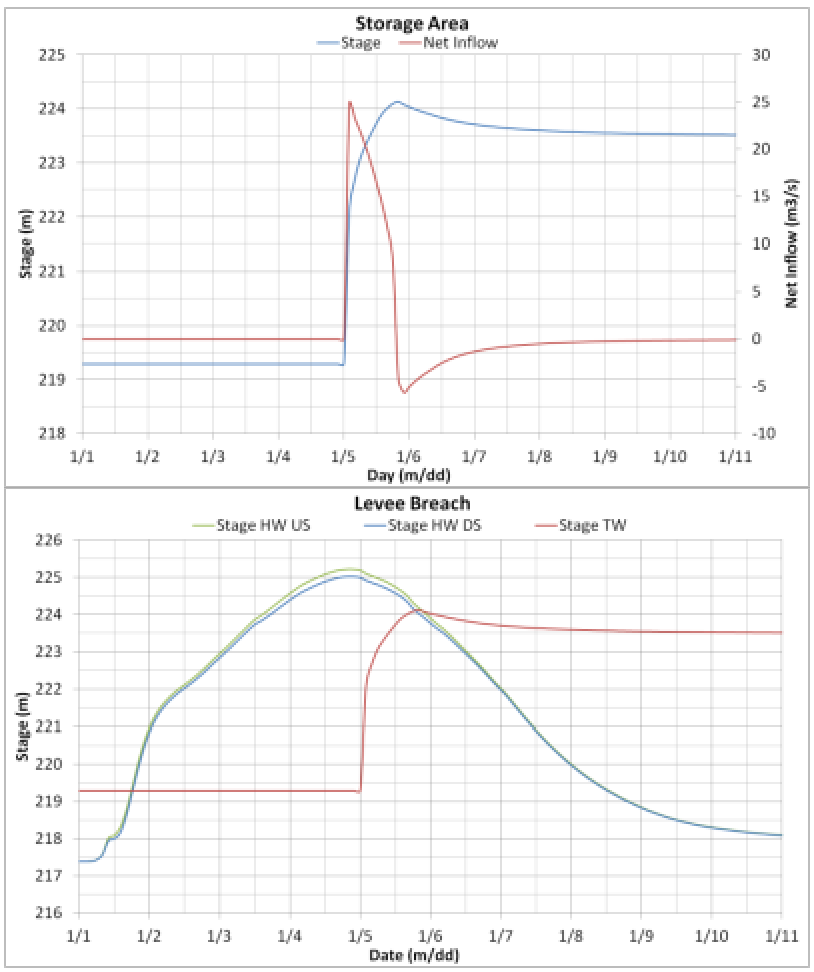

Figure 6: WR-C1 Stage and Flow Hydrographs

Though HD-C1 and WR-C1 protect the same area, a breach in HD-C1 is expected to produce a significantly different flood as compared to a breach in WR-C1. The impact of the difference in flooding source can be seen by noting several key changes in the shape of the stage and flow hydrographs in Figure 7. The first is the timing of the flooding. The peak of the Howland Ditch stage hydrograph, shown in series 'Stage HW US' in Figure 7, occurs much earlier than that of the WFK White River peak. The second and most notable difference is the difference in the magnitude and duration of flooding. The flooding resulting from a breach in HD-C1 is expected to create a flood stage that is 1.34 meters lower than the flooding caused by a breach in WR-C1, with a flood duration one day less.

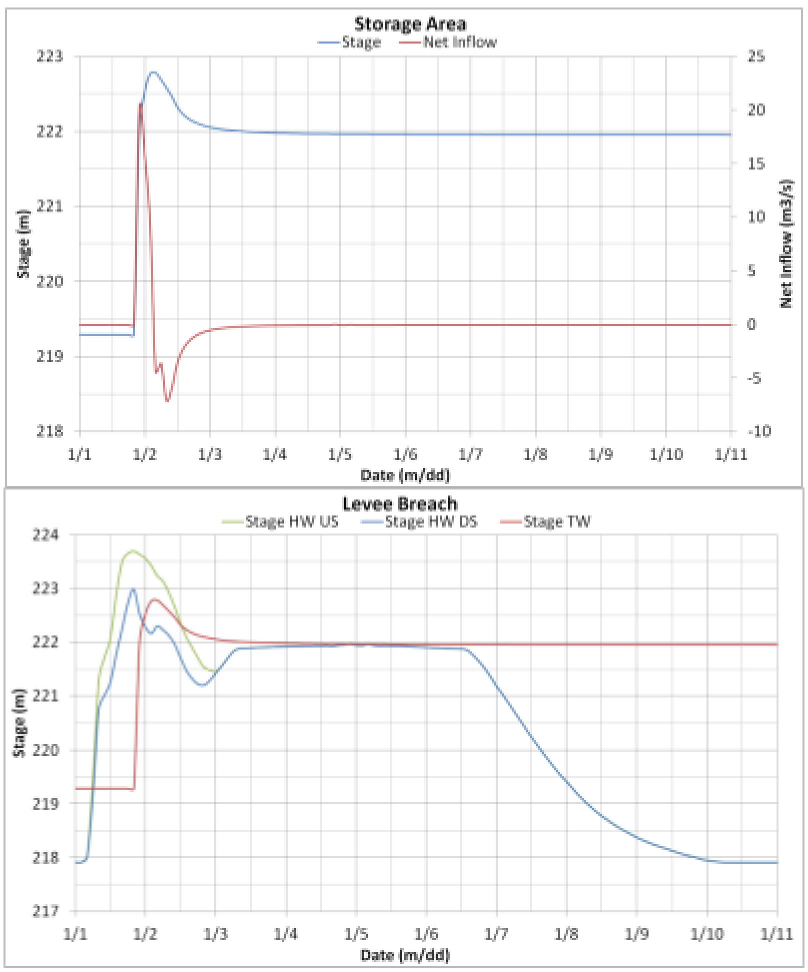

Figure 7: HD-C1 Stage and Flow Hydrographs

The connectivity of Howland Ditch to WFK White River also causes a slight amount of inflow to the storage area significantly after the levee breach has allowed the interior and exterior flood stages to equalize. If the flooding along WFK White River were more severe, a second rise in flood elevations behind the levee would have occurred due to increased backwater. Though the second flood wave experienced behind the failed levee was not severe, the prevention of flood subsidence lengthened the duration of flooding by several hours. Had the second flood wave been more severe, the duration could have been lengthened considerably.

The simulated levee breach in UNL suggests a more intense and severe flood wave into the area behind the levee. The height of the levee and the flooding capacity of the WFK White River are apparent in the magnitude of the peak inflow to the storage area. The breach diverts a sufficient amount of flow from the WFK White River to noticeably decrease the stage of the river during the initial levee breach. Inflow to the area behind the levee began to occur before the levee breach as a result of downstream portions of the levee having insufficient height. The flooding created by the levee overtopping alone has a relatively minor impact compared to the breach at the upstream end of the levee. Despite the large breach inflow, the storage capacity of the interior area and the insufficient downstream levee elevations prevent the equalization with the exterior flood stage for approximately 52 hours. Figure 8 contains the stage and flow hydrographs for the UNL levee breach model simulation.

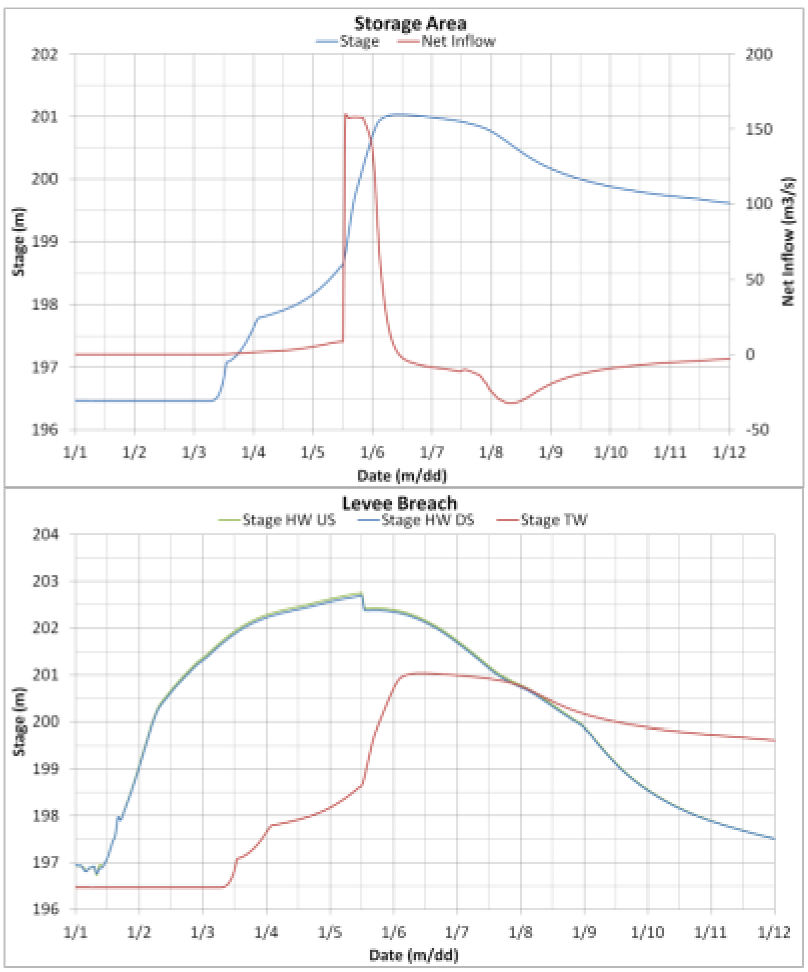

Figure 8: UNL Stage and Flow Hydrographs

Figure 9 displays the reaction of the storage area behind WR-02 to a levee breach at the upstream end of the levee segment. The size of the storage area behind WR-02 plays a significant role in the system's response to the levee breach. Despite the small breach area, the flood stage on the interior of the levee equalizes with the exterior stage within a matter of hours. The storage area filled to the level of the exterior flooding before the exterior flooding could begin to subside, allowing no reduction in flood elevation. The absence of a lag time between interior and exterior peak flood stage equalization suggests that the storage area could be modeled accurately using a steady-state model simulation; however, it is apparent from the other analyses that this is not a universal trait of levee breaches. The quick response time of the storage area behind WR-02 allows the storage area stage to lower at the same rate as the main river, preventing any extension of flood duration.

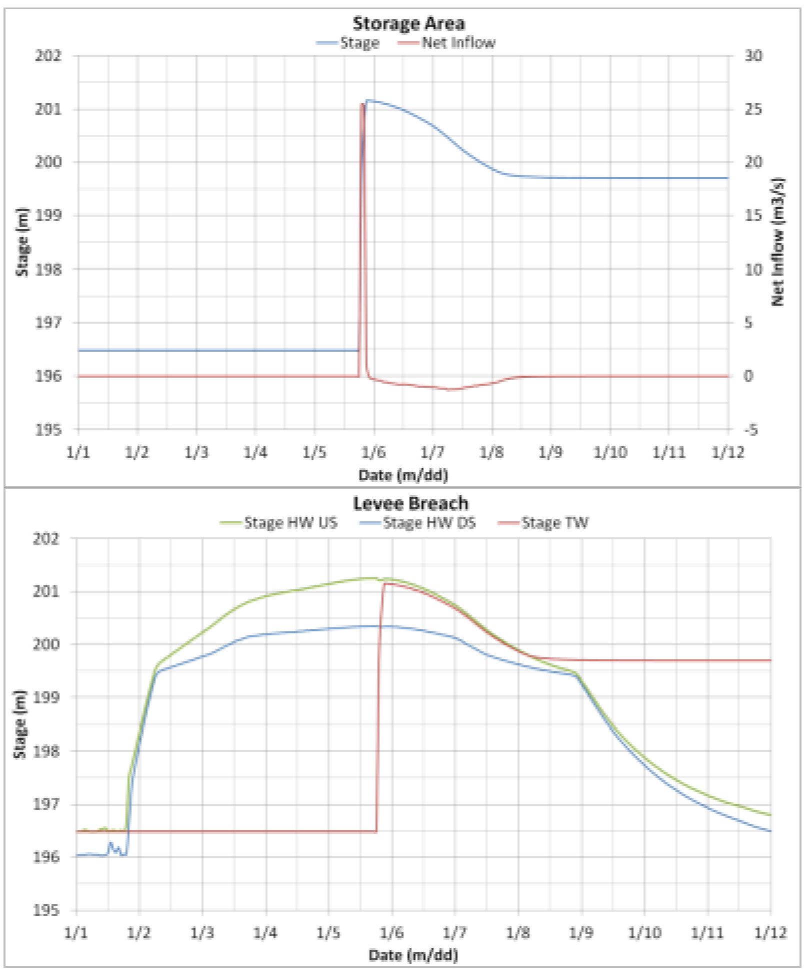

Figure 9: WR-02 Stage and Flow Hydrographs

The levee breach simulation for WR-03 was initiated by overtopping prior to the peak flood elevation along WFK White River. As a result, the area behind the levee is filled to the level of the WFK White River before the flood crests. The storage capacity behind WR-03 is quite small in comparison to the levee breach flow capacity, resulting in a near instantaneous filling of the area behind the levee. Insufficient levee height along the majority of the levee's length allows for constant interaction with WFK White River. This interaction results in a slight amount of instability within the model, as can be seen in the 'Net Inflow' series of the Storage Area plot in Figure 10. By inspecting the stage hydrographs associated with the levee breach, one can determine that the amount of flow both into and out of the storage area during this period of high fluctuation result in minor increases and decreases in stage, which suggests a negligible inflow value.

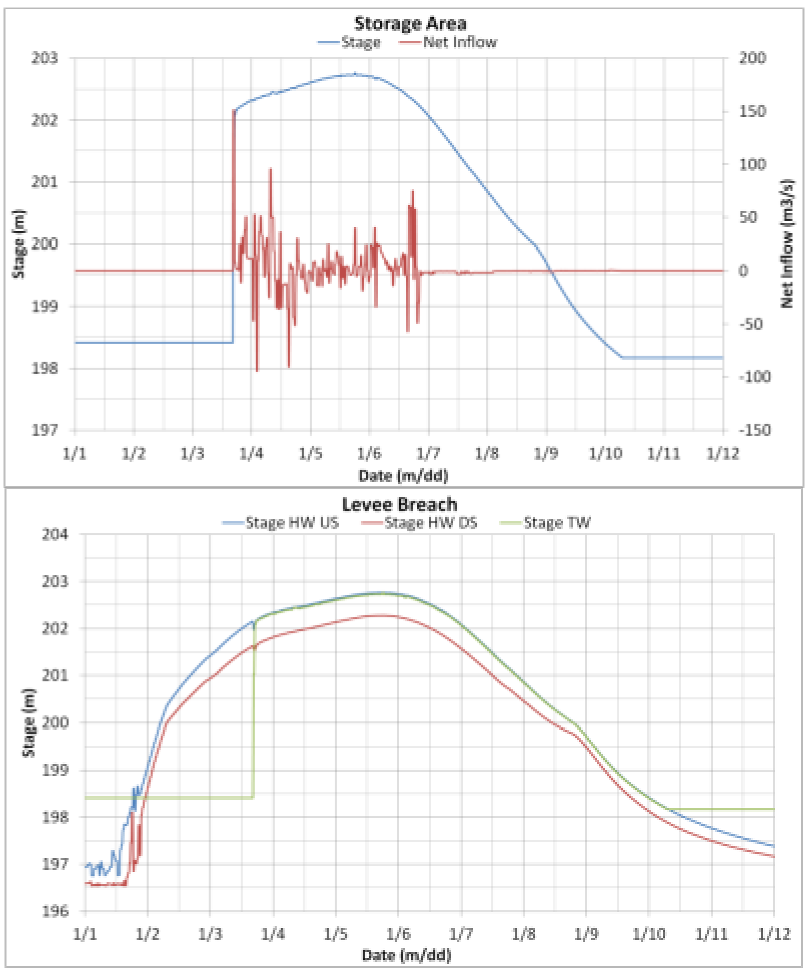

Figure 10: WR-03 Stage and Flow Hydrographs

The storage areas behind WR-C1, HD-C1, and UNL exhibit a considerable reduction in water surface elevation created by the lag between the peak of the river hydrograph and the peak in the storage area hydrograph. Longer equalization time, or time required for the interior and exterior stages to reach equilibrium, corresponds to a greater reduction in flood stage. A summary of the equalization time and stage loss for each storage area is shown in Table 7.

Table 7: Breach Equalization Time and Stage Loss Results

Breach Characteristic	Study Area				
	WR-02^R	WR-03^R	UNL^R	HD-C1^R	WR-C1^R
Breach Time (m/d/yy hh:mm)	1/5/10 18:00	1/3/10 16:30	1/5/10 13:00	1/1/10 20:00	1/5/10 0:00
EQ Time (m/d/yy hh:mm)	1/5/10 21:00	1/3/10 17:00	1/7/10 16:00	1/2/10 12:00	1/5/10 20:00
Equilization Time (hrs)	3.0	0.5	51.0	16.0	20.0
Stage Loss (m)	0.0	-0.1	1.4	1.3	1.0

[+] Overtopping

[++] Riverward peak water surface elevation at the upstream end of the breach

4.1.2 Inundation Areas and Depth Grids

Inundation area maps and depth grids were created to determine the geographic extent of the flooding, locations of extreme flooding, and to assess the validity of the assumption of perfect hydraulic connectivity within the storage areas behind the levee segments. Storage volumes and storage areas were compared to the total storage volume and area of the respective levee segments to determine the level of storage and floodplain consumption.

The floodplain resulting from a breach in WR-C1 is expected to produce a floodplain that consumes 80-percent of the surface area of the storage area and 87-percent of the storage volume. The most extreme flooding within the storage area occurs in the multi-family residential area in the northern portion of the flooded area, as depicted in Figure 11. The depth of flooding within the storage area is relatively consistent with an average of 1.5 meters of depth. The continuous nature of the flooded are suggests that the assumption of perfect hydraulic connectivity is reasonable for the given application and breach scenario.

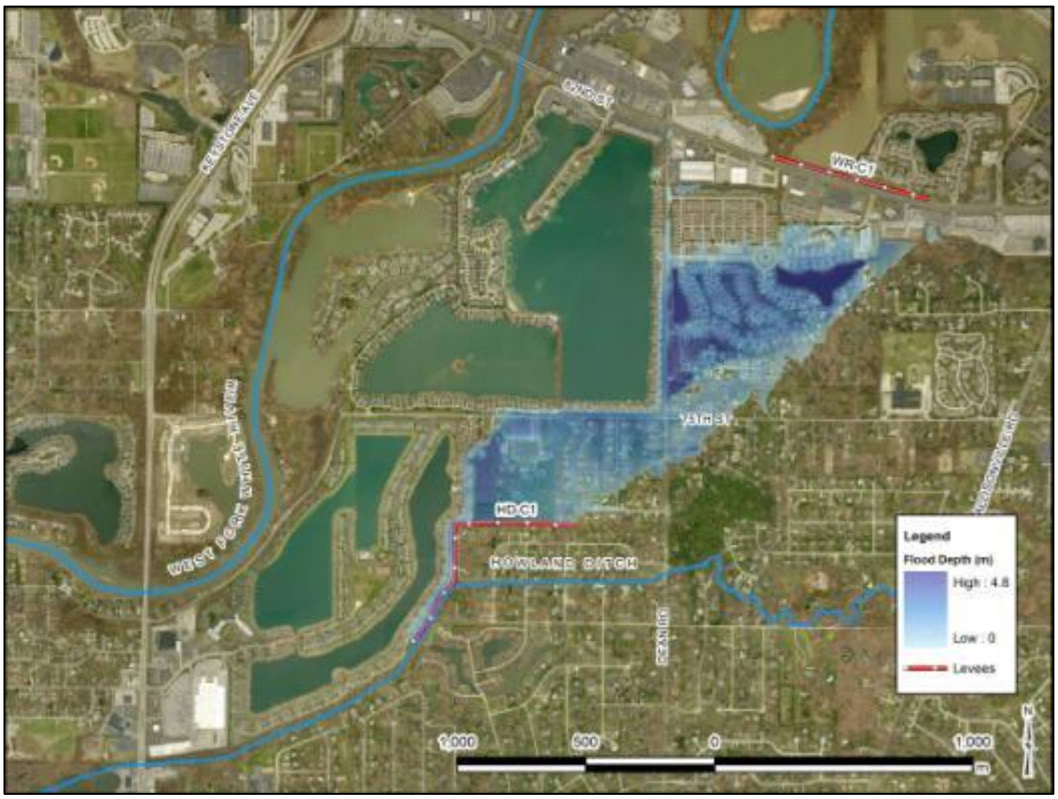

Figure 11: WR-C1 Levee Breach Depth Grid

The less severe flooding of the urban study area caused by a breach in HD-C1 produces a floodplain area that inundates approximately 42-percent of the total surface area and utilizes 46-percent of the storage volume. The low-lying residential area is once again the site of the most extreme flooding. Flood depths, shown in Figure 12, have an average of 0.9 meters. Though the floodplain area is not fully connected, it is plausible that overland flow could have allowed for substantial temporary hydraulic connectivity of the two main flooding areas.

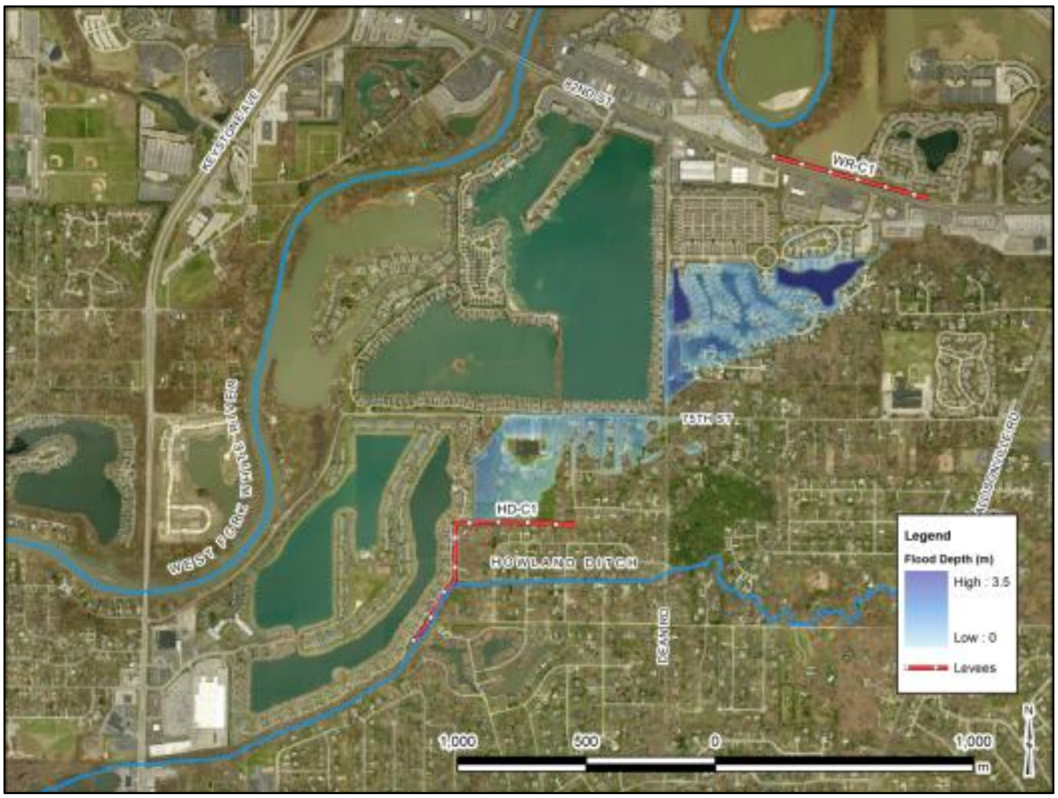

Figure 12: HD-C1 Levee Breach Depth Grid

Flooding within the storage area behind UNL occurs mainly in the southern portion of the protected area, as the land slopes generally to the south. Agricultural lands in the south are affected most heavily by the resulting flood. The levee breach is expected to allow for 88-percent of the available floodplain area to be covered by water, and 49-percent of the storage volume to be filled by the flood. Flood waters are expected to average 1.8 meters in depth. The depth grid shown in Figure 13 reveals a flood boundary with significant disconnects; therefore the assumption of perfect hydraulic connectivity appears to be slightly less plausible for the UNL storage area.

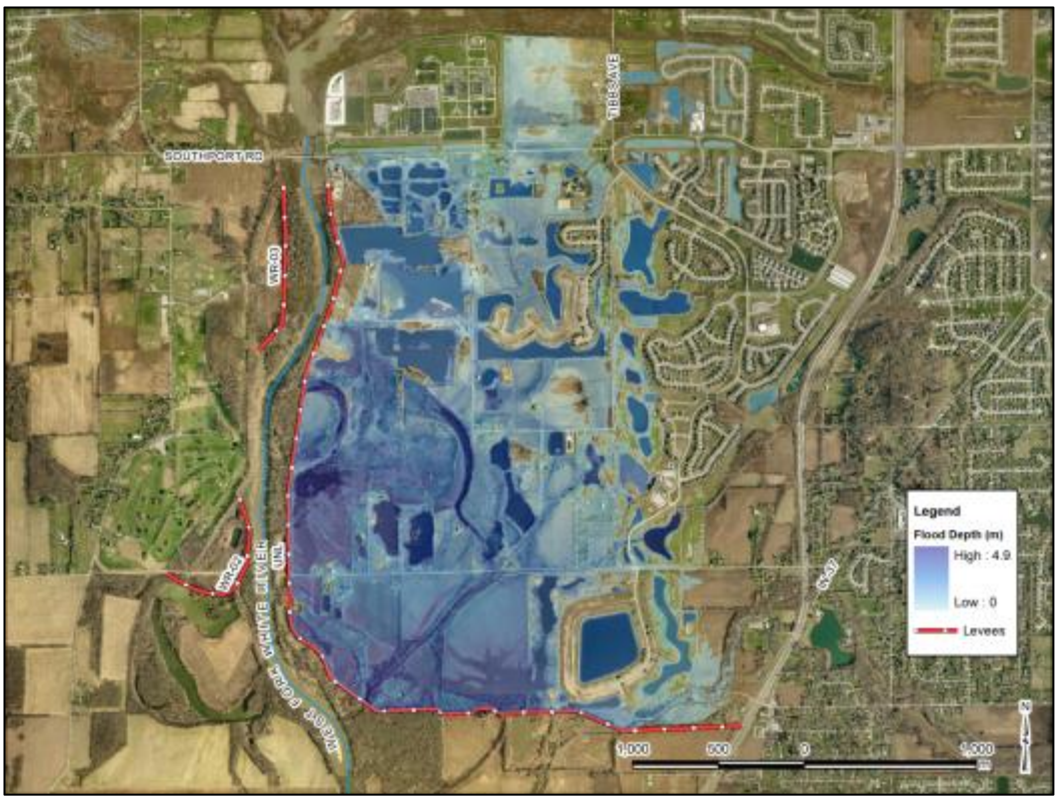

Figure 13: UNL Levee Breach Depth Grid

The size and relatively uniform topography of WR-02 creates a similarly uniform flood depth throughout the storage area. The average flood depth is 1.9 meters. The floodplain encompasses 98-percent of the total area, and 99-percent of the storage volume is consumed. The small size of the protected region and fact that the flood boundary nearly covers the entire area agree with the assumption of perfect hydraulic connectivity. The depth grid for WR-02 is shown in Figure 14.

Figure 14: WR-02 Levee Breach Depth Grid

The flooded area behind WR-03 is quite similar to WR-02. The storage area is small and has consistent flood depths over the whole surface; the flood depths have an average value of 1.8 meters. The flood is expected to cover 100-percent of the total area, and occupy 100-percent of the total storage volume available. As with WR-02, the size and connectivity of the flooded area suggest that the assumption of perfect hydraulic conductivity is valid. Figure 15 shows the depth grid for the flood resulting from a levee breach in WR-03.

Figure 15: WR-03 Levee Breach Depth Grid

4.2 Breach Damage Estimation Data

Expected losses for the levee breaches ranged significantly as a result of the varying levee lengths, levee heights, size of protected area, as well as with the type and number of buildings protected. Table 8 contains a summary of the losses due to the flooding caused by the levee breaches. Appendix D contains more detailed information concerning the losses determined by HAZUS.

Table 8: Breach Damage Summary

Storage Area	Breach Damage Parameters						
	Building Losses (x $1000)	Agricultural Losses (x $1000)	Transportation Losses (x $1000)	Utility Losses (x $1000)	Vehicle Losses (x $1000)	Displaced Citizens (#)	TOTAL BREACH DAMAGES (x $1000)
WR-C1 [U]	$24,794	$0	$0	$0	$2,395	391	$27,189
HD-C1 [U]	$5,950	$0	$0	$0	$757	212	$6,707
UNL [R]	$12,231	$157	$0	$0	$1,158	174	$13,545
WR-03 [R]	$64	$3	$0	$0	$5	1	$72
WR-02 [R]	$82	$2	$0	$0	$7	1	$91

[U] Urban Levee Segment
[R] Rural Levee Segment

No spatial data was available for local roads, railways, and bridges within the study areas; only state-owned infrastructure components were considered in the loss analysis. Utilities considered in the analysis were limited to key components of utility systems such as electrical substations, water and wastewater pump stations; these components were not impacted by the flood areas developed by the levee breaches.

4.3 Improvement Cost Data

Each levee segment had varying degrees of deficiency, with some needing only minor improvements and others requiring virtual reconstruction. Expected improvement costs ranged as widely as the levees' impaired condition. The large disparity in levee size creates an equally large gap in the amount of funding necessary to improve the levees. The most influential levee deficiencies were tree cover, inadequate levee height, and over-steepened side slopes. In the case of the rural levees, these deficiencies necessitate the disturbance of large areas of land leading to increase demolition and reclamation costs. Design and permitting fees of extensive repairs were also included as a percentage of the construction cost, which further increased the difference between the rural and urban levee segments as the urban levee segments will require less extensive permitting due to the decreased disturbance areas.

Table 9 contains a summary of the expected improvement costs for each of the levee segments. A more detailed form of the improvement cost estimates can be seen in Appendix E.

Table 9: Improvement Cost Summary

Storage Area	Improvement Cost Parameters						
	Demolition (x $1000)	Levees & Bank Stabilization (x $1000)	Site Restoration (x $1000)	Drainage (x $1000)	Misc. (x $1000)	Professional Services (x $1000)	Total Cost (x $1000)
WR-C1[U]	$0	$67	$215	$47	$216	$74	$650
HD-C1[U]	$0	$8	$79	$47	$83	$30	$252
UNL[R]	$482	$2,050	$1,930	$47	$3,390	$899	$9,938
WR-03[R]	$101	$473	$397	$12	$744	$218	$2,201
WR-02[R]	$84	$429	$341	$0	$649	$190	$1,920

[U] Urban Levee Segment
[R] Rural Levee Segment

CHAPTER 5 CONCLUSIONS

The condition of existing levees and lack of sufficient funding in the United States demonstrates a need for a standardized method of assessing the risk for home, land, and business owners whose property is situated behind a levee. The modeling and analysis procedures set forth in this study are capable of limiting the risk to both the properties in jeopardy and the limited funds available by prioritizing levees in order for decision makers to attain the highest possible return-on-investment. The results of the analysis have identified key components of levee breach assessment for prioritizing levee rehabilitation, including:

5.1 Unsteady-flow Modeling

The use of unsteady-flow modeling provides more information concerning the hydraulic system's response to a levee breach as well as producing more realistic results. Regulatory models utilize a steady-flow analysis which produces more conservative flood elevations; however, the procedure used in this study is not suggested for use in regulatory floodplain or floodway determination. Based on the intended use, the more realistic nature and increased amount of information provided by an unsteady-flow analysis is warranted. The purpose of prioritizing levees is to appropriate funds in the most beneficial way. For this to occur, a realistic, yet somewhat conservative view must be employed. Unsteady-flow modeling allows for the use of storage areas within HEC-RAS to determine the potential flood depths behind a levee rather than modeling the area as a flowpath for the channel. Simply ignoring that a levee exists is to ignore the true physical nature of the system. A levee breach may be quite small relative to the size of the protected area, preventing the consumption of all storage volume behind the levee prior to subsidence of riverine flooding. Storage areas behind levees may not fill to the peak water surface elevation along the river, or to the top of the levee crest during a flooding event, which are the possible floodplain determinations by current standards of practice. This is made evident by the results expounded upon in Section 4.1.1.

An examination of steady-state analyses of the same study segments reveals a large disparity between the predicted flood elevations. The steady-state flood elevations shown in Table 10 were produced using the peak flow rate from the unsteady-flow hydrographs for each study area channel reach. A comparison of unsteady-flow and steady-state flood depth grids can be seen in Appendix A.

Table 10: Comparison of Unsteady and Steady-flow Flood Elevations

Storage Area	Unsteady-flow Max Stage (m)	Steady-flow Max Stage (m)	Unsteady-flow Peak Storage (Mm^3)	Steady-flow Peak Storage (Mm^3)	Unsteady-flow Flood Area (km^2)	Steady-flow Flood Area (km^2)
WR-C1[U]	224.12	224.96	1.2	1.7	0.8	1.0
HD-C1[U]	222.78	223.70	0.4	0.9	0.4	0.7
UNL[R]	201.03	Varies	8.7	13.2	6.4	6.6
WR-02[R]	201.15	201.14	0.2	0.2	0.1	0.1
WR-03[R]	202.76	202.82	0.3	0.3	0.1	0.1

[U] Urban Levee Segment
[R] Rural Levee Segment

Finally, unsteady-flow analyses consider in-channel and floodplain storage of runoff. The runoff flow hydrograph for Howland Ditch has a peak value of 55.4 m^3/s; however, the peak flow passing through much of the channels length in the model was 41.1 m^3/s due to significant storage of flow behind bridges and other obstructions.

5.2 Storage Areas

As mentioned in the discussion of steady versus unsteady-flow simulation, the use of storage areas to represent the levee-protected areas more closely resembles the physical reality of the situation than the current standard of practice. Steady-state models are incapable of determining the duration of flooding, and are therefore not able to determine the impact of the relative size of the levee breach and storage area on the duration of flooding. Larger storage areas could experience increased flooding duration as a result of insufficient drainage of flood waters. The physical extent of the breach and inadequacy of the drainage system supporting the area may cause the area to retain the flood water at a higher stage than that experienced on the riverward side of the levee.

5.3 Priority Ratio

Cost-benefit analyses are regularly performed to determine if it is economically practical to construct a new levee. After a levee is constructed and is allowed to age and continue into a state of disrepair, a new issue arises. Development behind the levee prevents the issue from being solely economically based. Rather than questioning whether or not to build a levee, decision makers must decide if and when to improve the levee. By using only the expected damages resulting from a breach to prioritize levees, a tendency toward repairing larger levee segments or levee segments in more affluent regions will become apparent. In order to more wisely allocate funding, the cost of the repairs must also be considered. When comparing levees WR-C1 and UNL using the expected damages alone, UNL has a higher ranking; when using a ratio of expected damages to improvement cost, the priority ratio, WR-C1 has a higher ranking. By using the priority ratio, investors should achieve a higher return-on-investment.

In addition to helping prioritize levees, the development of a larger dataset of analyses could allow for determination of threshold ratio values to help suggest when decision makers should improve or abandon levees or when to buyout the homes and properties behind the levees to eliminate the hazard. The Property Acquisition Program currently run by FEMA focuses on purchasing private properties in repetitive loss areas. The local or state government is required to produce 25-percent of the total capital investment, while the remaining 75-percent is covered by FEMA. To be considered for a buyout, the entire buyout area must be a part of the National Flood Insurance Program; areas that have not elected to be included in the program are not eligible for property buyout projects. Following a buyout, the land becomes public property and must remain as open space, property resale and development is not an option. The land may remain as open space or be converted to parks, wildlife refuges, or other undeveloped, natural uses. (FEMA, 2010) The program has obvious benefits, both in terms of risk reduction and environmental well-being; however, the reactive approach of the method prevents continued risk, rather than eliminating risk before lives and property are lost. Utilizing a threshold priority ratio to identify areas that are subject to unsatisfactory risk could achieve the two-fold benefits of a buyout, without unnecessary exposure to flood damage and potential loss of human life.

A lack of decisive correlation between hydraulic, zoning, social, breach damage, and improvement cost statistics fails to provide a rule-of-thumb for determining levee priority. The priority ratio can serve as a means of sequencing levee improvements and property buyouts, in the absence of a simpler prioritizing scheme.

5.4 Decision Guidance and Project Justification

By using a standardized procedure and ranking levees against one another, decision makers may experience less public opposition to flood control projects. Complete removal of all public criticism is not likely, as risk assessments only constitute a small portion of regulatory control (National Research Council, 1983). Risk management policy, or the regulatory actions taken as a result of the risk analyses, may still be viewed with opposition. The presence of quantitative evidence of need for improvements and the sequence in which the improvements should be made can be used to increase stakeholder buy-in. Efforts should be made to limit the bureaucratic and political involvement in decision making and development of risk management policy.

CHAPTER 6 TOPICS FOR FUTURE DEVELOPMENT

6.1 Loss-of-life Modeling

The value of human life cannot be overlooked in any risk assessment that attempts to be widely applicable and accepted. Publicly available loss-of-life models have yet to be developed for analyses such as the one described by this study. Highly complex loss-of-life models have been used in other studies. The IPET study of the aftermath of Hurricane Katrina utilized proprietary modeling software developed specifically for the analysis of the New Orleans disaster. Development, or public release, of spatially referenced loss-of-life modeling software would enable analysts to more appropriately factor in the impact of levee breach progression and the rate at which floodwaters rise. Use of spatial reference would allow for the use of census data to incorporate the age of endangered citizens, as well as the size of families within the impacted areas.

6.2 Social and Cultural Impacts of Flooding

Social and cultural impacts of flooding disasters have been studied diligently in the recent years, specifically during the IPET evaluation of Hurricane Katrina. Though the social and cultural impacts were assessed, all data presented was strictly qualitative or anecdotal. A risk assessment methodology that includes social and cultural impacts and is able to withstand public criticism would require quantitative results. These results would likely need to be translated into dollar figures by loss estimation or increases in impoverishment in order to be combined with breach damage and improvement cost estimations. Valuing cultural and social artifacts in monetary figures will likely be a challenging and highly subjective endeavor. Use of 'willingness-to-pay' surveys may allow for analysts to quantify an approximate value for the endangered social and cultural stock.

6.3 Environmental Benefits of a Natural Floodplain

Quantification of environmental benefits of a natural floodplain suffers the same lack of development as social and cultural aspects. Qualitative and anecdotal evidence of environmental benefits is widely available. Unfortunately, disagreement exists between experts in the environmental fields associated with riparian areas. Without a clear consensus of what defines healthy, beneficial floodplain activity, inclusion of environmental benefits in an economical valuation within a risk assessment will likely be met with much skepticism and opposition.

6.4 Engineered Levee Breaches

Flooding along large rivers often develops over a substantial amount of time, which can allow for human intervention to prevent catastrophic failures of flood control components. During recent flooding along the Mississippi River, the USACE made an informed decision to remove large portions of levee upstream of Cairo, Illinois. The added floodplain storage lowered the peak flood elevation along the main river body by several feet, preventing potentially dangerous stress on the aged levee system. Though the methodology set forth by this study is not specifically intended for the use of assessing the value of storage areas for flood relief, it could be easily adapted to do so. An evaluation of the damages expected if the levee were breached versus the damages expected downstream would allow officials to make informed decisions, knowing the approximate cost of either decision.

Another potentially useful modification of the proposed methodology would be to identify deteriorated levees whose protected areas could serve as compensatory storage areas to provide relief for overdeveloped floodplain areas. Buyout costs could be determined and weighed against the expected reduction of economical risk. The evaluation could also be expanded to include other components of the total risk if more adequate methods of quantifying risk are developed.

6.5 Levee Breach Progression

Levee breach progression studies show that the development of earthen levee breaches is highly variable and contingent upon data that is not easily nor economically acquired. As a result, breach progression models having a high correlation to real-world breach scenarios are lacking. The levee breach progression used by this methodology is over-simplified and confined to a limited set of initiation processes. Breach flow rates are heavily dependent on the assumptions made concerning the size and progression of the levee breach. The height of the levee essentially dictates the magnitude of the peak breach flow rate, when using the IDNR guidelines for breach progression. As breach progression models increase in quality and number, the type of breaches examined in the risk assessment could be expanded to include multiple types of failure initiation and a wider application to levees with different physical components. The breach progression models used in this study are based on grand assumptions of uniform response to stress; however, the assumptions made are the current standard of practice. The importance of breach progression is apparent in the results from this study. Breach progression modeling will also serve as an important factor for any development in loss-of-life modeling due to the impact on the rate at which floodwaters rise and the time available for evacuation.

WORKS CITED

American Society of Civil Engineers. (2009). Report Card for American Infrastructure: Levees.

Baecher, G. B. (2009, September 8). Quantifying Flood Risk.

Bodenhamer, D. J., & Barrows, R. G. (Eds.). (1994). *The Encyclopedia of Indianapolis.* Indiana University Press.

Chow, V. T. (1959). *Open-Channel Hydraulics.* Caldwell, NJ: The Blackburn Press.

Christopher B. Burke Engineering, Ltd. (2007, October 9). City of Indianapolis Levee Assessment Summary. Indianapolis, Indiana.

Christopher B. Burke Engineering, Ltd. (2011). *Southport AWT Levee Extension and Improvements Opinion of Probable Costs.*

Federal Emergency Management Agency. (2008). *Fact Sheet: Requirements for Mapping Levees.*

Federal Emergency Management Agency. (2005). *Guidelines and Specifications for Flood Hazard Mapping Partners.*

Federal Emergency Management Agency. (2011, February). Indiana Flood Fact Sheet.

Federal Emergency Management Agency. (2010, August 11). *Property Acquisition Projects (Buyouts)*. Retrieved June 12, 2011, from Federal Emergency Management Agency: http://www.fema.gov/government/grant/mitmeasures/buyoutshow.shtm

Federal Emergency Managment Agency. (2005). *Flood Insurance Study for Marion County, Indiana, All Jurisdictions.*

Indiana Department of Natural Resources. (2001). *General Guidelines for New Dams and Improvements to Existing Dams in Indiana.* Indianapolis.

Interagency Performance Evaluation Task Force. (2009). *Performance Evaluation of the New Orleans and Southeast Louisiana Hurrican Protection System.* USACE.

Marion County, Indiana. (2010). GIS Data and Aerial Photography. Indianapolis, Indiana.

McCuen, R. H. (1998). *Hydrologic Analysis and Design.* Upper Saddle River, NJ: Prentice Hall.

National Committee on Levee Safety. (2009). *Recommendations for a National Levee Safety Program.*

National Oceanic and Atmospheric Administration. (n.d.). Precipitation-Frequency Atlas of the United States. *2 , 3.0.*

National Research Council. (1983). *Risk Assessment in the Federal Government: Managing the Process.* Washington D.C.: National Academies Press.

National Research Council. (2009). *Science and Decisions: Advancing Risk Assessment.* Washington D.C.: National Academies Press.

National Research Council. (2004). *Valuing Ecosystem Services.* The National Academy of Sciences.

Natural Resource Conservation Service. (2010). SSURGO .

Norquest, C. E. (1913). *March 1913 Flood.* Retrieved June 25, 2011, from http://www.crh.noaa.gov/ind/?n=march_1913_flood

Scawthorn, C. (2006). HAZUS-MH Flood Loss Estimation Methodology. II. Damage and Loss Assessment. *Natural Hazards Review* , 72-81.

United States Army Corps of Engineers. (2000). *Design and Construction of Levees.*

United States Army Corps of Engineers. (2006). *National Levee Database*. Retrieved January 14, 2011, from http://www.usace.army.mil

United States Geological Survey. (2010). *National Elevation Dataset*. Retrieved January 11, 2011, from http://seamless.usgs.gov/

United States Geological Survey. (2006). National Land Cover Database.

United States of America. (2009). *Code of Federal Regulations* .

APPENDICES

Appendix A – Flood of Record: 1913

The more recent flooding in Indiana, though significant, pales in comparison to the disastrous effect of an unusually large rainfall event in the spring of 1913. C. E. Norquest, a local newspaperman, provided a detailed description of the flooding and associated damages shortly after the flood. Norquest explained that in late March, a string of several days of rainfall led to a tremendous amount of runoff. Though the depth of rainfall was not unprecedented, the uniformity of rainfall over the entire watershed had a devastating effect. An average of 7.81 inches of rain fell over a 48 hour period, resulting in the highest flood elevations recorded for the WFK White River. The high rate of rainfall and saturated soils led to rapid river response to the rainfall. The river stage increased a total of 14 feet in a span of three days, surpassing the previous flood of record by 6 feet (Norquest, 1913).

Though Indianapolis had flood control infrastructure at the time, it was not capable of containing the flood waters. A total of 6 square miles was inundated by WFK White River. Several lowland areas in the city were entirely submerged by the raging river, and other areas suffered partial or complete inundation due to levee failures. Advance warning of failing levees allowed for a nearly complete evacuation of the affected areas, decreasing the loss-of-life considerably. Flooding over such a large area of a highly populated region undoubtedly causes a considerable amount of damage. A total of 12 fatalities resulted from the Flood of 1913, 5 of which occurred in Indianapolis (Norquest, 1913). Floodwaters often force families from their homes, leaving them displaced and without any recoverable personal items. The Flood of 1913 displaced approximately 4000 families as their homes were swallowed by the river. In addition to private losses corporate damages were also realized during the flood event. A total of four bridges were washed out as the flooded river neared its crest. Two of the bridges were the property of railroad companies, one municipal bridge, and one private bridge. Though not subject to floodwaters, the remainder of the city felt the impact of the flood as the local power company's boilers were put out by the elevated river, leaving the city without power. Without power, the entire city was left more vulnerable to fire due to lack of electricity for water pumps. Urban areas of the city were not the only areas suffering from the deluge of water, agricultural areas experienced immediate losses, as well as prolonged effects from the flooding of farm fields and pastures. Crops were destroyed and fields were left in poor conditions, with many being left fallow in subsequent years due to the thick layers of sand deposits left by the receding water. Rising water carried away some livestock and left many more stranded and lost (Norquest, 1913).

The financial representation of the losses helps to establish a sense of the true scope of the damage levied on Indianapolis. Property damage to homes, commercial and industrial equipment, and bridges represent nearly half of the financial toll; the estimated property losses totaled over $4.6 million. Agricultural losses in the amount of approximately $310,000 included lost crops and livestock; expected losses due to the sand deposits creating infertile fields were not included in the estimate. Economic losses were established by estimates of lost wages due to forced suspension of labor and product sales reductions; the total estimated amount of economic loss was $622,000 (Norquest, 1913). A significant amount of the total resulting losses from the flood were associated with the bridges destroyed by the raging flood. A total of approximately $4.8 million was recorded for damages to railroads and trolleys. The sum of all of the losses measured in financial terms was $10.4 million for the city of Indianapolis; this equates to nearly $230 million in 2010.

Appendix B- Hydrologic Modeling Data & Results

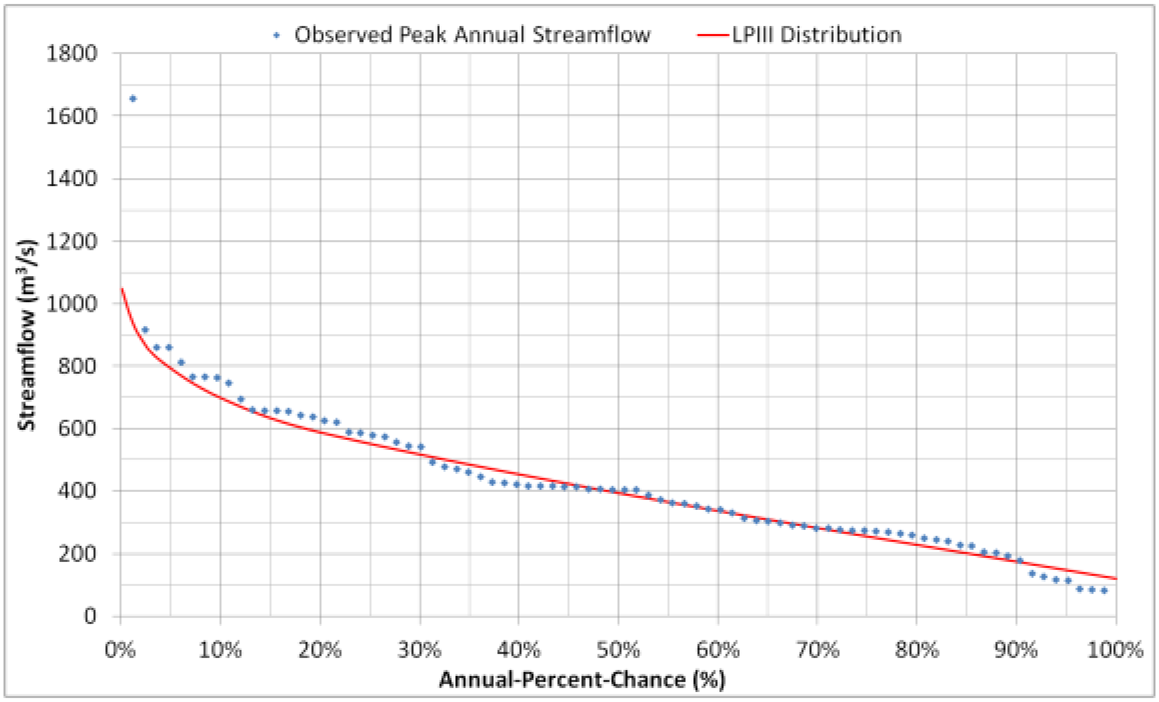

Figure B.1: Observed Peak Annual Streamflow vs. Log-Pearson Type III Distribution

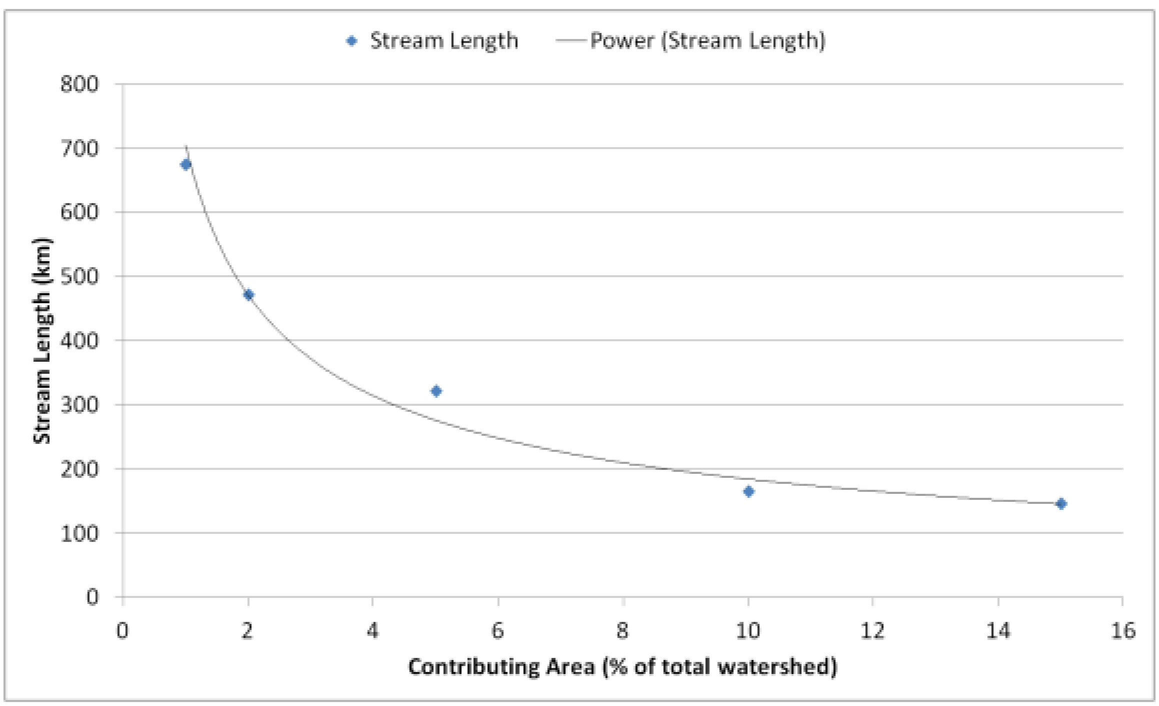

Figure B.2: Stream Length vs. Contributing Watershed Area

Table B.1: Hydrologic Parameter Summary

Flooding Source	Hydrologic Parameter					
	Watershed Area (km^2)	Average CN (-)	T_c (hrs)	Peak Flow (m^3/s)	Runoff Volume (Mm^3)	Hydrograph Duration (days)$^{++++}$
WFK White River^{+}	3026.6	80.5	90.3	954.4	261.9	10.5
WFK White River^{++}	4885.4	80.2	112.8	1037.1	392.5	13.9
Howland Ditch^{+++}	27.5	70.3	19.8	55.4	2.3	2.3

$^{+}$ At 82nd Street bridge

$^{++}$ At Southport Road bridge

$^{+++}$ At Dean Road

$^{++++}$ Duration of flow rates exceeding 0.1 m^3/s

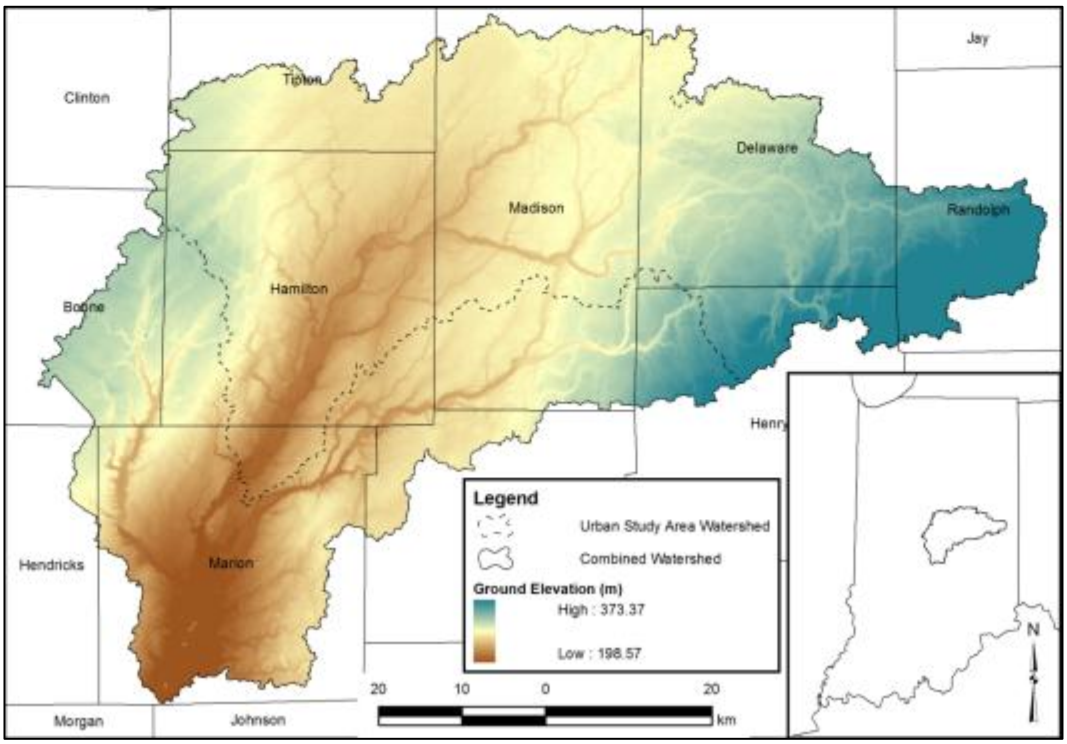

Figure B.3: Watershed Areas and Ground Surface Elevation

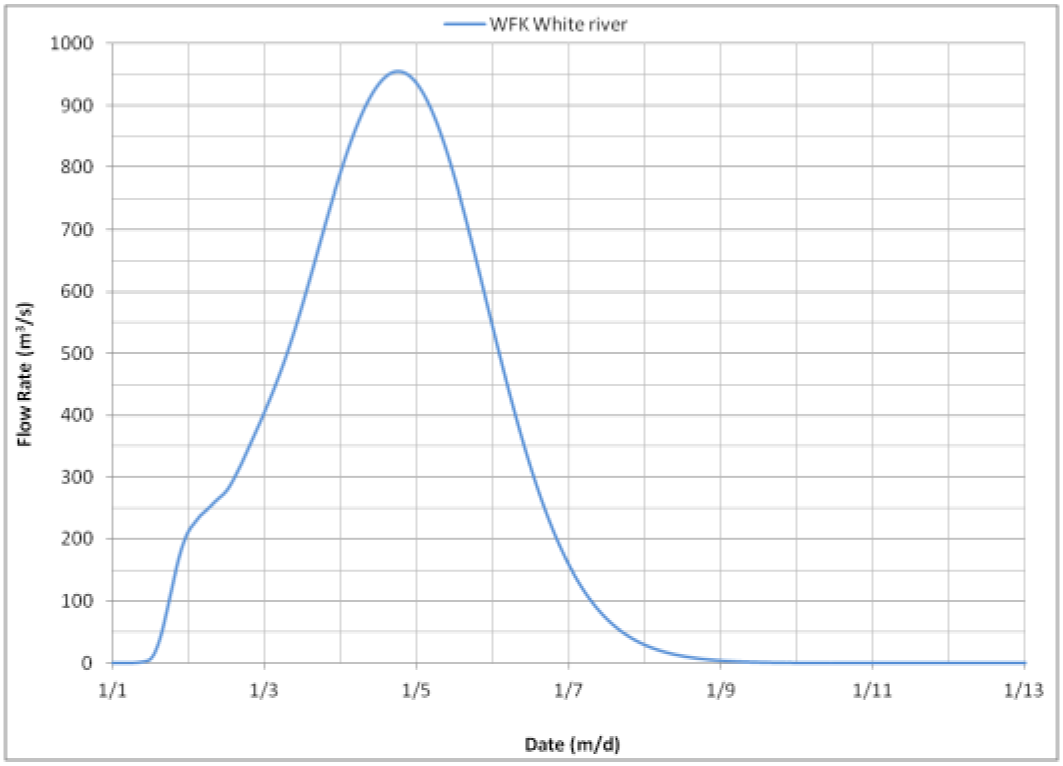

Figure B.4: WFK White River Streamflow Hydrograph at 82nd Street Bridge

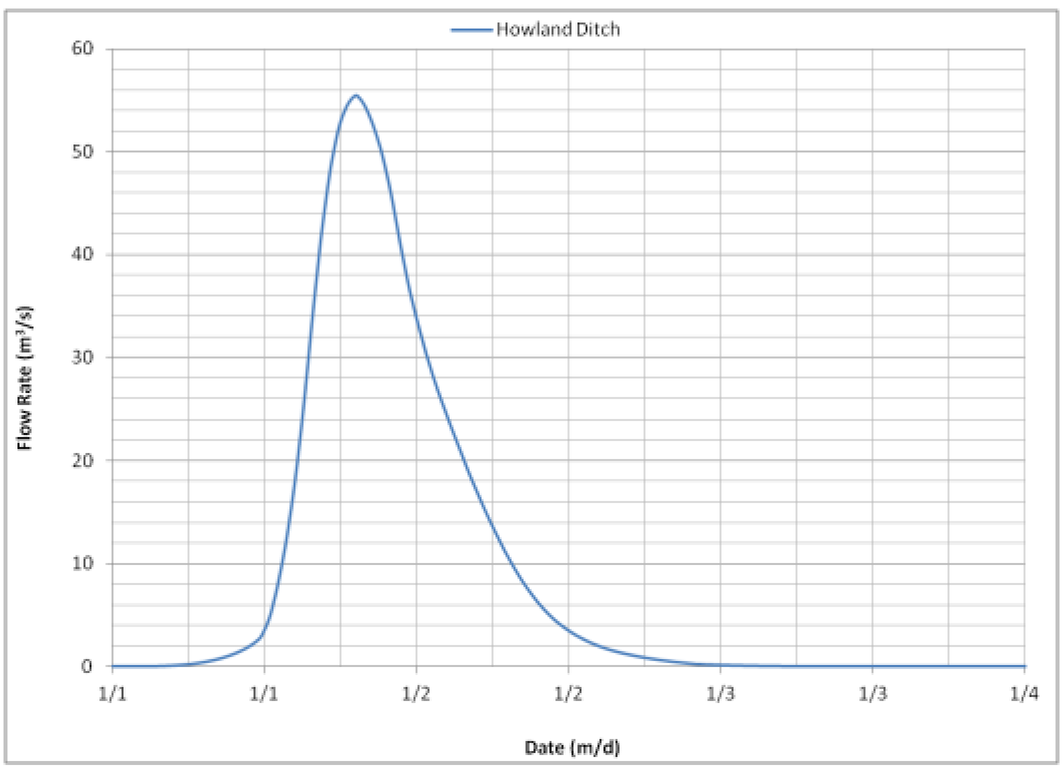

Figure B.5: Howland Ditch Streamflow Hydrograph at confluence with WFK White River

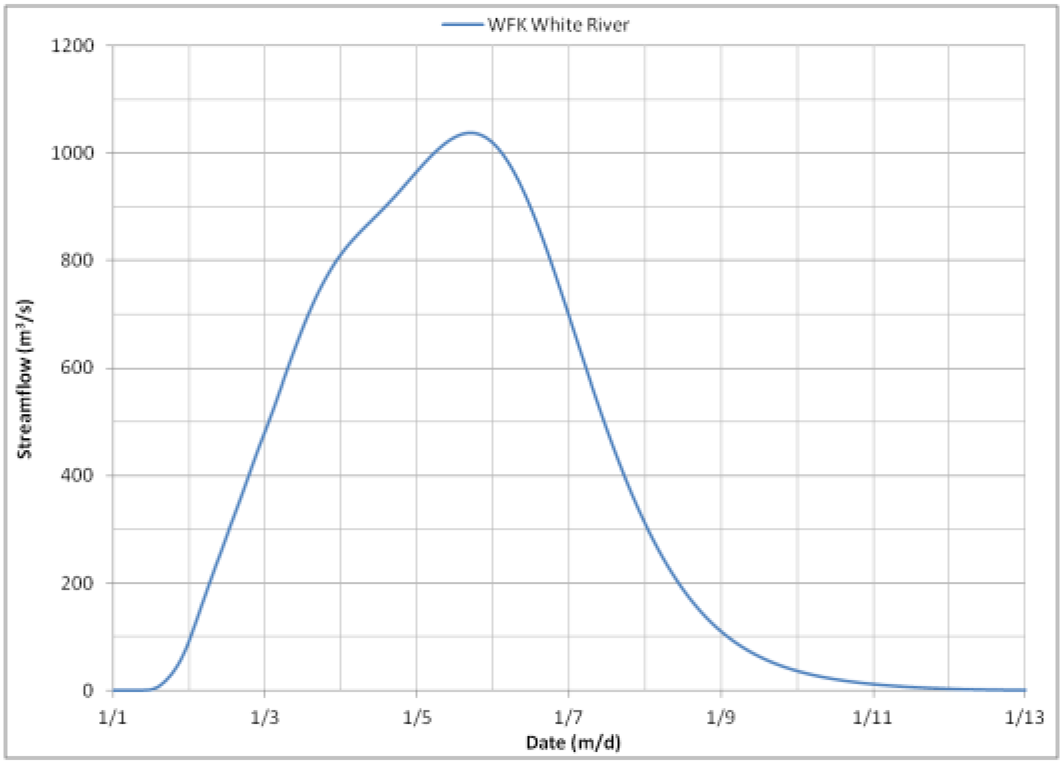

Figure B.6: WFK White River Streamflow Hydrograph at Southport Road Bridge

Appendix C– Hydraulic Modeling Results

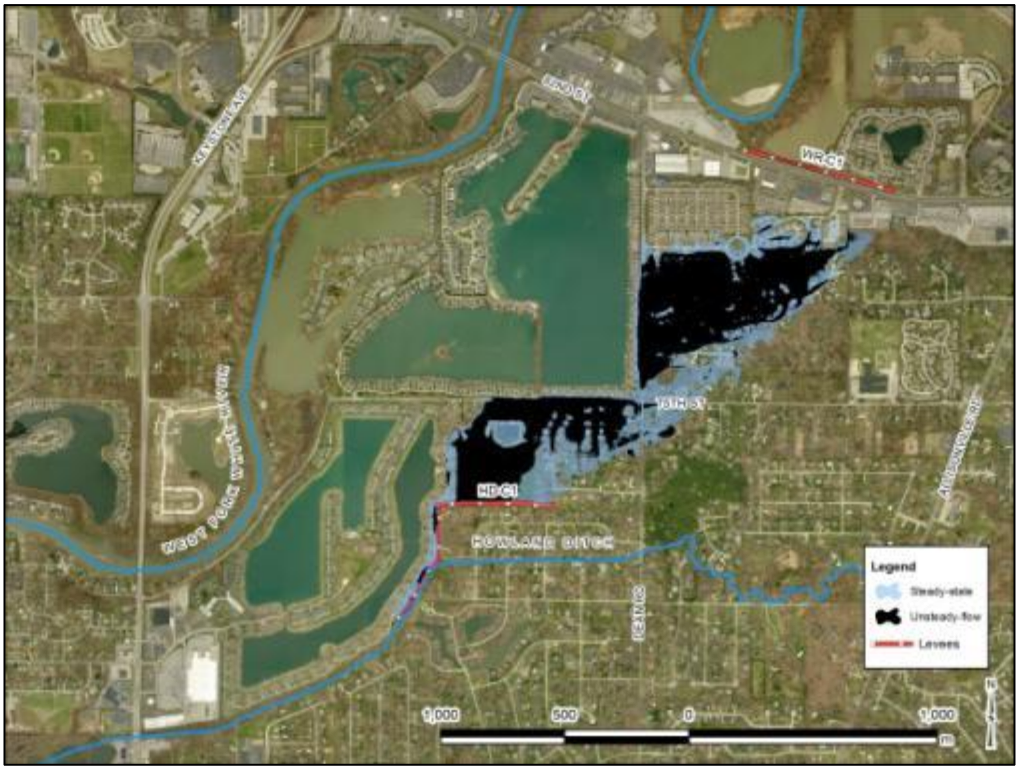

Figure C.1: Unsteady-flow vs. Steady-state Floodplain Boundaries for HD-C1 Levee Breach

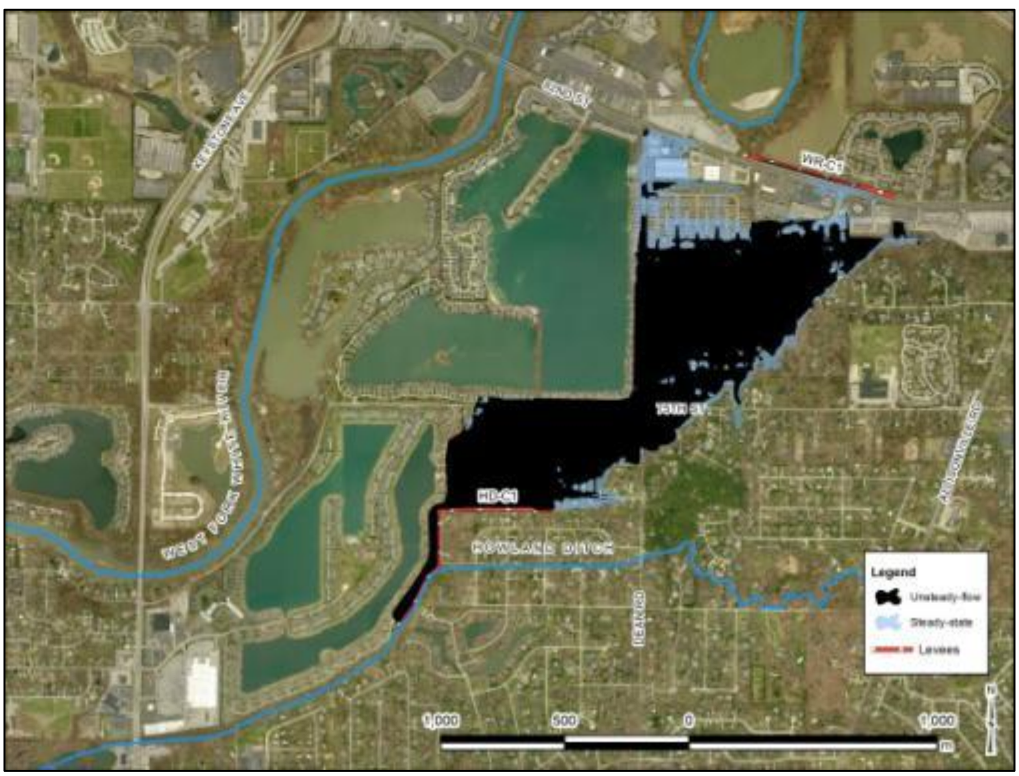

Figure C.2: Unsteady-flow vs. Steady-state Floodplain Boundaries for WR-C1 Levee Breach

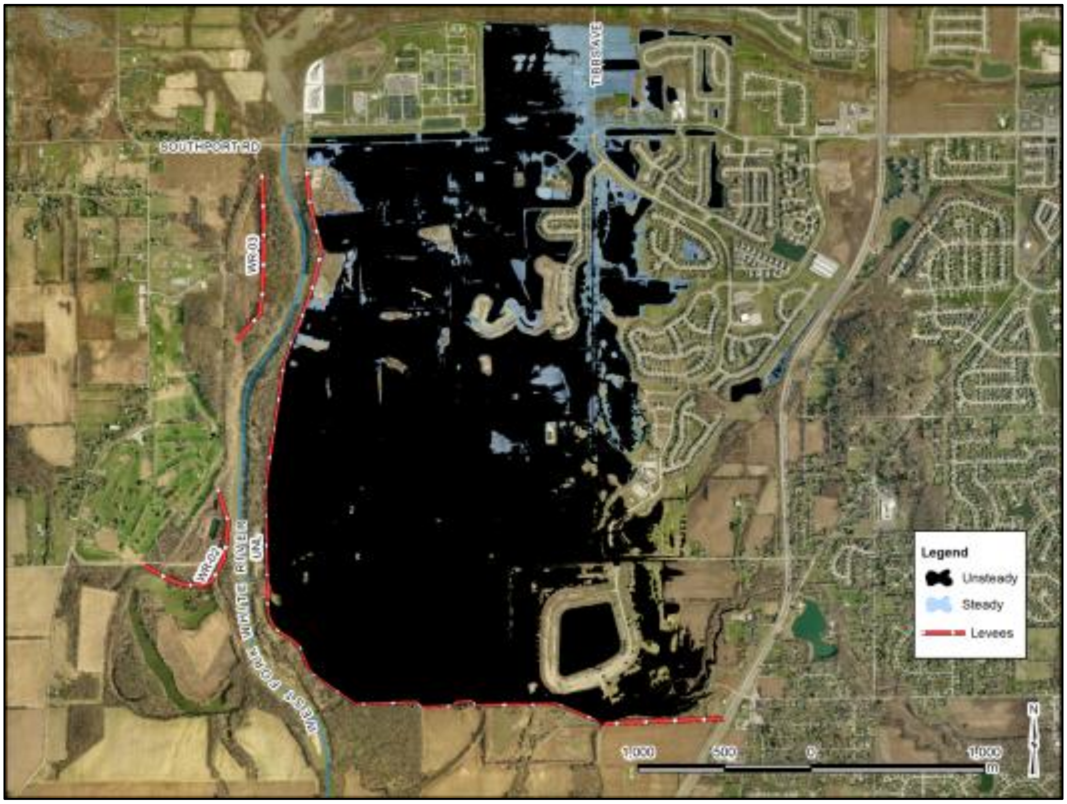

Figure C.3: Unsteady-flow vs. Steady-state Floodplain Boundaries for WR-C1 Levee Breach

Appendix D – Breach Damage Estimation

Table D 1: Summary of HAZUS Breach Damage Estimation

Economic Loss Type	Breach Damage ($ in thousands)				
	WR-C1	HD-C1	UNL	WR-02	WR-03
Building Losses					
Structure	$12,206	$2,538	$5,067	$37	$46
Contents	$12,440	$3,368	$7,091	$27	$35
Income	$148	$44	$73	$0	$1
Total Building Losses:	$24,794	$5,950	$12,231	$64	$82
Agricultural Losses					
Corn	$0	$0	$154	$3	$2
Soybeans	$0	$0	$0	$0	$0
Wheat	$0	$0	$3	$0	$0
Total Agricultural Losses:	$0	$0	$157	$3	$2
Transportation Losses					
Highway	$0	$0	$0	$0	$0
Railway	$0	$0	$0	$0	$0
Total Transportation Losses:	$0	$0	$0	$0	$0
Utility Losses					
Potable Water	$0	$0	$0	$0	$0
Waste Water	$0	$0	$0	$0	$0
Oil Systems	$0	$0	$0	$0	$0
Natural Gas	$0	$0	$0	$0	$0
Electric Power	$0	$0	$0	$0	$0
Communication	$0	$0	$0	$0	$0
Total Utility Losses:	$0	$0	$0	$0	$0
Vehicle Losses					
Daytime Flood	$2,395	$757	$1,158	$5	$7
Nighttime Flood	$2,293	$649	$918	$5	$7
Total Vehicle Losses:	$2,395	$757	$1,158	$5	$7
Displaced Citizens	391	212	174	1	1
TOTAL BREACH DAMAGES:	$27,189	$6,707	$13,545	$72	$91

Appendix E – Improvement Cost Estimates

Opinion of Probable Cost for WR-C1 Levee Improvements

Levee Prioritization Study

Line	Description	Estimated Quantities	Units	Unit Price	Estimated Cost
	Demolition				
1	Tree Clearing	0.0	AC	$ 15,000	$ -
2	Estimated Demolition Cost				$ -
3	**Levees & Bank Stabilization**				
3	Strip and Stockpile Topsoil	500	CY	$ 5	$ 2,500
4	Bench Existing Embankment	125	CY	$ 5	$ 600
5	Place and Compact Fill	6,700	CY	$ 4	$ 26,800
6	Obtain Fill Material	6,700	CY	$ 10	$ 67,000
7	Estimated Levees & Bank Stabilization Cost				$ 96,900
8	**Site Restoration**				
9	Finish Grading	13,100	SY	$ 2	$ 26,200
10	Obtain Topsoil Material	6,050	CY	$ 15	$ 90,800
11	Place Topsoil	6,550	CY	$ 5	$ 32,800
12	Seeding	13,100	SY	$ 1	$ 13,100
13	Erosion Control Blankets	13,100	SY	$ 3	$ 39,300
14	Mulching	13,100	SY	$ 1	$ 13,100
15	Estimated Site Restoration Cost				$ 215,300
16	**Drainage**				
17	Install Backflow Preventer	108	ID	$ 300	$ 32,400
18	Install Concrete Headwall	3	EA	$ 5,000	$ 15,000
19	Estimated Seeding Cost				$ 47,400
16	**Miscellaneous**				
17	Erosion and Sediment Control (5%)	1	LS	$ 18,000	$ 18,000
18	Construction Mobilization/Demobilization (5%)	1	LS	$ 18,000	$ 18,000
19	Construction Contingencies (50%)	1	LS	$ 179,800	$ 179,800
20	Estimated Miscellaneous Cost				$ 215,800
21	**Professional Services**				
22	Design Fees	1	LS	$ 57,600	$ 57,600
23	Geotechnical Investigation	1	LS	$ 9,400	$ 9,400
24	Site Surveying	1	LS	$ 7,400	$ 7,400
25	Estimated Professional Services Cost				$ 74,400
26					
27	**Total for Levee Improvements:**				**$ 649,800**

Notes and Assumptions

1. This cost estimate is conceptual in nature and was derived without a complete topographic survey, geotechnical analysis, or interior drainage investigation
2. All costs are estimates based on the engineer's knowledge of common construction methods and materials.
3. All costs are in 2011 dollars
4. Estimated costs have been rounded.
5. This estimate does not include unforeseen cost increases that may result from shortages in fuel and materials as a result of natural or man made disasters.
6. Cost for off-site borrow assumes that material is available within 5 miles of project site.
7. Improvement costs were performed without the benefit of an interior drainage analysis

Opinion of Probable Cost for HD-C1 Levee Improvements

Levee Prioritization Study

Line	Description	Estimated Quantities	Units	Unit Price	Estimated Cost
	Demolition				
1	Tree Clearing	0.0	AC	$ 15,000	$ -
2			Estimated Demolition Cost		$ -
3	**Levees & Bank Stabilization**				
3	Strip and Stockpile Topsoil	200	CY	$ 5	$ 1,000
4	Bench Existing Embankment	50	CY	$ 5	$ 300
5	Place and Compact Fill	800	CY	$ 4	$ 3,200
6	Obtain Fill Material	800	CY	$ 10	$ 8,000
7		Estimated Levees & Bank Stabilization Cost			$ 12,500
8	**Site Restoration**				
9	Finish Grading	4,800	SY	$ 2	$ 9,600
10	Obtain Topsoil Material	2,200	CY	$ 15	$ 33,000
11	Place Topsoil	2,400	CY	$ 5	$ 12,000
12	Seeding	4,800	SY	$ 1	$ 4,800
13	Erosion Control Blankets	4,800	SY	$ 3	$ 14,400
14	Mulching	4,800	SY	$ 1	$ 4,800
15		Estimated Site Restoration Cost			$ 78,600
16	**Drainage**				
17	Install Backflow Preventer	108	ID	$ 300	$ 32,400
18	Install Concrete Headwall	3	EA	$ 5,000	$ 15,000
19			Estimated Seeding Cost		$ 47,400
16	**Miscellaneous**				
17	Erosion and Sediment Control (5%)	1	LS	$ 7,000	$ 7,000
18	Construction Mobilization/Demobilization (5%)	1	LS	$ 7,000	$ 7,000
19	Construction Contingencies (50%)	1	LS	$ 69,300	$ 69,300
20		Estimated Miscellaneous Cost			$ 83,300
21	**Professional Services**				
22	Design Fees	1	LS	$ 22,200	$ 22,200
23	Geotechnical Investigation	1	LS	$ 4,600	$ 4,600
24	Site Surveying	1	LS	$ 3,400	$ 3,400
25		Estimated Professional Services Cost			$ 30,200
26					
27		**Total for Levee Improvements:**			**$ 252,000**

Notes and Assumptions

1 This cost estimate is conceptual in nature and was derived without a complete topographic survey, geotechnical analysis, or interior drainage investigation
2 All costs are estimates based on the engineer's knowledge of common construction methods and materials.
3 All costs are in 2011 dollars
4 Estimated costs have been rounded.
5 This estimate does not include unforeseen cost increases that may result from shortages in fuel and materials as a result of natural or man made disasters.
6 Cost for off-site borrow assumes that material is available within 5 miles of project site.
7 Improvement costs were performed without the benefit of an interior drainage analysis

Opinion of Probable Cost for UNL Levee Improvements

Levee Prioritization Study

Line	Description	Estimated Quantities	Units	Unit Price	Estimated Cost
	Demolition				
1	Tree Clearing	32.1	AC	$ 15,000	$ 481,500
2	Estimated Demolition Cost				$ 481,500
3	**Levees & Bank Stabilization**				
3	Strip and Stockpile Topsoil	51,300	CY	$ 5	$ 256,500
4	Bench Existing Embankment	12,825	CY	$ 5	$ 64,100
5	Place and Compact Fill	205,000	CY	$ 4	$ 820,000
6	Obtain Fill Material	205,000	CY	$ 10	$ 2,050,000
7	Estimated Levees & Bank Stabilization Cost				$ 3,190,600
8	**Site Restoration**				
9	Finish Grading	158,800	SY	$ 2	$ 317,600
10	Obtain Topsoil Material	28,100	CY	$ 15	$ 421,500
11	Place Topsoil	79,400	CY	$ 5	$ 397,000
12	Seeding	158,800	SY	$ 1	$ 158,800
13	Erosion Control Blankets	158,800	SY	$ 3	$ 476,400
14	Mulching	158,800	SY	$ 1	$ 158,800
15	Estimated Site Restoration Cost				$ 1,930,100
16	**Drainage**				
17	Install Backflow Preventer	108	ID	$ 300	$ 32,400
18	Install Concrete Headwall	3	EA	$ 5,000	$ 15,000
19	Estimated Seeding Cost				$ 47,400
16	**Miscellaneous**				
17	Erosion and Sediment Control (5%)	1	LS	$ 282,500	$ 282,500
18	Construction Mobilization/Demobilization (5%)	1	LS	$ 282,500	$ 282,500
19	Construction Contingencies (50%)	1	LS	$ 2,824,800	$ 2,824,800
20	Estimated Miscellaneous Cost				$ 3,389,800
21	**Professional Services**				
22	Design Fees	1	LS	$ 723,200	$ 723,200
23	Geotechnical Investigation	1	LS	$ 112,100	$ 112,100
24	Site Surveying	1	LS	$ 63,300	$ 63,300
25	Estimated Professional Services Cost				$ 898,600
26					
27	**Total for Levee Improvements:**				$ 9,938,000

Notes and Assumptions

1. This cost estimate is conceptual in nature and was derived without a complete topographic survey, geotechnical analysis, or interior drainage investigation
2. All costs are estimates based on the engineer's knowledge of common construction methods and materials.
3. All costs are in 2011 dollars
4. Estimated costs have been rounded.
5. This estimate does not include unforeseen cost increases that may result from shortages in fuel and materials as a result of natural or man made disasters.
6. Cost for off-site borrow assumes that material is available within 5 miles of project site.
7. Improvement costs were performed without the benefit of an interior drainage analysis

Opinion of Probable Cost for WR-02 Levee Improvements

Levee Prioritization Study

Line	Description	Estimated Quantities	Units	Unit Price	Estimated Cost
	Demolition				
1	Tree Clearing	6.7	AC	$ 15,000	$ 100,500
2		Estimated Demolition Cost			$ 100,500
3	**Levees & Bank Stabilization**				
3	Strip and Stockpile Topsoil	10,700	CY	$ 5	$ 53,500
4	Bench Existing Embankment	2,675	CY	$ 5	$ 13,400
5	Place and Compact Fill	47,300	CY	$ 4	$ 189,200
6	Obtain Fill Material	47,300	CY	$ 10	$ 473,000
7		Estimated Levees & Bank Stabilization Cost			$ 729,100
8	**Site Restoration**				
9	Finish Grading	32,800	SY	$ 2	$ 65,600
10	Obtain Topsoil Material	5,700	CY	$ 15	$ 85,500
11	Place Topsoil	16,400	CY	$ 5	$ 82,000
12	Seeding	32,800	SY	$ 1	$ 32,800
13	Erosion Control Blankets	32,800	SY	$ 3	$ 98,400
14	Mulching	32,800	SY	$ 1	$ 32,800
15		Estimated Site Restoration Cost			$ 397,100
16	**Drainage**				
17	Install Backflow Preventer	24	ID	$ 300	$ 7,200
18	Install Concrete Headwall	1	EA	$ 5,000	$ 5,000
19		Estimated Seeding Cost			$ 12,200
16	**Miscellaneous**				
17	Erosion and Sediment Control (5%)	1	LS	$ 62,000	$ 62,000
18	Construction Mobilization/Demobilization (5%)	1	LS	$ 62,000	$ 62,000
19	Construction Contingencies (50%)	1	LS	$ 619,500	$ 619,500
		Estimated Miscellaneous Cost			$ 743,500
1	**Professional Services**				
2	Design Fees	1	LS	$ 158,600	$ 158,600
3	Geotechnical Investigation	1	LS	$ 39,700	$ 39,700
4	Site Surveying	1	LS	$ 19,900	$ 19,900
5		Estimated Professional Services Cost			$ 218,200
		Total for Levee Improvements:			**$ 2,200,600**

Notes and Assumptions

1. This cost estimate is conceptual in nature and was derived without a complete topographic survey, geotechnical analysis, or interior drainage investigation
2. All costs are estimates based on the engineer's knowledge of common construction methods and materials.
3. All costs are in 2011 dollars
4. Estimated costs have been rounded.
5. This estimate does not include unforeseen cost increases that may result from shortages in fuel and materials as a result of natural or man made disasters.
6. Cost for off-site borrow assumes that material is available within 5 miles of project site.
7. Improvement costs were performed without the benefit of an interior drainage analysis

Opinion of Probable Cost for WR-03 Levee Improvements

Levee Prioritization Study

Line	Description	Estimated Quantities	Units	Unit Price	Estimated Cost
	Demolition				
1	Tree Clearing	5.6	AC	$ 15,000	$ 84,000
2				Estimated Demolition Cost	$ 84,000
3	**Levees & Bank Stabilization**				
3	Strip and Stockpile Topsoil	8,900	CY	$ 5	$ 44,500
4	Bench Existing Embankment	2,225	CY	$ 5	$ 11,100
5	Place and Compact Fill	42,900	CY	$ 4	$ 171,600
6	Obtain Fill Material	42,900	CY	$ 10	$ 429,000
7				Estimated Levees & Bank Stabilization Cost	$ 656,200
8	**Site Restoration**				
9	Finish Grading	27,900	SY	$ 2	$ 55,800
10	Obtain Topsoil Material	5,050	CY	$ 15	$ 75,800
11	Place Topsoil	13,950	CY	$ 5	$ 69,800
12	Seeding	27,900	SY	$ 1	$ 27,900
13	Erosion Control Blankets	27,900	SY	$ 3	$ 83,700
14	Mulching	27,900	SY	$ 1	$ 27,900
15				Estimated Site Restoration Cost	$ 340,900
16	**Miscellaneous**				
17	Erosion and Sediment Control (5%)	1	LS	$ 54,100	$ 54,100
18	Construction Mobilization/Demobilization (5%)	1	LS	$ 54,100	$ 54,100
19	Construction Contingencies (50%)	1	LS	$ 540,600	$ 540,600
20				Estimated Miscellaneous Cost	$ 648,800
21	**Professional Services**				
22	Design Fees	1	LS	$ 138,400	$ 138,400
23	Geotechnical Investigation	1	LS	$ 34,600	$ 34,600
24	Site Surveying	1	LS	$ 17,300	$ 17,300
25				Estimated Professional Services Cost	$ 190,300
26					
27				**Total for Levee Improvements:**	**$ 1,920,200**

Notes and Assumptions

1 This cost estimate is conceptual in nature and was derived without a complete topographic survey, geotechnical analysis, or interior drainage investigation

2 All costs are estimates based on the engineer's knowledge of common construction methods and materials.

3 All costs are in 2011 dollars

4 Estimated costs have been rounded.

5 This estimate does not include unforeseen cost increases that may result from shortages in fuel and materials as a result of natural or man made disasters.

6 Cost for off-site borrow assumes that material is available within 5 miles of project site.

7 Improvement costs were performed without the benefit of an interior drainage analysis

Appendix F – Levee Prioritization Statistics

Table F.1: Priority Ratio vs. Hydraulic Statistics

Storage Area	Damage: Cost Ratio	Levee Priority	Hydraulic Parameters					
			Max Stage (m)	Max Inflow (m^3/s)	Peak Storage (Mm^3)	Flood Area (km^2)	Flooding Duration (days)
HD-C1[U]	26.62	2	222.78	20.11	0.4	0.4	5.0
WR-C1[U]	41.84	1	224.12	24.88	1.2	0.8	6.0
UNL[R]	1.36	3	201.03	159.90	8.7	6.4	8.8
WR-03[R]	0.05	4	202.18	150.86	0.2	0.1	6.1
WR-02[R]	0.03	5	201.15	25.43	0.2	0.1	4.1

[U] Urban Levee Segment
[R] Rural Levee Segment

Table F.2: Priority Ratio vs. Zoning Statistics

Storage Area	Damage: Cost Ratio	Zoning & Development Parameters						
		Residential (%)	Commercial (%)	Special Use (%)	Agricultural (%)	Park (%)	Buildings (#)	Density (Bldg / ac)
HD-C1[U]	26.62	76.6	11.3	12.1	0.0	0.0	880	1.64
WR-C1[U]	41.84	76.6	11.3	12.1	0.0	0.0	880	1.64
UNL[R]	1.36	46.0	3.0	7.1	43.9	0.0	1711	0.61
WR-03[R]	0.05	0.0	0.0	0.0	42.0	58.0	0	0.00
WR-02[R]	0.03	0.0	0.0	0.0	10.3	89.7	4	0.05

[U] Urban Levee Segment
[R] Rural Levee Segment

Table F.3: Priority Ratio vs. Social Statistics

Storage Area	Damage : Cost Ratio	Social Parameters						
		Average Household Income (x $1000)	Avg. Property Value (x $1000)	Buyout Cost (x $1M)	Caucasian (%)	Black / African American (%)	Asian (%)	Other Race (%)
HD-C1[U]	26.62	$60	$279	$279.0	68.2	27.0	1.4	3.4
WR-C1[U]	41.84	$60	$279	$279.0	68.2	27.0	1.4	3.4
UNL[R]	1.36	$53	$55	$94.2	94.8	1.7	1.1	2.4
WR-03[R]	0.05	$0	$0	$0.00	0.0	0.0	0.0	100.0
WR-02[R]	0.03	$0	$13	$0.04	0.0	0.0	0.0	100.0

[U] Urban Levee Segment
[R] Rural Levee Segment

Table F.4: Priority Ratio vs. Breach Damage Statistics

Storage Area	Damage : Cost Ratio	Breach Damage Parameters						
		Building Losses (x $1000)	Agricultural Losses (x $1000)	Transportation Losses (x $1000)	Utility Losses (x $1000)	Vehicle Losses (x $1000)	Displaced Citizens (#)	TOTAL BREACH DAMAGES (x $1000)
HD-C1[U]	41.84	$5,950	$0	$0	$0	$757	212	$6,707
WR-C1[U]	26.62	$24,794	$0	$0	$0	$2,395	391	$27,189
UNL[R]	1.36	$12,231	$157	$0	$0	$1,158	174	$13,545
WR-03[R]	0.05	$64	$3	$0	$0	$5	1	$72
WR-02[R]	0.03	$82	$2	$0	$0	$7	1	$91

[U] Urban Levee Segment
[R] Rural Levee Segment

Table F.5: Priority Ratio vs. Improvement Cost Statistics

Storage Area	Loss : Cost Ratio	Improvement Cost Parameters						
		Demolition (x $1000)	Levees & Bank Stabilization (x $1000)	Site Restoration (x $1000)	Drainage (x $1000)	Miscellaneous (x $1000)	Professional Services (x $1000)	Total Cost (x $1000)
HD-C1[U]	41.84	$0	$8	$79	$47	$83	$30	$252
WR-C1[U]	26.62	$0	$67	$215	$47	$216	$74	$650
UNL[R]	1.36	$482	$2,050	$1,930	$47	$3,390	$899	$9,938
WR-03[R]	0.05	$101	$473	$397	$12	$744	$218	$2,201
WR-02[R]	0.03	$84	$429	$341	$0	$649	$190	$1,920

[U] Urban Levee Segment

[R] Rural Levee Segment

Appendix G– Further Considerations

Lack of funding or excessive delays could prevent levees from being improved before a failure occurs. As described previously, the timeliness of risk assessment results is often more important than the absolute accuracy of the results. The methodology described provides a simplified process of analysis by utilizing existing information and making reasonable and conservative assumptions, which allows for a quick analysis turnaround. The relatively small time requirement to replicate the analysis could allow for a more widespread assessment of levees, which, in turn, could lead to a better allocation of available funding.

The method set forth in this study attempts to establish a relative scale used to compare and prioritize levees based on the economic losses expected from a levee breach. The process involves the development of hydrologic and hydraulic models, assessment of existing infrastructure, determination of breach damages, and the estimation of improvement costs for the levee.

Key Components of Risk Assessment Strategies

The use of risk assessments to help prioritize levee rehabilitation requires the determination of key factors in the design of the risk assessment strategy. Risk assessments must address three main features of a given situation, in the case of this analysis, a flood. The first component of a risk assessment is the determination of the hazard level, or the probability that the event will actually occur. The vulnerability to the hazard is the likelihood that the event will have an unfavorable outcome. The third component is the consequence, which is often described by the negative cost impact of a given scenario (Baecher, 2009). Though the consequence is often identified by the negative outcome, it should be noted that many events have both positive and negative results. Natural floodplains provide a number of services; flood protection, pollution abatement, groundwater recharge, and increased biodiversity are all potential benefits of maintaining an active and natural floodplain (National Research Council, 2004). In addition to the three key components, risk assessments must consider the use of the information developed.

Other factors, beyond the easily quantifiable components, can have an effect on the design of a risk assessment. The intended use of the risk assessment can affect the desired output. For instance, if it is known that a levee will suffer a catastrophic failure if a flooding event occurs, the absolute magnitude and precision of the expected damages is less important than releasing the results of the study as quickly as possible to allow for proper action. Another important aspect of risk assessment concerns the audience to which the study is directed. Stakeholder involvement is essential for risk assessment design to provide meaningful results (National Research Council, 2009) Budget committee members and members of historical societies will likely view the expected flood damages from a levee breach quite differently. One could anticipate that the budget committee members would be interested in how much economic loss is expected compared to the cost to prevent the damages; one might also surmise that the historical society members would be focused on what it would take to protect the artifacts within the damaged area, with little regard for the cost of such protection. The risk assessment methodology presented in this study is designed with these key components in mind. The applicability of the methodology to any given levee system is also heavily considered when determining what output will best address the risk associated with deteriorating levees.

The highly variable nature of the physical systems responding to the hazard creates a need to establish default decision options within the risk assessment. For the purpose of comparability and prioritization, the suggested default decision options have been selected to be impartial and conservative in nature. Conservative assumptions are warranted when significant data gaps exist. Use of conservative assumptions is reasonable in priority-setting assessments, such as the methodology described in this study (National Research Council, 1983). The hazard level has been fixed for all analyses performed. The 1%-annual-chance rainfall event has been selected to serve as the design event. The vulnerability of the levee system has also been fixed; the methodology detailed herein assumes that all levees will breach during the 1%-annual-chance flooding event. This leaves only the consequence to serve as the differentiating component of the risk assessment. The intended use of this process is to help decision makers maximize the return-on-investment of flood control infrastructure rehabilitation. As a result, the data output developed is highly quantitative and economically based.

Current Standards of Practice

A large inventory of floodplain maps have been developed through FEMA's various flood map development programs, which are all a part of the National Flood Insurance Program. These programs have been instituted to increase the public awareness of the hazard of flooding as well as to help reduce the risk to human life and human investments. The results of flood mapping projects must be submitted to FEMA for approval based on the guidelines developed by FEMA. Currently, flood maps are developed using a steady-state flow regime to generate expected water surface profiles. Several steady-state flow rates are used in the analyses; peak runoff from the 0.2%, 1%, 2%, and 4%-annual-chance events is simulated (FEMA, 2005). The water surface profiles are then compared to existing topography to determine the extents of the flooding along the river system for the various risk classes. Flood areas being delineated within the 1%-annual-chance flood boundary are considered high risk areas, or Special Flood Hazard Areas.

Flood mapping for areas protected by levee systems are assessed in a slightly different manner. If levees are deemed capable of providing protection for the 1%-annual-chance event, and meet the requirements of 44 CFR 65.10, the area behind the levee is shaded as and labeled 'Zone X' or 'Zone AH'. Shaded 'Zone X' denotes areas that have a moderate risk of flooding, or risk between 0.2% and 1%-annual-chance of occurrence. 'Zone AH' identifies areas that are at risk of shallow flooding during the 1%-annual-chance event. In the event that a portion of a levee is not capable of meeting the requirements of 44 CFR 65.10, the base flood elevation behind the levee is set equal to the water surface elevations generated by modeling the system as if the levee did not exist (FEMA, 2005). The floodplain boundaries delineated behind levees essentially provide the flooding if the levee were to have never been built, or if the levee were to be removed entirely.

Risk assessments for areas prone to flooding currently utilize FEMA's HAZUS program to develop the expected economic risk posed by flooding. Currently regulatory models are used to determine the extents of flooding. Depth grids or floodplain boundaries are used to within the HAZUS program to produce the expected losses associated with the defined flooding. For areas that do not have hydrologic and hydraulic models, floodplain extents can be determined in HAZUS using simplified hydrologic and hydraulic methodologies.

In 2009, the Interagency Performance Evaluation Task Force completed an in depth risk assessment of Hurricane Katrina. The post-hurricane assessment was used to predict the future risk to flooding based on redevelopment scenarios. Experts from multiple engineering fields were employed to precisely capture the reaction of the hydraulic system and the impacts of the flooding. Model calibration was performed using extensive survey data collected after the flooding had subsided and recovery efforts were underway. In addition to monetary loss estimates, the study attempted to identify the social, cultural, and environmental impacts of the flooding. Loss-of-life modeling was performed for the impacted areas and compared to the real-world data. (IPET, 2009) The volume and quality of data used within the risk assessment, as well as the expertise of the individuals carrying out the assessment was unprecedented.

Hydrologic Model Development

The development of hydrologic models is a complex and well-studied topic. Many federal and state agencies, as well as some municipalities have their own particular guidelines for hydrologic model development. Two options are considered for hydrologic model development, the use of existing regulatory hydrologic models, and the development of entirely new hydrologic models. To maintain the economic feasibility of performing hydrologic analysis on an extensive levee system, it is desirable to use the most up to date and available data, rather than creating an entirely new hydrologic model. These models are updated on a decennial basis or more frequently, and require significantly less modification and data collection than areas without base flood models. In addition to reducing the amount of work required to develop a model, the use of existing regulatory models ensures that the model has been adequately reviewed and tested for unsound assumptions and errors. Larger streams and rivers typically have hydrologic models available from Flood Insurance Studies required by FEMA. The obvious benefits to using existing hydrologic models validate their use as the default option.

As an alternative to using existing hydrologic models, new models can be developed using the guidelines set forth by the agency or government body having jurisdiction over the study area. This option is much less desirable due to the amount of data collection and time necessary to review the models; however, regulatory models do not exist for all streams, especially smaller tributaries which are heavily impacted by the larger waterways to which they contribute. In cases where a new model must be developed, the highest resolution data available can be utilized. High resolution soil, land use, and topography data are publicly available for the United States from the USGS Seamless Server. Higher resolution data may be available at the county or local level. USACE's HEC-GeoHMS extension for geospatial software provides an efficient and cost effective way to develop complex hydrologic models which are capable of using high resolution data to describe the hydrologic response of a given watershed. The use of computer-aided watershed delineation and curve number generation can help to eliminate subjective decision making, helping to achieve the goal of unbiased assessments. Rainfall from the National Climate Data Center on-line rain gage data and streamflow records from USGS stream gages can be used to calibrate and verify the accuracy of the hydrologic model. However, by comparing two or more levees to each other, the absolute accuracy of the data becomes less important compared to the relative accuracy between the different analyses. To help maintain relative accuracy, the same approach, either calibrated or un-calibrated, should be used for all levees being prioritized in a given study. The quality of the data available to calibrate all models should be considered prior to determining whether a calibrated or un-calibrated model should be employed.

Though the 1%-annual-chance flooding event was not intended to serve as a design parameter, the use of the event as the minimum protection required for exclusion from mandatory flood insurance has caused it to be used to design many levees (ASCE, 2009). Due to the fact that many levees include this as a design consideration and the requirement for all FEMA accredited levees to provide the 1%-annual-chance level of protection with adequate freeboard, the simulation of the 1%-annual-chance event is an obvious choice for design flow rate to be used to prioritize levee repairs.

Hydraulic Model Development

As with hydrologic modeling, hydraulic modeling of riverine systems is highly complex topic that has been well-studied; guidelines for model development have been created by federal and state agencies, as well as some municipalities. The default option of for hydraulic model development is the use of the current regulatory model from the FIS study. Unlike the hydrologic model, the hydraulic model must be modified from its original form in every case. Levee, storage area, and unsteady-flow data must be added into the model to perform the suggested analysis. Regulatory hydraulic models are used to determine the base flood elevations which are then used to determine the specific flood hazard zones. The proposed method assesses the risk associated with a levee based on the assumption that the levee will breach during a flooding event.

For stream reaches that do not have regulatory models, or for reaches with outdated models, new hydraulic models must be developed, or updated to characterize the stream response. Jurisdictional guidelines can be utilized to assist in developing high quality hydraulic models. The development of models based on jurisdictional guidelines would also allow for the creation of an accepted regulatory model with considerably less effort and capital investment. New hydraulic models should be developed in accordance with the guidelines developed by the agency or government body having jurisdiction in the area. The highest resolution data should be used to develop the hydraulic model. Terrain and other physical components which contain vast amounts of variability can be incorporated using USACE's HEC-GeoRAS extension for geospatial software used to develop complex hydraulic models for open channel systems. By using high resolution DEM's, aerials, and land use maps, floodplain topography, surface roughness, and obstruction characteristics can be assimilated at much lower costs than performing physical surveys of the areas of interest. Though the information provided in DEM's and land use datasets is not as specific as the information provided by a site survey, the loss in absolute accuracy is acceptable due to the tradeoff for cost effectiveness, comparability to other levee studies, and timeliness of results. If flood inundation maps or historic flood elevations exist for the area, the model can be checked to prevent unreasonable or unprecedented flood profiles from being used for assessing the flooding risk. Once the standard components of the hydraulic model have been assimilated, a levee breach must be included in the newly developed model for comparison purposes as well as for determination of expected losses during a levee breach scenario.

The hydraulic model must be modified to incorporate the levee breach and the storage area created behind the levee. The topic of levee breach development is an ongoing area of study. Widely accepted breach progression models do not exist; however simplistic assumptions are often used for these types of analyses. Variability in the materials and construction types used to construct levees, the level of deterioration experienced by the levee, and the many modes of levee failure create a physical system that is difficult to adequately and accurately describe; thus necessitating the use of simplifying assumptions. The most commonly used hydraulic modeling software, HEC-RAS, has the capability of simulating a levee breach based on user stipulated input which governs the modeled development of the levee breach. A storage area can be linked to the levee breach to simulate the protected area behind the levee which would be inundated by the breach wave and stored floodwater. A geospatial analysis of the protected area must be performed to develop a relationship between river stage and the inundation area to utilize this method of levee breach modeling. By simply determining the relationship between the area behind the levee that is below the stage of the flooding source, an implicit assumption of perfect hydraulic connectivity is made; additional consideration may be necessary for interior areas which are highly compartmentalized to establish a more realistic stage-storage model. Without additional research and more complex modeling, the protected area must be modeled as a storage area without an outlet for the water to escape. This assumption is not valid for levee systems that have operable interior pumping systems which remove seepage and interior runoff; however, it is likely that these pumping systems would be disabled by damage to their power source, as was the case with Hurricane Katrina (IPET, 2009). Thus the assumption that interior pumping stations will not perform properly is conservative.

The location and timing of the levee breach and the breach formation assumptions dictate the impact of a levee breach. In keeping with the conservative approach set forth in this method, the levee breach is placed at the upstream-most portion of the levee. For earthen levees, breach characteristics can be established by the jurisdictional mandates for dam breach analysis. Though dams and levees serve different purposes, their construction and response to elevated water levels are similar, and the use of standardized assumptions improves the comparability of the different levees in the prioritization study. Levee systems having floodwalls at the upstream end should be modeled as suffering a catastrophic failure; instantaneous failure of the concrete or sheet pile walls should be assumed. Standard breach criteria for these types of structures are also often specified by dam safety officials. To account for variability in the level of protection provided by levees based on levee height, the levee breach simulation can be modeled to begin when water surface elevations reach the crest of the levee at the breach location, or when the flooding source hydrograph reaches its peak.

As more sophisticated levee breach models become available and cost effective, the method of simulating levee breaches can be adjusted to more adequately determine the extent and characteristics of the breach wave. For the purposes of this method, the important factor is the standardization of the assumptions made when simulating the levee breach. Using a set of default assumptions allows for more equity in the determination of breach damages.

Breach Damage Estimation

Losses resulting from a levee breach come in many forms. The infrastructure supporting the local community as well as the impact on local businesses and workers must be considered. HAZUS, a widely accepted and applicable computer application developed by FEMA, contains methodology for estimating damage and losses resulting from natural disasters. The flood hazard assessment tool within the HAZUS program allows the user to determine the impacts of a flood on a specific study area. HAZUS uses multiple algorithms to determine the impacts of flooding on a specified area. Building, utility, agricultural, critical facility, income, and vehicle inventories are considered in conjunction with user provided flood information. A large number of depth-damage curves are contained in the model to appropriately account for the loss that could be expected given the level of flooding experienced by an area. The model estimates damage to the general building stock (i.e. homes and businesses), essential facilities (i.e. water/wastewater treatment facilities and hospitals), lifeline systems (i.e. bridges and major roadways), vehicles, agriculture (i.e. crop damage) and various indirect economic impacts. Using the user specified flood depths paired with the depth-damage curves, the economic loss is estimated for the specified study area (Scawthorn, 2006). By using such a widely accepted and applicable program, the process of estimating the economic loss associated with a levee breach is streamlined and made consistent between the levee segments to be prioritized.

Improving Deficient Levees

Levee improvement costs are entirely dependent upon the level of degradation present in a levee system. The deficiencies of a levee are based on criteria set forth by FEMA and the Army Corps of Engineers. Typically, visual inspections are used to assess the condition of levees. In cases where the materials physically forming the levees come into question, soil borings are performed to allow for a geotechnical analysis. Visual inspections of floodwalls are also performed as a means to identify compromised sections. Similar to earthen levee inspections, the visual inspection is sometimes supplemented by soil borings near floodwalls to determine the adequacy of the supporting soil materials as well as concrete cores to determine the floodwall's condition.

Some of the most common deficiencies are the existence of woody vegetation on the levee slopes and crest, inadequate freeboard, lack of adequate backflow prevention, steep slopes, and damage from animal burrows and erosion. In the case of floodwalls, foundation settling can lead to cracking and instability, lack of erosion protection for floodwall foundations can lead to undermining and serious instability, and poor maintenance can lead to joint deterioration and oxidation of reinforcing steel and eventual structural failure. Freeboard requirements for levees are set for in 44 CFR 65.10; this code states that levees must have three feet of freeboard above the 1%-annual-chance base flood elevation, plus an additional foot of freeboard within 100 feet of a bridge or other form of constriction to flow. A minimum of six inches of additional freeboard above the three foot requirement must be present at the upstream end of the levee, with the additional freeboard requirement tapering to the three foot minimum at the downstream end of the levee segment (FEMA, 2008). Base flood elevations, or the flood elevations created during the 1%-annual-chance flooding event, should be gathered from the Flood Insurance Study (FIS) profiles published by FEMA. Levee side slopes are recommended to be a maximum of 2 Horizontal: 1 Vertical (2H:1V); however the side slopes may need to be decreased from this value based on a slope stability analysis using the geotechnical properties of the soil constituents of the levee, or for maintenance reasons. Damage from animal burrows and erosion can be detected from a visual inspection of the levee.

Improvement Cost Assessment

To determine the cost associated with correcting deficiencies, a case-by-case analysis must be performed. No two levees are identical in layout, material composition, or vulnerabilities due to infrastructure being placed through or below the levee embankment. A determination of the full extent of the deficiencies is required to develop the associated cost of improving the levee. Several means of determining levee improvement costs can be utilized. Bid tabulations from public works projects can often be retrieved from local municipal records. These tabulations must be used with caution to ensure that appropriate unit rates are used when translating improvement quantities to improvement costs. A more suitable means of determining improvement costs may be to consult with one or more local contracting firms. Construction costs can vary considerably by region due to material supply issues as well as the skill of the labor force. By utilizing the expertise of contractors who are familiar with performing the type of work required to repair a deficient levee, some of the uncertainty and ambiguity often associated with cost estimating can be eliminated. It is also likely that the cost of estimating the improvement costs will be cheaper based on the experience and skill set of a contractor as compared to a design engineer or analyst. The development of a partnership between the engineering firm performing the analysis and the contracting firm can lead to a lower cost and a more accurate assessment of the expected improvement cost.