Risk to life due to flooding in post-Katrina New Orleans

Alissa Miller
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Advisor:

Dr.ir S.N. Jonkman

M.Sc. Commitee:
Dr. Ir. Mathijs Van Ledden
Prof.dr.ir J.K. Vrijling
Dr.Ir. O.A.C.Hoes

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Executive Summary

In August of 2005, Hurricane Katrina resulted in massive flooding to the city of New Orleans, Louisiana. 80% of the city was left submerged causing immense economic and social damage as well as loss of life. In response to the catastrophe, the US government committed to upgrade the flood protection to protect against storms with at least a 100 year return period. Consequently, in the period since Katrina, significant effort and fiscal resources have been employed to provide this level protection. The city is now better protected than at any previous point in its history.

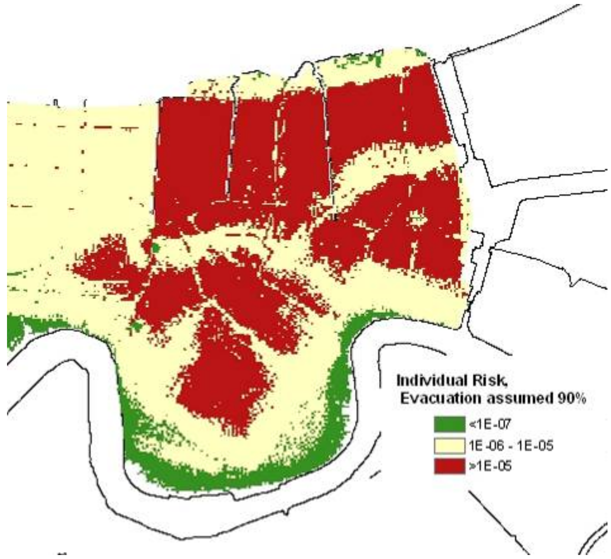
Yet, despite significant progress made to reduce the city's risk to flooding, the situation remains that a city of dense urban population, growing economic infrastructure, and great cultural significance is protected to a 1/100 year level. What level of risk remains considering the upgraded system of protection? And what does the management of flood protection consist of in view of the increasing risk of flooding in the future?

To provide information regarding such questions, this study quantifies and evaluates the residual risk to the city. While risk can more commonly be quantified monetarily, the risk is quantified in terms of life loss. In engineering applications, risk is often quantified as a factor of the probability of an events occurrence and the potential consequences of that event. Therefore the possible impacts of an event, no matter how small the probability, can be taken into consideration in the development of flood management.

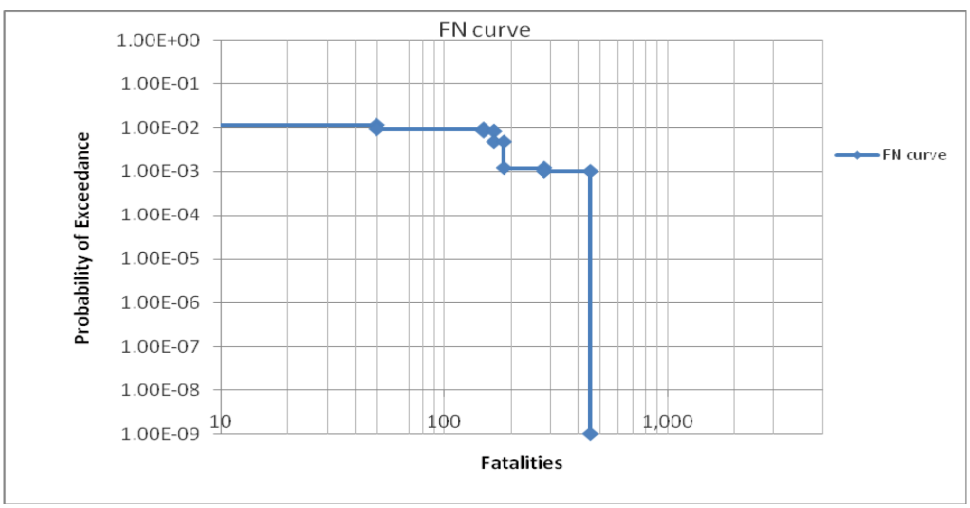
The evaluation of risk to life is determined through a 'scenario' based approach, simulating potential flood events. For each scenario, a probability of occurrence is evaluated, and a life loss estimate is attributed to the event. In lieu of a full reliability analysis, the probability of each scenario is based upon the design guidelines for the upgraded protection system along with expert judgment. The scenarios are selected to model the various ways the New Orleans metro bowl can flood, and model flooding due to both hurricane and high river events. The defined flood scenarios are developed in a two-dimensional hydrodynamic flood model, resulting in flood characteristics for each scenario. These characteristics are used as input into a life loss model based on previous flood data from Hurricane Katrina (Jonkman et al., 2009), which relates flood characteristics to mortality rate for each scenario. The mortality rate is correlated spatially to the population values and estimates a life loss consequence. In this way, the effects of evacuation are considered in the consequence estimate.

The mortality rates for the simulated events range from .05% for a canal breach scenario, leading to 50 fatalities, to 2% for a breach due to a high river event, leading to 450 fatalities. Consequences due to this scenario are most disastrous as the duration of the flood wave is several weeks and the polder will continue to fill until the breach is closed.

The assumed event probabilities and the consequences of each scenario are combined to result in risk to life. The risk is quantified in various metrics such as individual risk (IR) and societal risk. The individual risk expresses the probability that an individual at a certain location gets killed due to a flood. The individual risk level for New Orleans while assuming a 90% evacuation rate, is found to exceed a level of 10^{-5} per year for much of the metro bowl.



Societal risk, which describes the probability of a large, multi fatality event, can be expressed most simply as the expected value E(n) of fatalities, or the average fatalities in a year. In this study, the expected value is calculated as the weighted average of the scenario outcomes. This results in roughly 2 fatalities per year. Societal risk can also be depicted in an FN curve, which plots the probability of exceedance (in a year) as a function of the number of fatalities. The simulated scenarios are plotted to determine this curve. It can be seen the river event scenario, the worst modelled scenario, leads to 450 fatalities and has an estimated flooding probability of 1/1000 per year. The other flood scenarios with smaller numbers of fatalities, ranging from 50 to 180, have a probability of exceedance of 1/100 to 1/1000 per year.



The risk is compared against existing criteria found in literature for flood risk and against acceptable life risk in other civil sectors. The resulting IR values are found to be relatively low when compared to some accident probabilities such as driving a car (10^{-4} per year), and limit criteria proposed by the USACE (also of 10^{-4} per year). However, the IR is found to exceed further considered limit criterion as discussed in literature, such as by Vrijling, 1995, which proposed a limit of 10^{-5} per year).

For the evaluation of societal risk, the resulting FN curve is compared against limit lines. The results for societal risk as determined in this study are found to exceed discussed limit lines proposed both in literature and those considered acceptable in various industries on a national scale in both the United States and the Netherlands.

As the quantified risk does not fall within acceptable limits found in literature and industry, the effectiveness of reduction measures is analyzed. Measures to reduce the probability of flooding, such as increased protection are considered. Protection levels that would meet the various ‘acceptable’ proposed risk standards are derived and would result in protection levels between 1/1000 and 1/50,000 year. Measures to reduce the load are also recommended to reduce event probability, but are not discussed in detail in this thesis.

Measures to reduce consequences are discussed such as evacuation and the raising of existing infrastructure. The cost effectiveness of reduction measures can be compared by expressing the CSX of the various measures, the cost of saving an extra statistical life. Comparison of the cost effectiveness show that the cost effectiveness of raising protection further may not be as cost effective as the implementation of other measures such as home elevation and improvements of emergency management to increase evacuation effectiveness.

In conclusion, the current situation of New Orleans is that of a densely populated area having a high loss of life potential with a 1/100 year flood protection level. The current risk of the city to flooding exceeds tolerable or acceptable risk criterion for both individual risk and societal risk found in literature and other industries. This indicates the necessity of further discussion regarding the city’s risk to flooding. Thus while decisions regarding flood risk are complex and involve many factors such as perception and economic considerations, the results presented here can provide input for flood management in the future.

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1 Introduction

1.1 Background

Hurricane Katrina reached the southern coast of the United States on August 29, 2005. The storm was ranked as a ‘category 3’ on the Saffir-Simpson Hurricane wind scale as maximum wind gusts reached up to 140 mph (225 km per hour). The hurricane-force winds extended outward 120 miles (190 km) from the storm’s center when it made landfall, and produced a storm surge of up to 27.8 feet (8.5m) along the coast of the Gulf of Mexico. The flood protection of the coastal city of New Orleans, Louisiana, was overwhelmed by the surge and catastrophic flooding occurred throughout the metropolitan areas. The impacts of the storm led to one of the worst natural disasters in American history. Economic damages to the city are estimated to have totalled more than 20 billion dollars, and the loss of life caused by flooding number roughly 1100.

Category	Wind speed	Storm surge
5	≥250 (km/h)	>5.5 (m)
4	210–249 (km/h)	4.0–5.5 (m)
3	178–209 (km/h)	2.7–3.7 (m)
2	154–177 (km/h)	1.8–2.4 (m)
1	119–153 (km/h)	1.2–1.5 (m)
Trop. storm	63–117 (km/h)	0–0.9 (m)
Trop.depr.	0–62 (km/h)	0 (m)

Figure 1-1 Storm path of Hurricane Katrina (Wikipedia, NASA, US National Hurricane Center, 2005, Jonkman 2007).

In the wake of the disaster, the United States government committed to provide the people of New Orleans 1/100 year flood protection system by June of 2011. Thus far, innovative engineering and substantial costs have been employed to provide this protection level to New Orleans. It can be said the city now stands better protected today than any other point in its history.

However, an important discussion remains regarding the residual risk associated with upgraded protection system. The risk of the city to flooding is expected to increase in the future due to several factors, with the most significant factor expected to be economic and population growth as the city continues to rebuild. The updated 1/100 year protection level is defined by pre existing national flood insurance policy. However, by considering the risk specific to the city of New Orleans, an acceptable or optimized protection level could be discussed.

Other engineering fields, such as the field of nuclear and chemical engineering, utilize risk in decision making regarding hazardous events. As flooding can also be considered a large consequence -low probability event, the same approaches can be applied in the context of flood protection (Vrijling, 2001). Risk, in the context of engineering applications, is a function of the probability and the consequences of an unwanted event. A risk based design in the context of flood protection allows the potential consequences of a flood to be incorporated into decision making.

The consequences of a flood event can be quantified in various categories, economic damages, life loss, environmental loss, cultural loss and others. When such consequences are combined with the system reliability, the risk can be described in various 'risk dimensions', such as economic risk and risk to life.

In regard to flooding, quantifying this loss allows investment in flood protection to be weighed against the benefits of risk reduction (Vrijling,1995). For example when the economic risk is quantified, an economically optimal risk level and in turn, an optimized protection level can be determined. Such is the approach used in the design of flood protection for the low lying areas of the Netherlands. After the disastrous flooding in 1953, updated flood protection was designed by considering potential **economic** losses balanced with the cost of flood protection. The optimal level of protection is the point at which the total costs in the system are minimized. The result is a protection design of economically justified optimal levels of protection for the country (Van Dantzig , 1965).

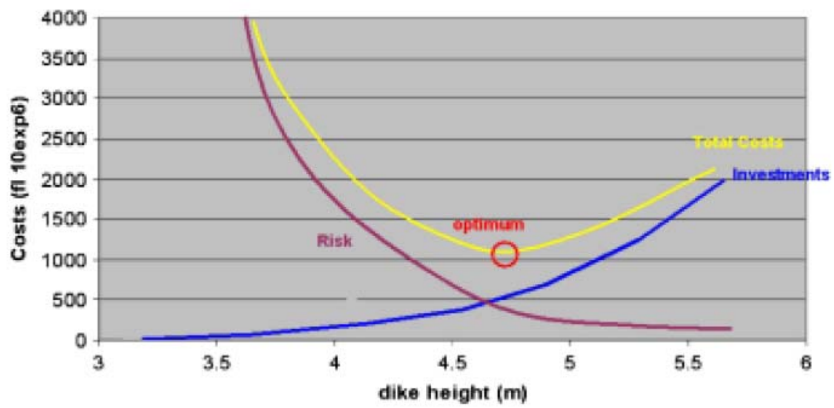


Figure 1-2 **Principal of economic optimization approach (Van Danzig, 1965).**

Figure 1-3 Flood protection levels in the Netherlands circa 1960 based on economic risk principles (Link, 2009)

While quantifying damages monetarily allows a cost benefit approach to decision making, a more complete approach to the design of flood protection incorporates other risk dimensions as well. The consideration of **risk to life** allows the loss of life potential of a flood event to be involved in the decision making process. When quantified, the risk to life can be compared against acceptable risk levels as found in other industry fields and proposed in literature.

Finally, as well as providing optimum and acceptable criteria for flood protection levels, the quantification of risk allows the effects of potential risk reducing measures to be quantified, an important aspect of managing and mitigating the overall risk. Thus while the decision making regarding flood risk management is complex, including social, political and economic factors, evaluating the risk provides valuable technical data to be used in decision making for flood protection.

1.2 Problem

Thus while New Orleans' newly upgraded system flood protection system provides a 1/100 year level of protection, the basis for this level can be improved upon. As mentioned, the 1/100 year flood event has been historically considered by the National flood insurance program (NFIP) to be the 'base' flood event for an acceptable risk level. Therefore a 1/100 year level has served as a risk standard within the government's regulation of flood protection. The use of this standard implies flood events of a lower frequency are too infrequent to worry about (Moser, 1997), and indeed, this standard may apply to areas of relatively low population and economic value. However, it may not be sufficient in more densely populated urban areas. And, with an anticipated increase in risk level in the future, information regarding the current risk is important for decision making.

1.1 Objectives

This study aims to build on the current discussion regarding the city's residual risk to flooding. Considering the catastrophic loss of life due to the Hurricane Katrina, as well as due to flood events in other parts of the world, the safety of human life is an important aspect in decision making on flood protection. Therefore this work researches the following regarding the upgraded protection system for the city of New Orleans:

- **What is the current risk to life associated with the upgraded protection system?**
- **How does the evaluated risk compare to 'acceptable' risk to life criterion found in literature and other engineering applications?**
- **What measures can be applied to reduce the risk in the context of the New Orleans flood protection system and what is their effectiveness?**

In order to answer these questions, a general risk assessment is carried out. The approach framework for the quantification is similar to that found in FLORIS project (Flood Risk In the Netherlands) (Reidstra, 2010). The steps involved in this assessment will be discussed in chapter 2.

1.1 Outline

The remainder of the report is outlined as follows: A brief overview of the applied approach is described in chapter 2. The definition of the system and qualitative analysis phases of the risk assessment are discussed in chapter 3. Chapter 4 describes the quantification of the risk to life of New Orleans. In chapter 5, this risk estimate is evaluated by comparing it to criterion in literature and estimates of other industries. The final step is risk reduction, discussed in Chapter 6 where measures to mitigate risk are briefly analyzed. Conclusions and recommendations are presented in chapter 7.

2 Risk Approach

A brief overview of the risk approach used in this paper is described in this chapter. The work expounds on current and past work regarding the application of risk in regard to the management of flood protection for New Orleans.

2.1 Ongoing application of risk approaches

The application of risk based principals can be found in current engineering guidelines of the United States Army Corps of Engineers (USACE), the federal agency responsible for providing flood protection. These guidelines recognize risk as a ‘useful index for communicating’ to the public so that they might understand the randomness of hydraulic events and the probability that their property will flood (USACE, 1996). More recently since Katrina, risk approaches have been discussed with regard to updated tolerable risk levels for dam and levee safety guidelines (USACE, 2010). Such guidelines will be discussed in the comparative discussion of this work.

Also, reliability analyses of New Orleans’ upgraded protection system were carried out by the Interagency Protection Emergency Team or **IPET,** a team which assembled after Hurricane Katrina. This effort, developed by academic, government and private entities, applied state of the art methods to studied the risk to the city. The results of this work consist of inundation maps for selected return periods and associated life and economic consequences. These maps have been made public in an effort to increase risk communication. The results of the IPET will be compared in this work.

Finally, of note is a study estimating the economic risk of the city carried out in 2008 (Jonkman et al, 2008). The study applied the risk based approach used to design the flood protection for the Netherlands in 1965 to the situation of New Orleans. While preliminary, the results of the study estimated an economically justified protection level in the order of a 1/1000 to a 1/5000 level of protection for the city.

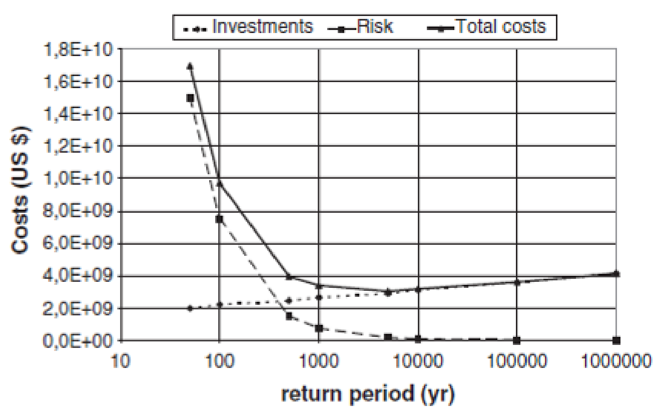


Figure 2-1 Results of economic optimization for central new Orleans polder (Jonkman et. al., 2008)

Thus it is within this context of ongoing research that this thesis looks to estimate the risk. The following section will describe the approach used to do so.

2.2 Risk Assessment

A risk assessment is generally thought to consist of the identification, quantification and evaluation of risk. A general approach to risk management (Jonkman, 2007, CUR, 1997) is followed in this thesis consisting of the flowing steps. The steps are briefly described.

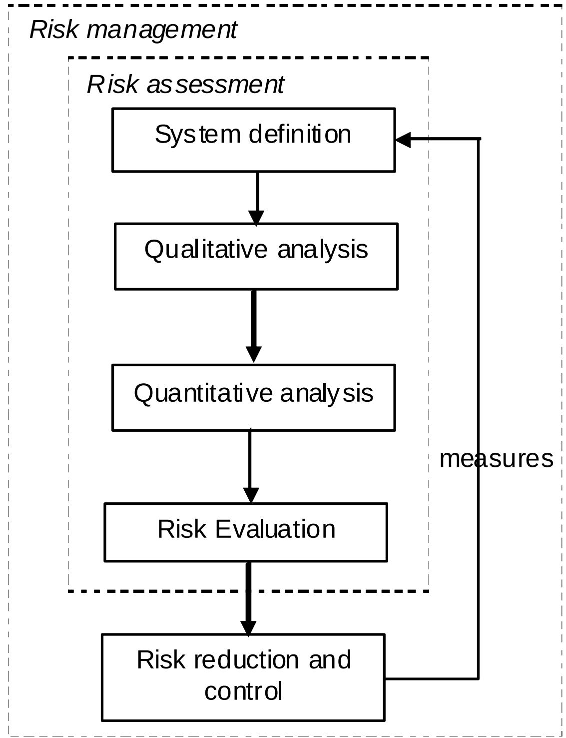


Figure 2-2 Steps of a Risk Assessment (Jonkman, 2007)

2.2.1 System Definition

The system to be analyzed is first defined and described. In this step, simplifications and assumptions are used to model the system based on the scope of the analysis. The **location** of the system and its interaction with the surroundings is considered. The physical components of the system are defined. In this case this consists of the New Orleans upgraded **hurricane protection system**. This complex system is comprised of a variety of subsystems, structures and components that include earthen levees, floodwalls, pump stations, canals, gate closures and other components. Existing riverine protection that protects from river flooding is part of the system definition as well.

2.2.2 Qualitative analysis

A description of the qualitative risk analysis, or the various hazards considered in the evaluation follows next. Risk hazards to loss of life could include for example, exposure to nuclear impacts or contaminated substances. Here, the risk to life solely from flooding is considered.

The city is threatened by flooding due to various hydraulic mechanisms. High river discharge, storm surge and severe rainfall can all contribute to the risk to flooding. In this study, the risk due to high river discharge and hurricane surge only is considered.

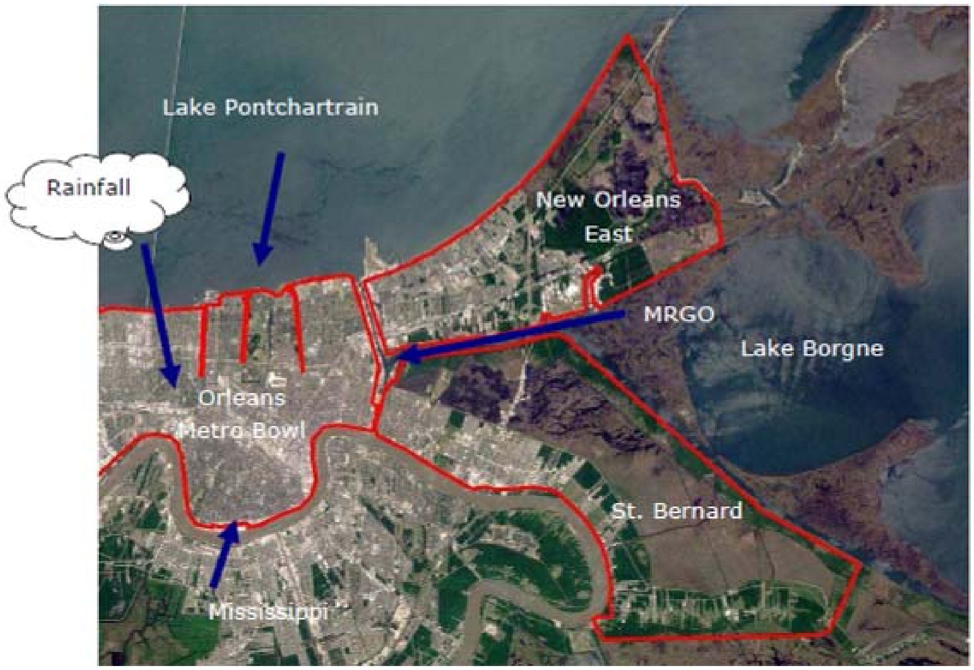


Figure 2-3 Protection system of New Orleans and the potential sources of flooding (blue arrows) (Aalberts, 2008).

The failure modes of the system will be described in this step. As hurricane surge and high discharge are the considered hazards, failure modes caused by each specific hazard are discussed.

2.2.3 Quantitative Analysis

The quantitative analysis is made up of several sub steps. As mentioned, in engineering applications, risk is commonly considered a function of the probability and the consequences of an undesirable event. To determine a quantitative value of risk to the city, this study uses a 'scenario based approach' where possible flood events are simulated and their associated consequences estimated (Jonkman, 2007). The quantification is accomplished in four steps:

1) Estimate system reliability of the system
2) Simulate defined flood scenarios
3) Determine consequences
4) Quantify Risk

1) Estimate reliability of the system

Reliability can be described as the probability of proper functioning of a system, or component within a system. Reliability in the context of flood protection is the probability the flood defence to withstand the design load. A complete reliability analysis considers all possible load conditions and strength (or resistance) parameters of the protection system. Loads consist of the hydraulic parameters which act on the system such as wave and water level. Strength parameters can include soil or material characteristics of the defence. As load and strength parameters are characterized by means of stochastic variables, and their uncertainties can be taken into account by their probability distribution functions. Failure of the protection is represented by the overlapping area of the load and resistance distributions seen below depicting where the load exceeds the resistance.

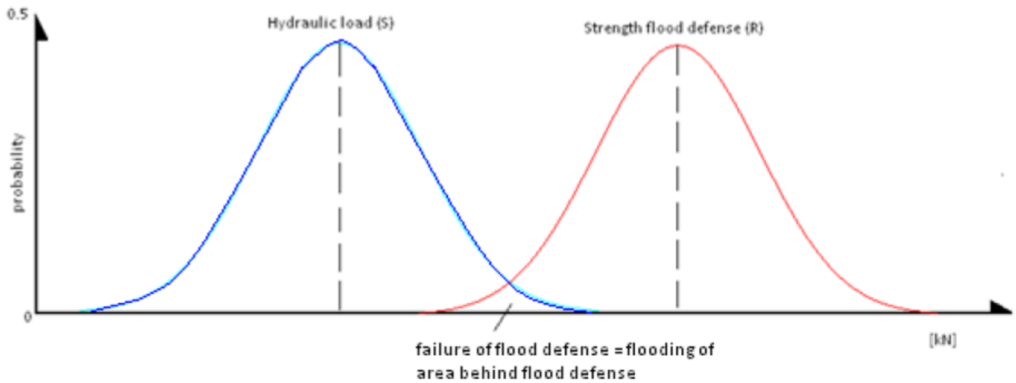


Figure 2-4 Probability density function of inundation of a polder (Hoss, 2010)

For simplification in this thesis, failure probabilities will be assumed based on the design guidelines of the system in combination with expert judgement.

2) **Simulate defined flood scenarios**

Simulations of the flood events (scenarios) are developed through the use of the SOBEK 1D-2D overland flood model. A previously developed model has been used which includes a DEM (digital elevation model) of the current topography and the updated protection system in the area of interest. The boundary conditions are modelled as one dimensional (1-D) and the overland flow is modelled as two dimensional (2-D). The hydraulic boundary condition of each scenario models that of an event with a return period at which failure is assumed to occur. Thus there is a direct relation between the water levels and the probability of failure of the defense. Other model input includes how fast the breach will grow, and the initial and final breach parameters.

The model results provide information on the pattern and extent of potential flooding, and characteristics of flood waters such as depths and velocities. Below hydraulic characteristics from a modelled river breach scenario for South Holland are plotted.

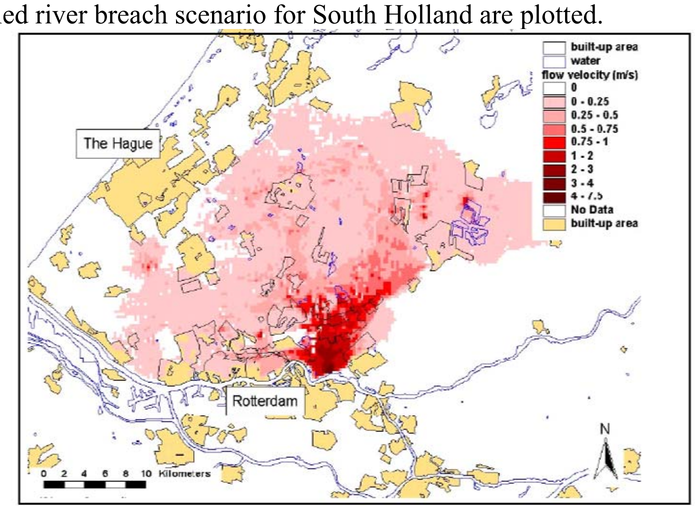


Figure 2-5 Maximum flow velocities (m/s) as computed by SOBEK for simulated breach in South Holland (Asselman and Heynert, 2003).

3) Determine consequences

Consequences can be defined in several forms: economic, cultural, environmental and life loss. In this thesis, an empirical method for estimating the **loss of life** due to flooding is applied. The method consists of life loss functions based on data from Hurricane Katrina (Jonkman 2007, Maaskant 2007). The functions relate the hydraulic circumstances of flooding to the number of fatalities. Thus using the results of simulation output, a fatality estimate is determined for each simulated scenario.

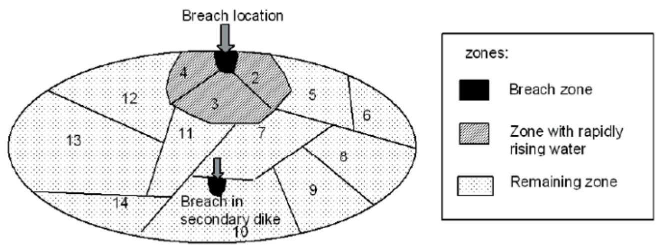


Figure 2-6 Mortality functions based on classified hazard zones as defined by specified criterion (Jonkman, 2007).

4) Quantify Risk

The resulting simulations provide a distribution of outcomes. When combined with the results of the reliability analysis, the risk can be expressed by various risk measures. A risk measures can be described as "an expression or graph which quantifies or depicts risk as a mathematical function of the probabilities and consequences of a set of undesired events". The risk measures play an important role in communicating the risk analysis, and provide the basis for evaluation of risks and decision-making (Jonkman, 2007)

As mentioned, this thesis quantifies risk measures that describe the risk to life consequence. Other risk measures, such as economic risk, could be quantified in further work.

Individual risk is a common measure used to describe the risk to life of a single person. It is defined as the probability of death of an unprotected person at a given location. Therefore it is a characteristic of a location, characterizing the level of risk spatially. Individual risk data can be applied for zoning and land use planning and is generally depicted on a risk map showing contours of risk level (Jonkman, 2007).

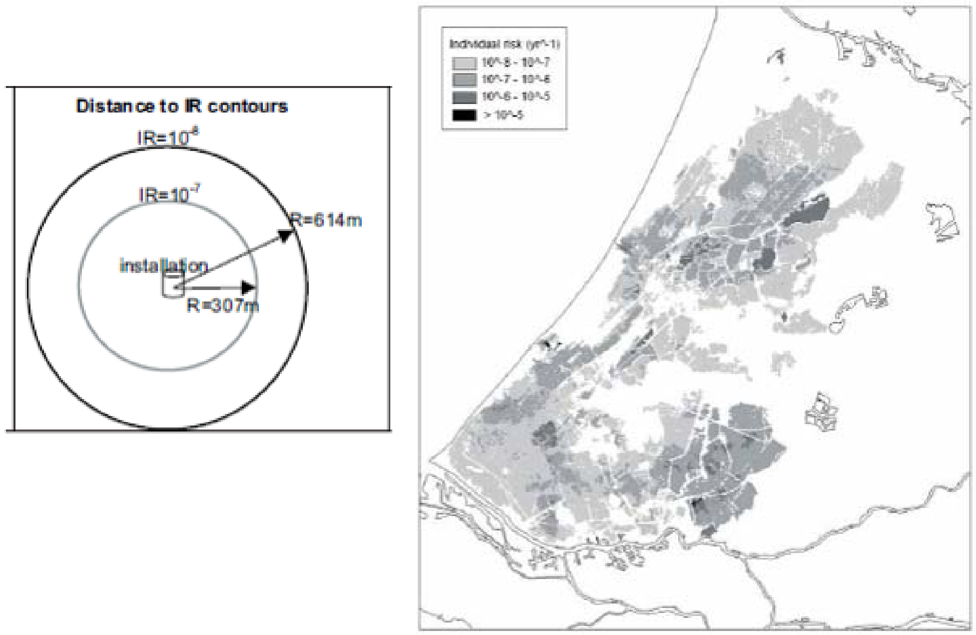


Figure 2-7 IR contours mapped with increaceing risk levels approaching the location of a hazardous installation (left) , and levels of IR plotted spatially due to flooding in South Holland (right) (Jonkman, 2007).

Risk to life is also commonly quantified as **societal risk**, representing the risk of a large amount of fatalities due to a single event. It can be expressed most simply as the **expected value** E(n) of fatalities per year. This represents the weighted average of all possible outcomes . The weight is the probability of each outcome.

It is expressed as the probability of exceedance (in one year) of a certain number of fatalities due to one event in a given population. As societal risk is a function of the ‘exposed’ population, considerations for evacuation and shelter are considered.

A common way of depicting societal risk is with an FN curve, which plots the probability of exceedance of a certain number of fatalities. Data from real and manmade events can be plotted in such a curve.

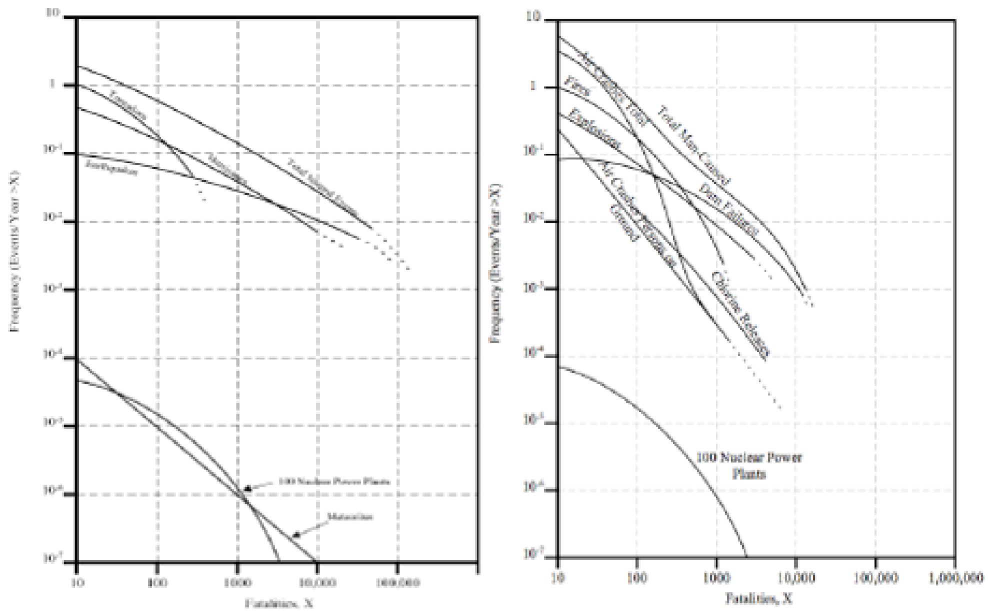


Figure 2-8 Natual Events (left and man made events (right) causing fatalities (USNRC, 1975) .

In this study, the FN curve plots the probability and consequence estimates of the simulated flood scenarios are modelled.

2.2.4 Risk Evaluation

The quantified risk is evaluated by comparing it to limits of ‘tolerable’ or acceptable risk criterion. Proposed limits found in literature and industry are discussed for both flood events and other hazards.

For example, the FN curve used to visualize the societal risk, can be compared to criterion depicted in the form of a limit line. One such limit line is that proposed in the Dutch safety standards. In the approach, the limit line is a mathematically represented function dependent on the frequency of exceeding number of fatalities (CUR 1997).

$$1-F_{Ndj}(x) < 10^{-3}/x^2 \qquad \text{(eq 1-1)}$$

Where $1-F_{Ndj}(x)$ is the frequency of exceeding the number of fatalities in a year as a result of activity i and location j. x is the number of fatalities at location j as a result of activity i.

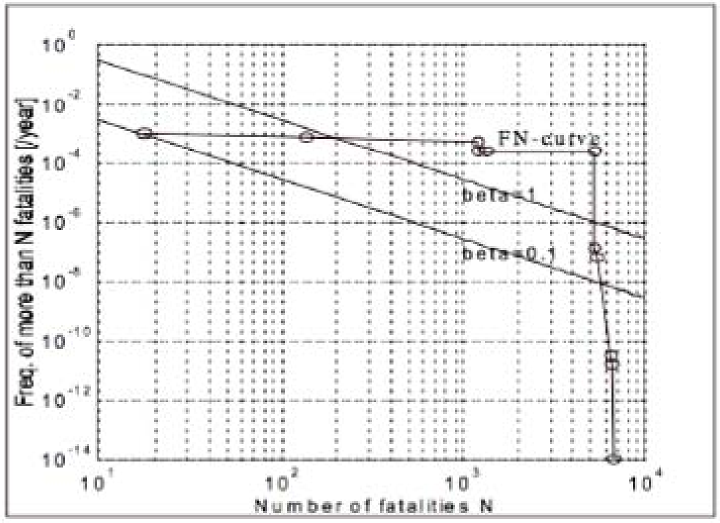


Figure 2-9 FN curve for flooding of the Brielse polder (Vrijling and Van Gelder 1997).

The societal risk of New Orleans to flooding can be compared to other common infrastructure risks. Such comparisons provide insight into the order of magnitude for an acceptable risk, and determine whether the risk is acceptable and assist in deciding appropriate flood safety standards.

2.2.5 Risk Reduction

Ultimately, the risk assessment allows decisions to be made regarding the acceptability of the risk. Decisions to reduce the risk are supplemented with information regarding the effectiveness of reduction measures. This effectiveness can be determined with the risk evaluation as it provides a basis from which alternatives can be evaluated. Measures will be considered which reduce the probability of occurrence or reduce the resulting consequences. The cost effectiveness of risk reduction measures can be evaluated and provide added information regarding reduction.

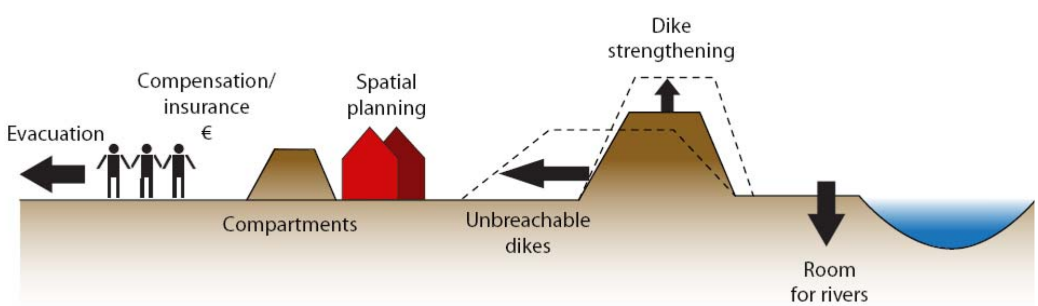


Figure 2-10 Possible risk reduction measures (source: S.N.Jonkman, taken from Hoss, 2010).

The next chapters describe the steps of the analysis in detail.

3 System Definition and Qualitative Analysis

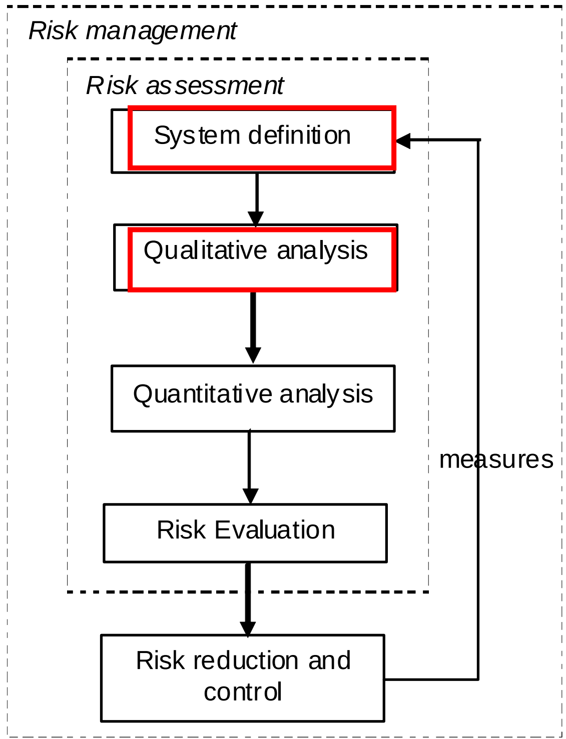


Figure 3-1 Steps of a Risk Assessment (Jonkman, 2007)

This chapter defines the system to be analyzed in the risk assessment consisting of overview of the natural environment surrounding New Orleans and the hydraulic systems that threaten it. The assumptions made in defining and modelling the protection system are defined and the boundaries of the modelled system are discussed.

3.1 New Orleans

The city of New Orleans is located in the deltaic plain of the Mississippi river where it discharges into the Gulf of Mexico. The city is bordered by Lake Pontchartrain to the north and Lake Borgne to the east. Coastal wetlands lie south of the city and serve as a buffer to the sea.

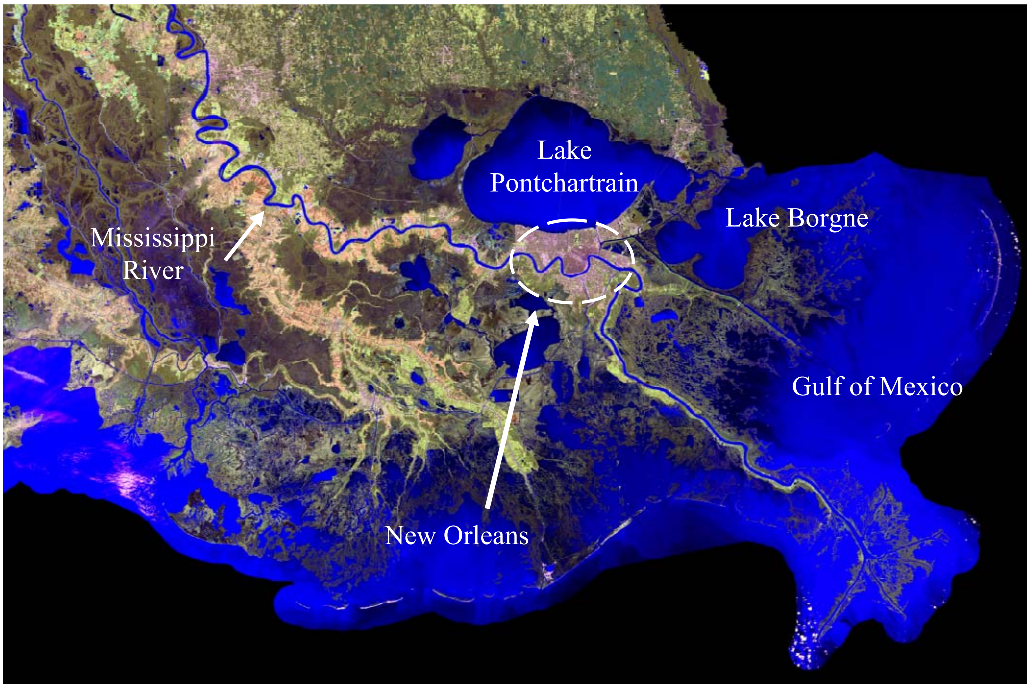


Figure 3-2 Surroundings of New Orleans (Landsat imagery 2002).

In addition to being largely surrounded by water, the elevation of the city further leads to its vulnerability to flooding. The initial settlement (1718) of the city was built on the banks of the river, the highest area of land. Over time, the city expanded away from the river and onto lower lying areas. This expansion was enabled by canals dug in the 1930's draining the once marshy low lying areas so that they could be inhabited by the city's residents. The draining of the marshlands led to further subsidence of the area.

Currently, it is estimated over half of this metropolitan area sits below sea level, in some areas up to 9.4 feet/ 3 meters below sea level. Continued subsidence of the area is estimated at a rate of 5-7 millimeters per year on average (Campenella, 2006) .

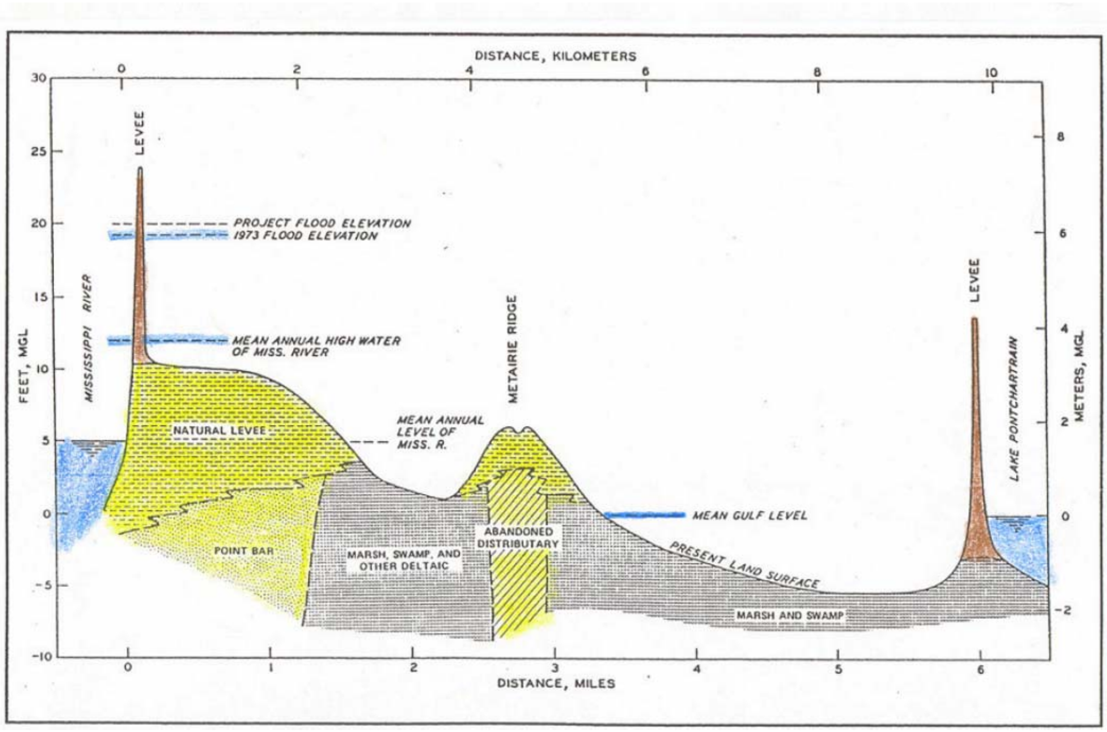


Figure 3-3 Schematic of cross section (Kolb and Saucier, 1982, taken from Rogers, 2006).

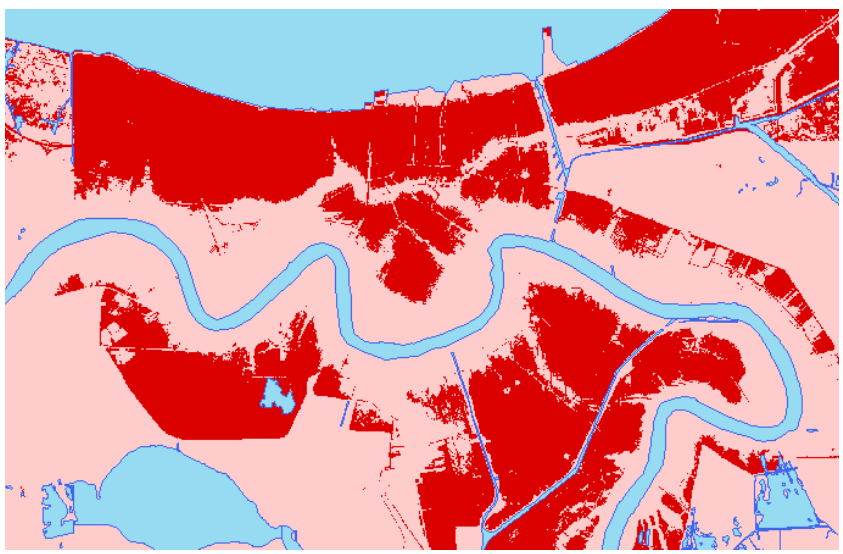


Figure 3-4 Metropolitan New Orleans above and below sea level (Dark red are areas below 0 MSL (mean sea level). Based on 2009 USACE LIDAR.

The metropolitan area of the city, including neighbouring suburbs, is home to roughly 1,235,650 people. As a low lying urban area surrounded by water, the city is vulnerable to flooding. Multiple mechanisms can lead to flooding of the urban areas, storm surge, river flooding and excessive rainfall. For this study, only two such mechanisms are considered, storm surge and river flooding.

3.1.1 Flooding due to storm surge

Due to New Orleans' increasing proximity to the Gulf of Mexico, flooding due to surge poses an increasing threat. During summer months, the warm temperature of the gulf enables the formation of tropical storms. These low pressure storm systems cause the water to rise and forced in the direction of the storm winds. Upon reaching land, the elevated water is pushed against the impermeable land and 'bunches up' creating even greater surge heights.

In the deltaic region surrounding New Orleans, the bathymetry is shallow and gently sloping. Surge is not able to disperse as it might in an area of deeper bathymetry (NOAA). When the hurricane reaches landfall, the surge is pushed on land by the storm.

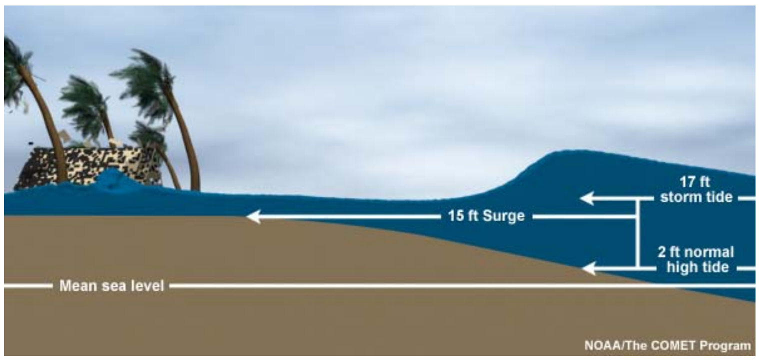


Figure 3-5 Surge over a shallow shelf (NOAA, 2005).

Surge has caused destructive flooding to New Orleans repeatedly in the past. One of the most direct hits to the city was Hurricane Betsy in 1965, which inundated the coast. 164,000 New Orleans homes were flooded and 58 lives were lost in the Orleans parish alone (Boyd 2011).

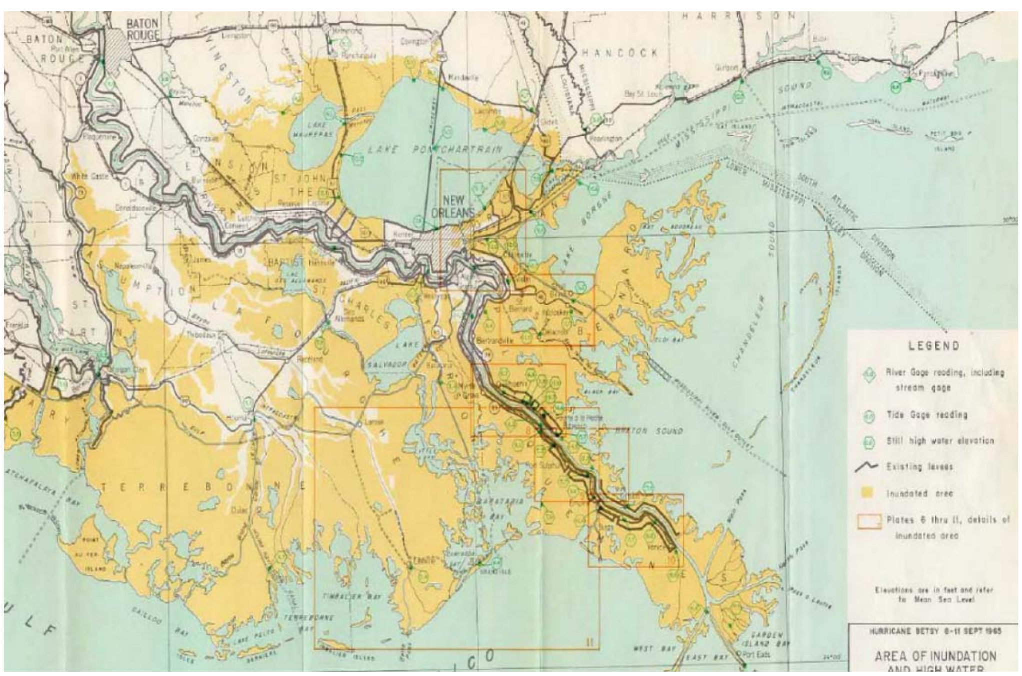


Figure 3-6 Inundation cause by Hurricane Betsy (Rogers, 2006).

Despite improvements in flood protection after Hurricane Betsy, the surge produced by Hurricane Katrina in 2005 once again overwhelmed the flood protection at the time. Surge heights reached the highest ever recorded on the coast at an elevation of 27.8 feet (8.5m), recorded 100 miles to the east of New Orleans at Pass Christian, Mississippi. The orientation and clockwise motion of the storm pushed storm surge into the city from multiple directions, via Lake Ponchartrian at the north and from the east via Lake Borgne.

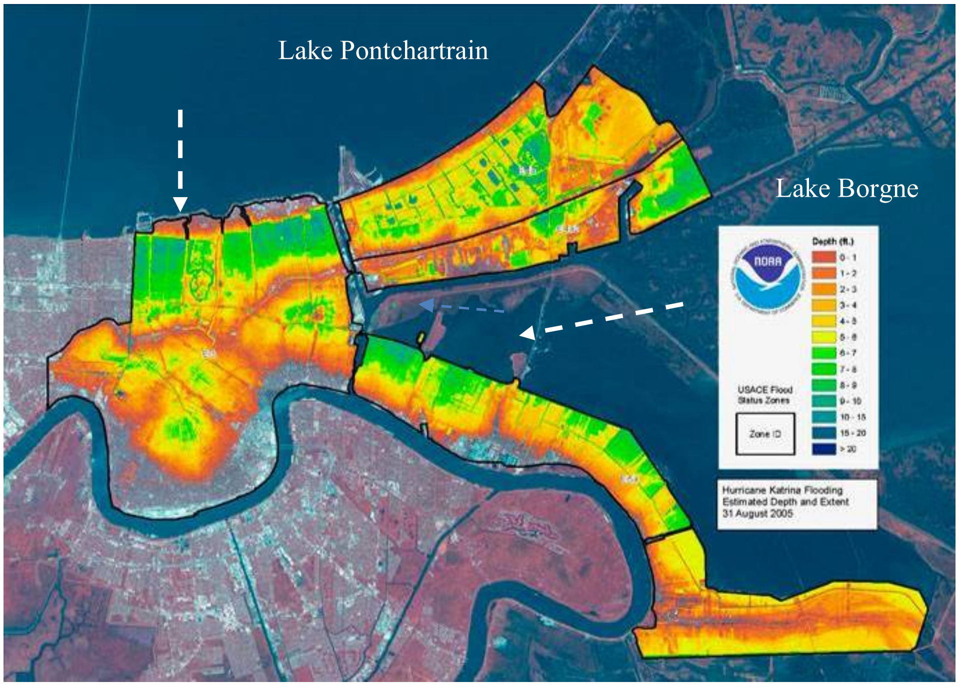


Figure 3-7 Enhanced satellite image of the flood extent due to Katrina (NOAA). Surge entered from the lake at the north and canal at the east of the city.

It is estimated 80% of the metropolitan areas were left submerged and it took 6 weeks before the city was completely dewatered (Boyd, 2011).

The opportunity for storm surge to reach the city is ever increasing. Sea level rise and continued ground subsidence contribute to this. Some data suggests that climate change effects have produced storms of increasing intensity in recent years, an additional source of increasing risk, although this has not been verified a consistent long term trend. Finally, the coastal wetlands are also eroding. This is due largely to the man made impoundment of the river, which inhibits sedimentation. The erosion of this land reduces the buffer between the city and the sea.

3.1.2 Flooding due to river discharge

The second hydraulic mechanism that threatens flooding to the city is the Mississippi River. The drainage conduit for over 40% of the continental United States, the river drains the third largest basin in the world at 3,224,550 square kilometres. It passes the city at an average flow of 18,000 m3 /s en route to the Gulf of Mexico. Due to its location on the river, New is hub for water borne commerce, transporting cargo between the agricultural and manufacturing canter of America and the rest of the world. If the 5 ports lying between New Orleans and Baton Rouge, the city roughly 66 river miles to the north are combined, the ports could be considered the third largest port in the world, handling 475 million tons of cargo per year.

Prior to manmade protection, the river's seasonal high flows would cause the river to overflow its banks and inundating the floodplain adjacent to the river channel. As the city of New Orleans grew, local levees were built to protect the residents. However in 1927, a year of heavy rain led to extreme water level which overwhelmed the protection north of the city. Levees which protected southern portions of the city were dynamited in fear of breaches at the more urban areas of the city. Portions of the poorer areas of the city were inundated with water.

Overall, the 1927 flood claimed roughly 250 lives and forced 700,000 residents from their homes. Direct economic losses along the lower Mississippi River were estimated by the Red Cross and the U.S. Weather Bureau to be between $250 and $350 million (RMS, 2007).

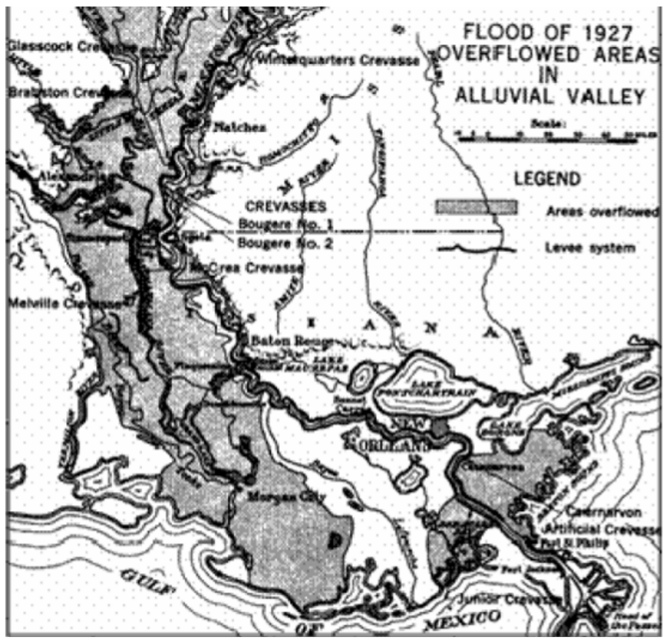


Figure 3-8 Map of land inundated during height of river flood in 1927 (RMS, 2007).

This event initiated the federal control of the river protection system, now the longest protection system in the world. Since its implementation, this system has been challenged several times, most notably in 1973 when high waters almost scoured away a diversion structure that was diverting flood waters. More recently, flooding in 2011 saw record stage levels along the southern portions of the river. The opening of bypass structures as well as the dynamiting of emergency levees was required to control river levels in the delta area of the river.

A recent study has suggested the risk of river flooding events such the event of 1927, show a statistically significant increase in occurrence over the past 30 years and that this is consistent with climate model outputs (implying a connection with climate change) (Milly et al, 2002). Thus the consideration of increasing risk regarding river flooding is also important to this study.

3.2 Study Area: New Orleans Metro bowl

For the scope of this study, the risk assessment is limited to a portion of the metropolitan areas of the city. From a flood protection perspective, the city can be divided into three 'rings' of protection. The rings are part of the 'systems based approach', which provides protection in complete 'rings' designed by uniform guidelines. The protection rings follow the outline seen here.

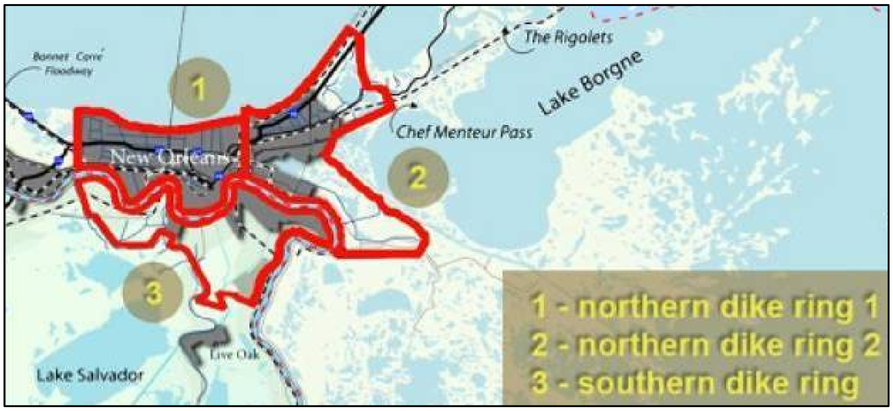


Figure 3-9 Rings of New Orleans flood protection (Jonkman et al. 2008).

For this study, only a portion of dike ring 1 will be considered, the portion which is within the border of the Orleans parish lines. This area is bordered by the lake at the north and the river to the south, an area of roughly 100km^2 (40 square miles). The western border of the considered area is the parish line.

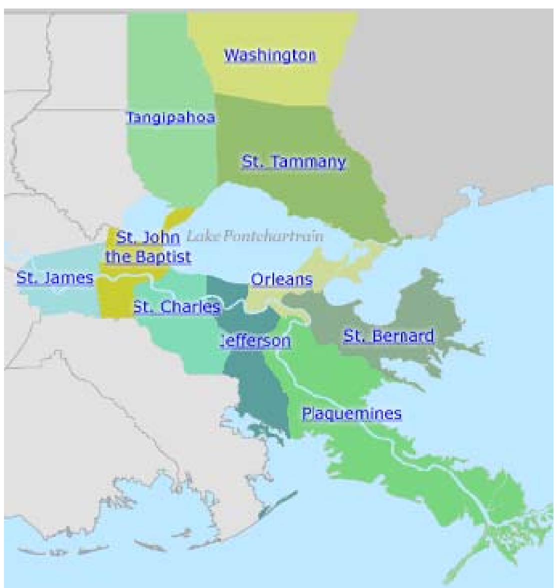


Figure 3-10 Area of study, the Orleans parish portion of the metro bowl is seen on the right. The parish in context is seen on the left.

This area of the city will be referred to as the '**metro bowl**'. Although the flooding effects during hurricane events have been seen to affect the other areas as well, the metro bowl is the most populous and historic region of the city. It is also the area of highest economic value.

Based on the system modelled here, the boundaries of this study include:

- The lake boundary at the north, which is subject to storm surge.
- A canal boundary at the east, which is subject to storm surge. This canal is referred to as the IHNC, the Inner Harbor Navigation Canal
- The river boundary at the south, subject to loads due to both storm surge and high discharge.

It is assumed that the probability of flooding of the polder from the west is very small and therefore it is not taken into consideration. This assumption can be investigated further in future work.

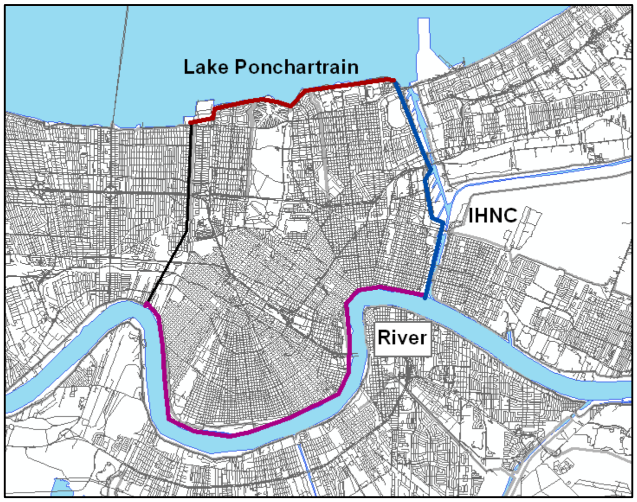


Figure 3-11 System boundaries for the New Orleans metro bowl.

A brief discussion on each boundary is provided.

3.2.1 Lakefront Boundary

The lakefront boundary protects from Lake Pontchartrain at the north of the city. The lake is connected to the sea to the east and is therefore subject to the effects of storm surge. A system of protective floodwalls and levees began in the 1930's along the lake to protect from periodic flood events (Campanella, 2006).

An important component of the lake boundary is the system of drainage canals used to pump water out of the city to the lake during a rain event. Prior to Katrina , the pump stations were located solely at the city centre where in order to drain the city , the water needed to be pumped to an elevated level at the station, creating a situation of increased water levels adjacent to low lying residential areas. This situation required flood protection to line the canals to the lakefront where the canal opened freely to the lake. This situation also allowed lake levels to affect canal water levels.

The surge during Katrina was therefore permitted to flow from the lake into the canals. The surge overloaded the floodwalls intended to provide protection along the canals. Overtopping of the protection and several wall failures occurred (IPET, 2007).

The upgraded protection system has closed off the canals where they meet the lake by constructing new pumping stations at the lakefront. The result is effectively a 'shortened' lake coastline. This is an important factor in limiting the flood risk as it significantly reduces the probability of these elements in the system failing.

3.2.2 IHNC Boundary

To the east of the metro bowl is a man made shipping canal, the Inner Harbor Navigation Canal (IHNC). The canal connects to the Mississippi river where a ship lock allows navigation to the river, and connects to the lake at the north. At the east lies the confluence of two navigable waterways, the Mississippi River Gulf Outlet (MRGO) and the Gulf Intercoastal Water Way (GIWW).

Prior to Katrina and the installation of upgraded protection measures, the IHNC boundary was highly vulnerable to the effects of storm surge. A funnel effect is created as the protection adjacent to the waterways meet and funnel the surge into the canal .

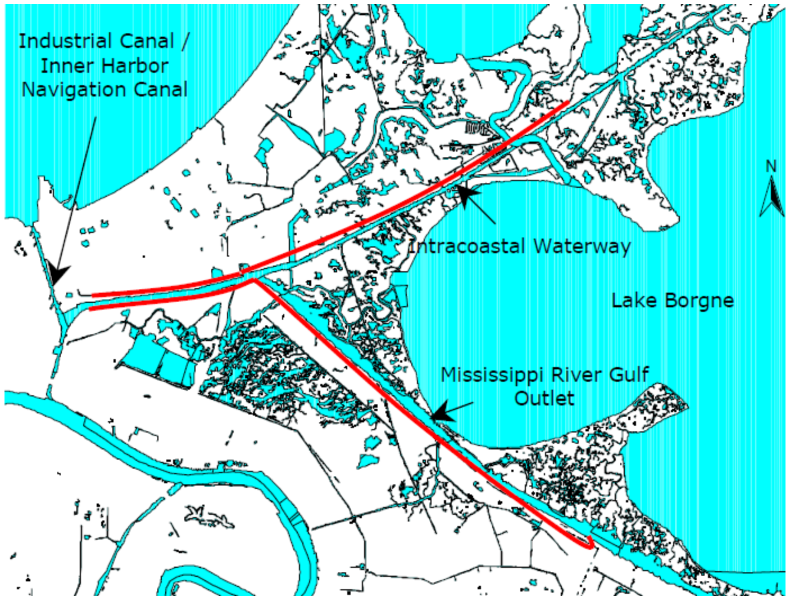


Figure 3-12 Storm surge from Lake Borgne is funnelled into the IHNC. The red lines are drawn along the man made canals (Maaskant, 2007).

1965 saw the winds of Hurricane Betsy funnel storm surge from the sea into the IHNC where breaches and severe overtopping occurred at both sides of the canal protection.

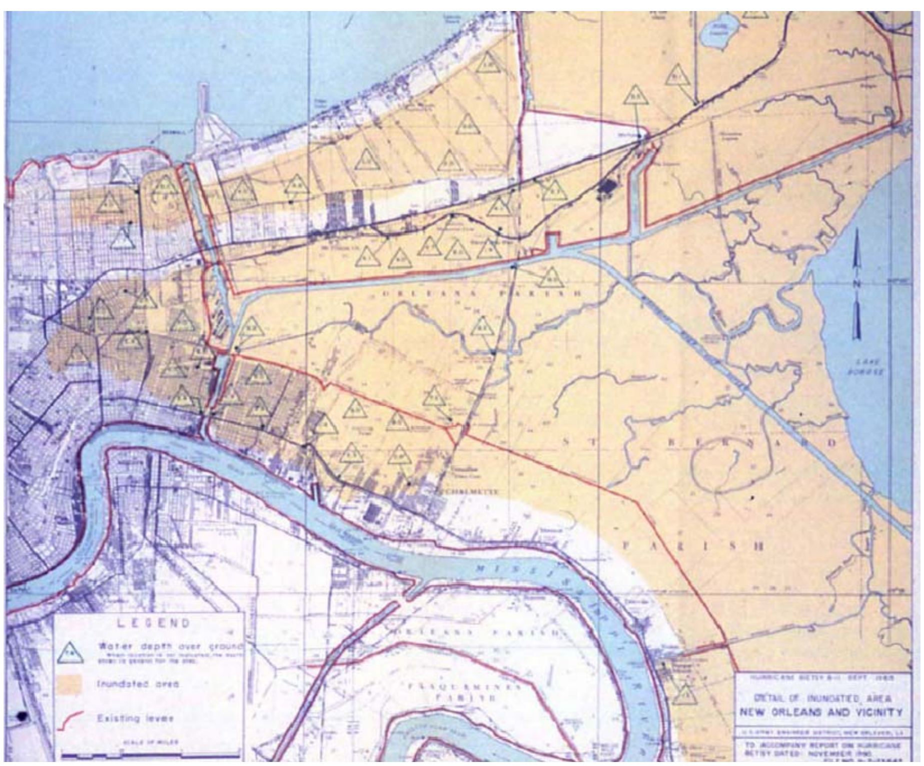


Figure 3-13 Map of flooding caused by Hurricane Betsy (1965) showing areas affected by overtopping (due to surge from Pontchartrain and Lake Borgne) (USACE,1965).

While the U.S. Army Corps of Engineers' Hurricane Protection Program began as a result of Hurricane Betsy, the IHNC boundary remained a source of vulnerability. This was confirmed by the events of Katrina which saw surge levels in the canal reached 14ft (4.25m) Once more, protection along the canal was overtopped or failed (some of the same sections of the canal floodwall failed during hurricane Betsy as did during Katrina.)

Since Katrina, upgraded protection system along the canal includes two new barriers that significantly reduce the opportunity for surge to enter the IHNC. A barrier flood wall referred to as the Lake Borgne barrier has been constructed at the confluence of MRGO and GIWW. This barrier ensures the IHNC is closed off from incoming hurricane storm surge and limits water levels allowed into the canal. The barrier include two gates that close during a storm event.

The canal was also opened to the lake at the north end, and thus water levels in the canal were influenced by the lake levels during a storm event. The future installation of barrier at the north end of the canal, the Seabrook barrier, will ensure that lake surge is unable to enter the canal via the lake.

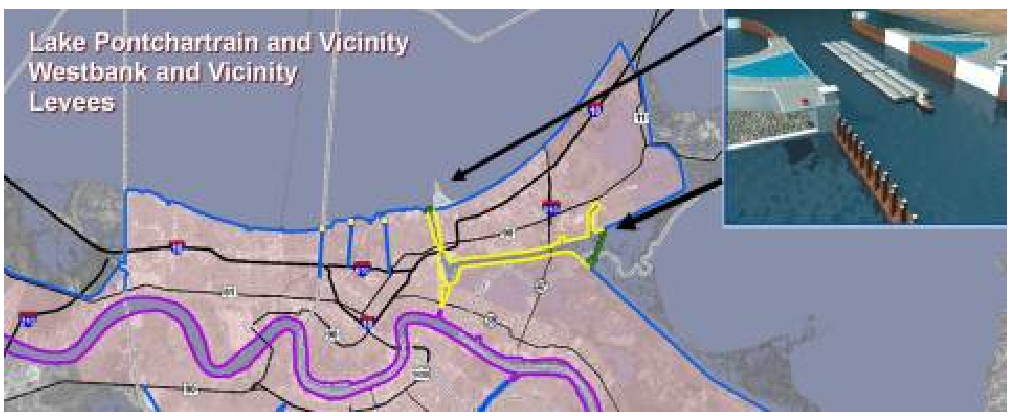


Figure 3-14 The yellow indicates area protected along the IHNC canal by the new barriers (Link, 2008).

3.2.3 River boundary

The final boundary of the ring of protection considered is the river boundary. The threat of both high discharge and storm surge are considered at this boundary as the water levels here are a function of both. This ‘transitory zone’ is affected by tides and storm effects downstream level while river discharge experiences seasonal high and low flows at the upstream level.

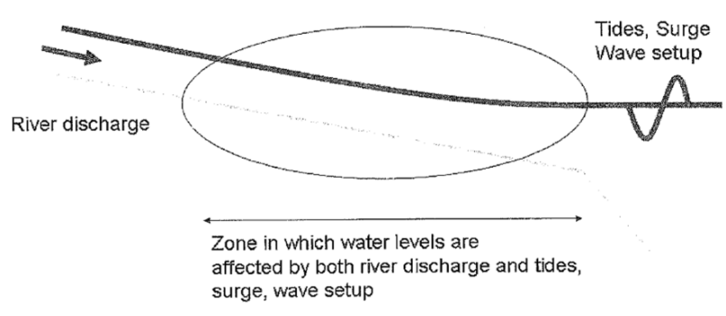


Figure 3-15 Schematic of the Mississippi River at New Orleans with both river discharge and sea as dominant forces.

While there has been no history of flooding to the city due to surge at the river boundary, water levels at the city are becoming increasingly influenced the sea due to continued coastal land loss and ground subsidence and sea level rise . The transitory zone will continue to move upriver with increased marine influence.

An examination of water levels on the river during Hurricane Katrina demonstrates the influence of storm surge on the water level. A surge increase of over 12 feet (3.24 m) was seen in the river at the stage gage located at the city during hurricane Katrina.

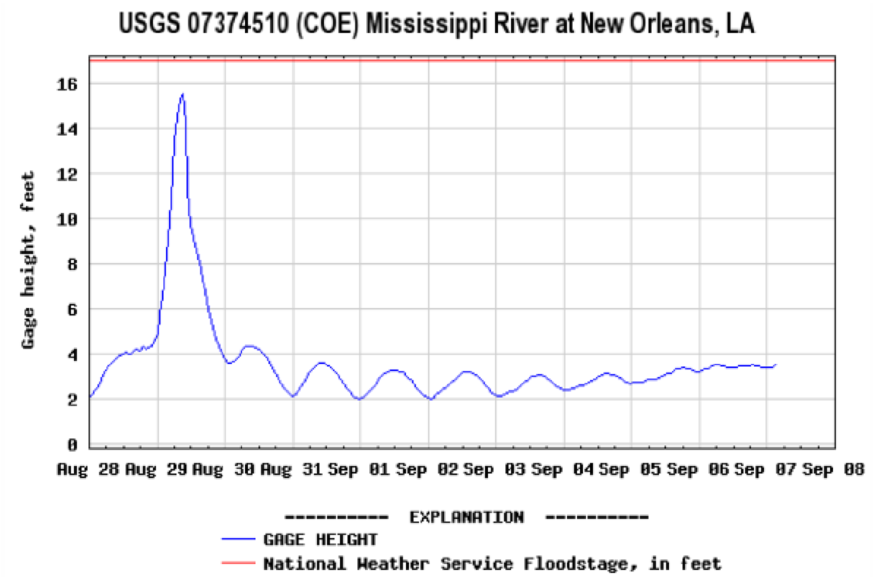


Figure 3-16 Mississippi River gage reading at New Orleans during Hurricane Katrina.

The river protection at the city is therefore designed to protect from both hydraulic mechanisms. This protection at the city of New Orleans currently consists of levee and floodwalls.

As a final part of the qualitative risk description of the modelled system, the various ways that the system protection can fail is discussed.

3.3 Failure modes of the New Orleans protection system

The protection system is subject to loads from river and storm events, and the flood protection system for New Orleans is designed to resist both river and hurricane flooding. The protection is comprised of various types of protection such as earthen levees, flood or sea walls and gates. Typical failure modes can be attributed to various protection types and due to the different hydraulic mechanism.

Failure of the flood protection can be caused by the flood water level exceeding the design level, or overtopping, but failure can also occur prior to this event due to various geotechnical, structural and other failure types.

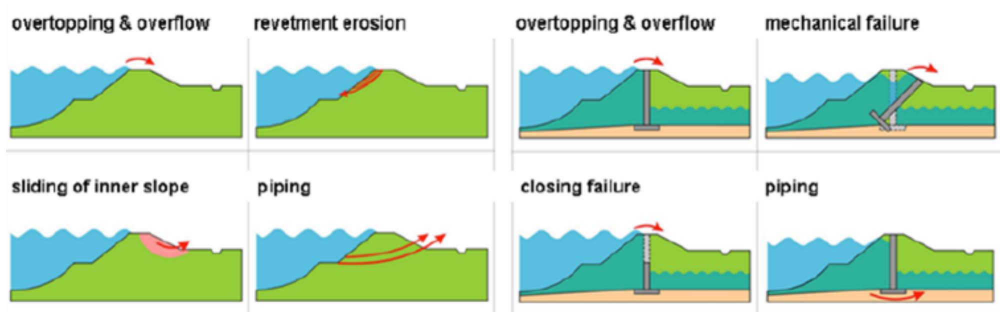


Figure 3-17 Failure mechanisms of earthen and structural protection measures (Riedstra, 2010).

Hurricane vs. river protection
The protection is designed to resist the various hydraulic mechanisms loading the system. Hurricane protection is designed to prevent flooding due to 'high energy tidal events of relatively short duration, and is built in low lying unstable areas. River protection meanwhile, is built along the stable natural ridge of the river and prevents flooding from slow rising river events (Boyd, 2010). Thus typical failure mechanisms can also be identified for the hydraulic loads acting on the system.

Common failure mechanisms for **hurricane** protection mechanisms specific to walls or structures include failure by overturning or sliding. Geotechnical failures such as piping, where a pressure differential in the flood defence causes pressurized water to flow under the structural protection.

During Katrina, failure of the hurricane protection was of various types. Overtopping of the levees was seen to cause scouring and erosion of protection from the inner side. Piping caused water to flow under flood protection walls and earthen protection, causing instability and collapse. Failure of walls due boat impact was also experienced.

In the context of **river** protection, a more common failure mechanism is the sloughing of the outer slope (river side) during periods of rapid drawdown, or drop in water level. The saturated earthen protection fails by sliding or sloughing off. Such was the case in the 1973 flood which saw water levels drop 7ft (2. 2 m) in 9 days, causing sloughing of the river levee (Rodgers, 2011).

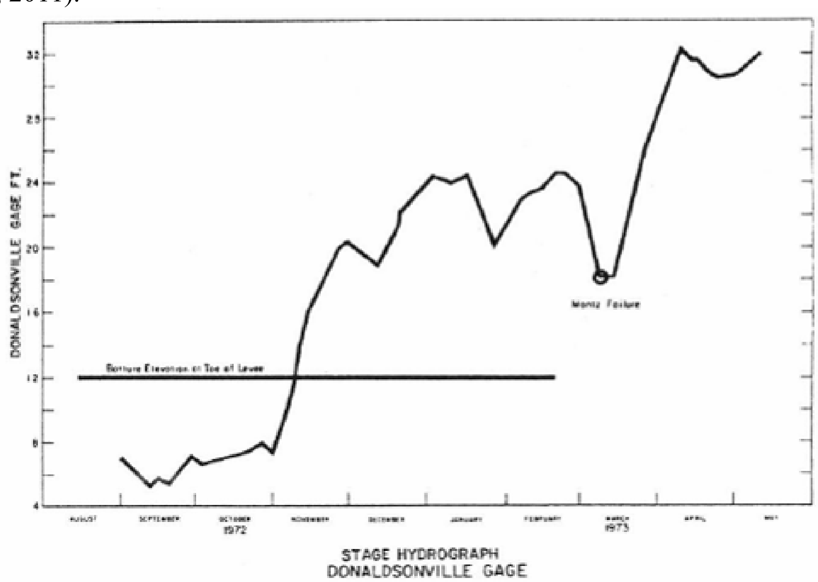


Figure 3-18 A' rapid drawdown' of the river stage caused failure of river protection during the 1973 river flood (Rodgers, 2011).

Thus the city's location and environmental surroundings make it uniquely vulnerable to flooding. The qualitative risk description provides an overview of the city's protection system and the hydraulic systems that threaten it. Assumptions made for the modelled system as well as the mechanisms by which the upgraded protection system can fail are described as well. The next chapter will provide a step by step analysis of the quantitative risk to the city.

4 Quantitative analysis

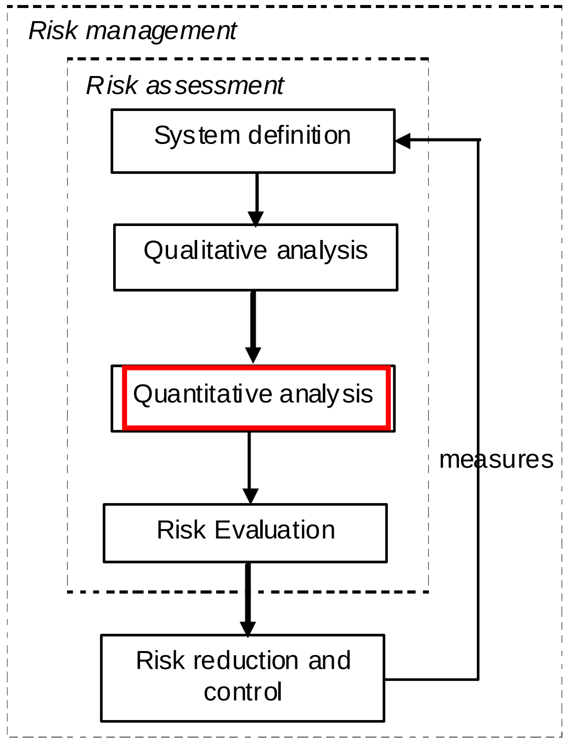


Figure 4-1 Steps of a Risk Assessment (Jonkman, 2007).

To determine a quantitative risk estimate, the probabilities of failure of the flood protection and consequences due to failure must be estimated. To accomplish this, a scenario based approach is used similar to that is being applied in the FLORIS project, (Flood Risk In the Netherlands, see Riedstra, 2010). In following this approach, the following 'sub-steps' to the risk assessment are carried out:

1) Estimate reliability of the system
2) Simulate defined flood scenarios
3) Determine consequences
4) Quantify risk

These steps are described in detail.

4.1 Step 1: Estimate reliability of the system

As mentioned a complete reliability analysis of the system would consider all possible load conditions and strength (or resistance) parameters of each element in the protection system. When quantifying the strength variable, the analysis can consider failure of the defence due to multiple failure mechanisms. The reliability analysis for determining failure probabilities as conducted by IPET considers multiple failure mechanisms of the New Orleans protection system. Such work could be used to elaborate on the reliability of the system in future work.

For simplification in this thesis, the probability of failure is based on the **design guidelines** of the upgraded protection system, in combination with expert judgment. The resulting probability values are expressed as probabilities per unit time, (i.e., per year).

4.1.1 System elements

To estimate the system reliability, the defined flood defence system is divided into system elements or reaches. The entire system is considered to be a linear defence where the failure of any reach leads to failure of the system, in this case, the inundation of the protected area. Therefore the probability of failure of the system is a function of probabilities of failure of each of the sub system elements. From a reliability standpoint, the system is a serial system where the probability is dominated by the weakest link. The contribution of each reach or element to the overall probability of flooding is seen.

Thus reaches are divided into protection sections of similar strength characteristics subject to similar hydraulic loads. Enough variation in characteristics of the elements allows failure of the reaches to be assumed **independent**. This assumption allows the overall probability of system failure is approximated as the **sum** of the failure probabilities of the reaches (Vrijling, 2001). Correlations or dependencies in failure probabilities between different sections and considerations for the length of the reach would be taken into account in a more detailed analysis.

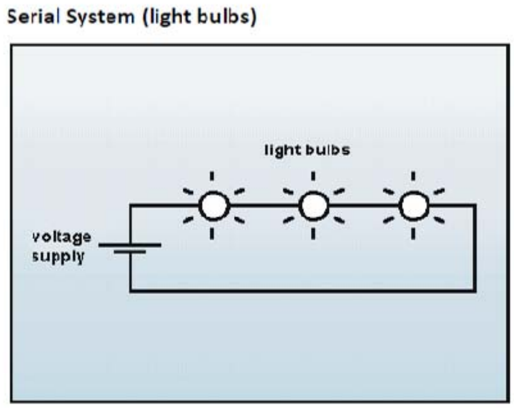


Figure 4-2 Principle of serial systems. The failure of any element causes the failure of the system (CT4130 lecture notes).

Based on the system as defined in chapter 2, the boundaries of this study included:

- The lake boundary at the north, which is subject to storm surge.
- A canal boundary at the east, which is subject to storm surge. This canal is referred to as the IHNC, the Inner Harbor Navigation Canal
- The river boundary at the south, subject to loads due to both storm surge and high discharge.

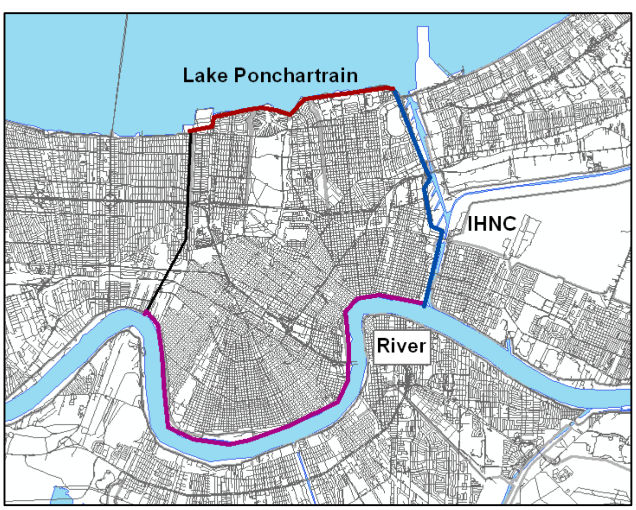


Figure 4-3 System boundaries for the New Orleans metro bowl.

Considering the discussed system boundaries and assumptions of the modelled system, a fault tree can depict the various events that could lead to inundation of the system. The occurrence of any of these events would lead to the undesired event of system failure. The relation between the element failures and the unwanted consequence can be depicted in a fault tree:

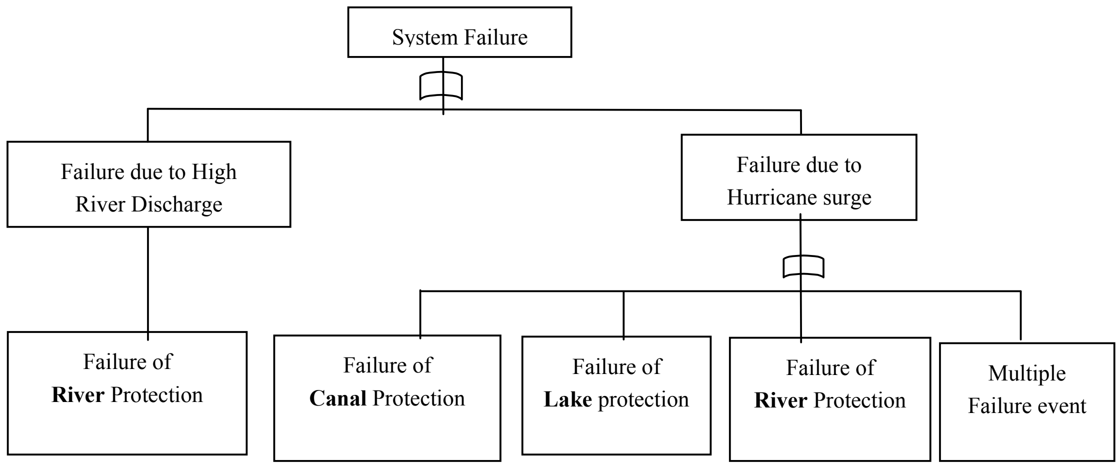


Figure 4-4 A fault tree depicts the various system elements whose failure would lead to the failure of the system.

It can be seen that a multiple failure event has been included in the fault tree. In observed historical flood events, the flooding had been characterized by multiple breaches. This was also the case during Katrina.

With the system reaches defined, a failure probability can be assumed for each reach. These failure probabilities are based upon the system design guidelines

4.1.2 Hurricane protection guidelines

The probability of failure is based on the existing design criteria Hurricane Protection Design Guidelines. Following Katrina, design guidelines for the upgraded hurricane protection system were updated using existing engineering guidelines (USACE manuals), best practices as experienced by the USACE, and guidance taken from the Interagency Performance Evaluation Team (IPET) report.

The revaluated criteria for geotechnical and structural design is deterministic in approach, applying safety factors to achieve required protection reliability. For example, in the consideration of a geotechnical failure due to sliding or instability of an earthen flood defence, a stability analysis is carried out to consider a large number of potential slip surfaces under multiple hydraulic conditions. The required factor of safety applied to each situation depends on the hydraulic condition being considered and the method of analysis, and results in the acceptable level of reliability in the earthen defence.

Analysis Condition	Required Minimum Factor of Safety	
	Spencer Method[1]	MOP[2]
End of Construction[3]	N/A	N/A
Design Hurricane[4] (SWL)	1.5	1.3
Water at Project Grade (levees)[5]	1.4 (1.5)[6]	1.2
Water at Construction Grade (levees)[5]	1.2	N/A
Extreme Hurricane (water @ top of I-Walls)[5]	1.4 (1.5)[6]	1.3
Extreme Hurricane (water @ top of T-Walls)[5a]	1.4 (1.5)[6]	1.2
Low Water (hurricane condition)[7]	1.4	1.3
Low Water(non-hurricane condition)[8] S-case	1.4	1.3
Water at Project Grade Utility Crossing[9]	1.5 (1.4)	1.3 (1.2)

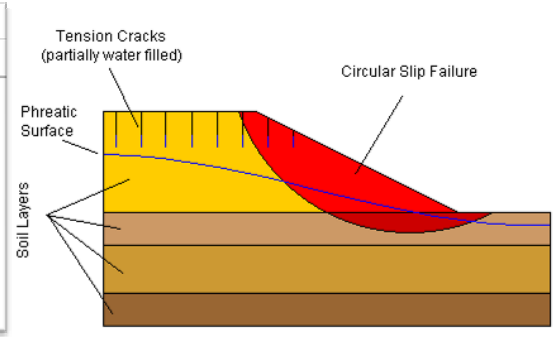


Figure 4-5 Load factors are utilized in considering structural and geotechnical failure mechanisms. Above is an example of safety factors for slope stability design (USACE HSDRRS guidelines, 2008).

In contrast to the deterministic methods utilized in the geotechnical and structural design, the approach used in the hydraulic design guidelines is largely probabilistic. While the deterministic method aims at preventing the worst case scenario, and does not consider the likelihood of the circumstances, the probabilistic approach incorporates the uncertainty in the input parameters. Based on the difference in design approaches, it is assumed the probabilistic approach followed for the hydraulic design guidelines allows for a more **liberal** design and that failure due to geotechnical and structural failures is smaller than that of hydraulic failures (the design water level being reached). This being the case, failure probabilities of the system are assumed based on **the hydraulic design criterion**, in combination with expert judgment. The hydraulic design criteria will therefore be discussed in more detail. (Note that other failure mechanisms not considered in the protection guidelines, such as organizational failures, human error and others can also be considered.)

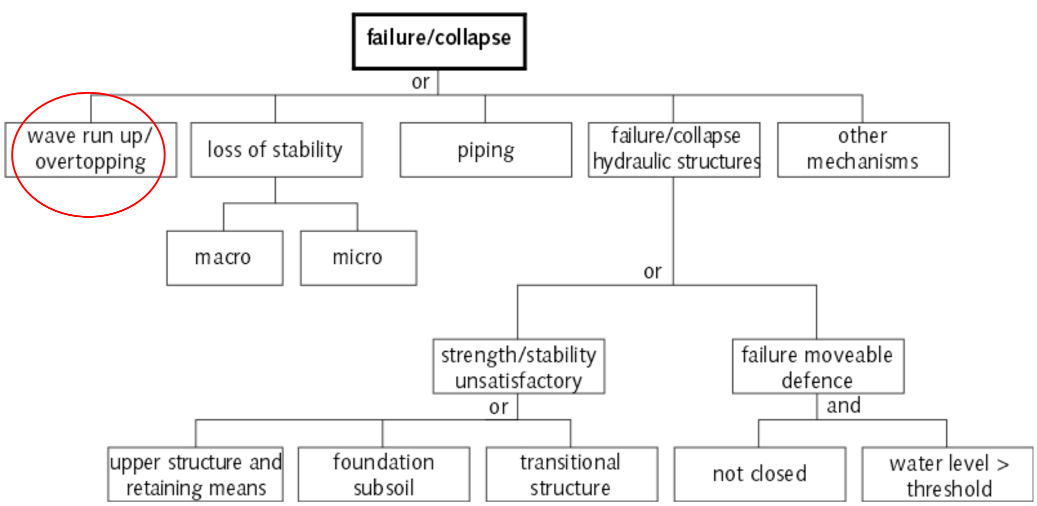


Figure 4-6 Fault tree showing the various mechanisms that lead to the failure of the defence (Lecture notes, CT 4390). The hydraulic guidelines which consider the failure mode of overtopping are used as a basis for the failure probability as they lead to a more liberal design.

Hydraulic design criterion

Data sources

The hydraulic design criterion results in the protection elevation of the upgraded HSSDRS system which provides protection that can withstand a 1/100 year event. To determine these elevations, parameters of a 1% storm event are required. Following Katrina, a hydraulic modelling effort, the Joint coastal surge study , utilized a statistical approach to estimate return periods of storm events. The approach improved upon the existing approach to estimating potential surge events, where the formulation of a “design storm event” approach where the chosen storm had specific characteristics of an event of a long return period. The past approach relied on unrepresentative historical data which limited the ability to describe the storms.

The updated process to produce frequency curves of various storm parameters included the use of a Joint probability model that took into account the conditional probability density functions of various storm parameters. The probable combinations, (joint probability) of the parameters were used to model a set of **synthetic storms** each with an associated probability. The resulting frequency curves are available for a range of return periods (1/10 – 1/500 year). The results of this modelling effort have been used in multiple applications including the IPET study to be discussed.

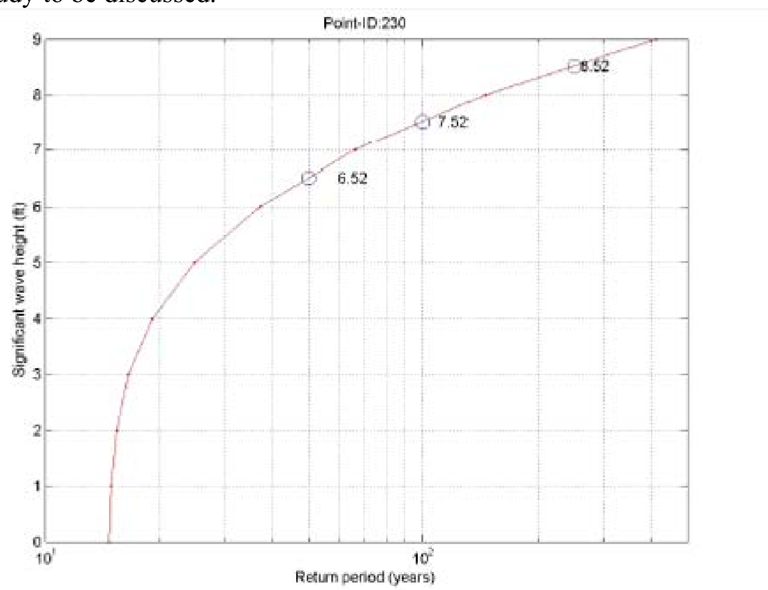


Figure 4-7 Frequency curve of the wave height.

Design elevations

The frequency curves provide the 1% storm characteristics required to design the upgraded protection system. These characteristics, combined with the geometry of the protection are used to determine an overtopping rate for each reach of protection. The Van der Meer equation for overtopping, (see e.g. TAW, 2002) results in an overtopping rate for the 1% event. The approach is probabilistic in that a statistical distribution of the overtopping rate, (through the use of a Monte Carlo simulation) is determined for each reach. The 50% and 90% confidence limit values of the overtopping rate are compared with defined thresholds for levees and floodwalls. An iterated design height can then be specified per reach to meet the overtopping criteria.

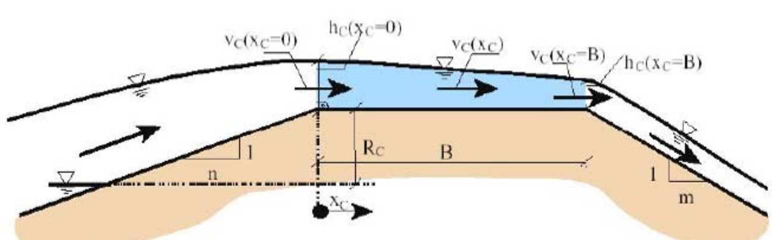


Figure 4-8 Wave overtopping definition sketch (Schuttrumpf and Oumeraci, 2005, see USACE 2010).

Guidelines for establishing the overtopping rate threshold, the point at which damage would start to occur, is based on values found in the US Army Corps Engineering Design Manual. The criterion is consistent with values used in both Dutch and Japanese design approaches.

ft^3/s (cfs)	m^3/s	Confidence interval
.01	.001	50%
0.1	.01	90%

Table 4-9 Summary of overtopping criterion to which the protection system is designed.

Resiliency or the ability of the defence to withstand the design load, in included in the design by ensuring the design elevation of the protection is higher than the 1/500 year flood event surge levels. Also the 1/500 year overtopping rate is also compared against the overtopping criteria, providing insight for how the levee might perform during a 1/500 year event. Such measures of resiliency help to ensure the levee will withstand the design load.

Hydraulic design guidelines for river boundary

As the river boundary is influenced by both the sea and the river discharge, the probability of high water levels at this point is also a function of both the storm surge and the river discharges. Recent design considerations for the hurricane protection have included surge as a variable for hurricane water level statistics at this boundary.

To include considerations for the river stage in the design of protection, the joint probability method that computes the surge level probability was amended to include probabilities of river discharges. In this estimation the **river discharge** and **surge** are assumed independent events.

For this analysis, three probability density functions are utilized. The updated probability for a surge levels up the river given a certain discharge is considered to be:

$$P(\eta /Q)= \int p(\eta \mid Q)p(Q)dQ \qquad \text{(eq 3 .1)}$$

Where η is the surge level and Q is the river discharge. The probability of the river discharge associated with hurricane surges is considered to be:

$$p(Q)=\sum p_m(Q)p(m) \qquad \text{(eq 3.2)}$$

Where p(m) is the percentage of hurricanes that will occur in month m.

-The density function $p(\eta|Q)$ was derived from storm simulation runs using various combinations of discharge and storm characteristics. The storm modelling suite from which the parameter frequency curves were derived was extended to include storms that describe hurricane surge effects up the river.

-For the hurricane probability density function, p(m), historical data of hurricanes in the Gulf of Mexico from 1941 – 2005 are examined and the probability density of the storms is the number of storms per month.

-Finally, the discharge probability function pm(Q) used flow data of the lower Mississippi. for hurricane months during a selected period of 1976 – 2002.

With these three probability density functions, a matrix of interpolated expected surge levels is created (USACE, 2010). The matrix includes three fixed discharges, (90% confidence intervals, as well as a median flow) and the surge levels for fixed return intervals.

Having determined the surge levels along the river for events of selected return periods, the elevation of protection is calculated. This is accomplished using the same process as was used for the remainder of the hurricane protection system. The resulting design elevation govern design elevations (and therefore protection design) for roughly the lower 87 miles (140 kilometres) of river protection. River mile definitions start at a designated location, the 'Head of Passes', a designation point near the mouth of the river.

As the city of New Orleans lies between river miles 93 and 104 (150-167km), the protection design elevations are governed by high river discharge design criteria, to be discussed in the next section.

4.1.3 River discharge Protection Design Guidelines

Finally, the contribution to the probability of failure of the river levees due to the second hydraulic mechanism, river discharge, is based upon design criteria of the river protection.

To determine the maintained elevation of the river levees the project flood 'flowline', has been designed for. This flowline describes the stage at which the river discharge is that of the 'project flood', representing a worst case scenario river event.

The project flood is based off a project design storm series analysis. The project design storm series consists of various combinations of actual storms and the resultant floods that had a reasonable combination of occurring. The project flood was the result of the storms that had the greatest probability of producing the greatest discharge. The flowline was last updated in 1973 using discharge capacities observed during the 1973 flood. The final river levee elevation is equivalent to the project flood plus a specified freeboard that varies based on location.

As the flowline is based on a discharge stage analysis, the event frequency of the protection level varies by location along the levee system. Thus no specific frequency has been assigned to this project flood, but in general it is estimated in the range of 100 to 500 year (Louque, 1976.)

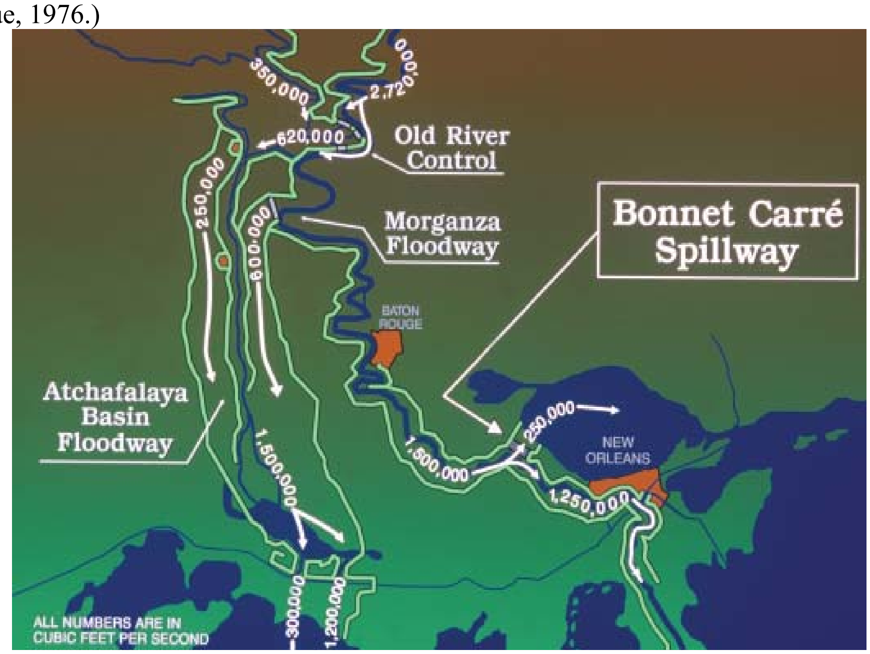


Figure 4-10 Schematic of the Bonnet Carre Spillway and other control structures. (http://www.mvn.usace.army.mil/pao/bro/Bonnet_Carre_041220.pdf)

The flowline at the city of New Orleans is affected by the upstream bypass structures. The Morganza floodway and Bonnet Carre spillway are located upstream of the city and protect New Orleans from flooding by the Mississippi River by reducing the volume of flow that passes the city. Both spillways must reach capacity for the stage at New Orleans to exceed a 1/100 year level. This event corresponds to a discharge of 2,100,000 cfs (60,000 m/s^2) at the Morganza spillway structure. Based on preliminary analysis in combination with expert opinion, the frequency of this discharge is roughly **a 1/880 year event** (USACE, 2010). The probabilities of failure of the spillways are neglected here due to the assumption of a long warning time. It is noted that the protection along the river is largely earthen and only partially manmade as underlying layers of the river protection are the result of natural process of the river flooding. These natural levees are also very wide, such that the river levees are robust and geotechnical instability is unlikely (Mashiriqui, 2005).

4.1.4 Probability assignments for system elements

Therefore based on the hydraulic portion of the hurricane protection design guidelines, the elements which have been segmented based on similar load and strength characteristics, are assigned failure probabilities for the two flood mechanisms. Of course these probabilities are assumptions and need to be verified by means of actual reliability calculations.

As the hurricane design guidelines provide protection of a 1/100 year level, an overall failure probability due to a hurricane event of 1/100.

IHNC boundary assumption
Chapter two provided an overview of the risk reduction of the IHNC due to the installation of two new barriers, the Seabrook barrier and the Lake Borgne Barrier. The design heights of the current protection along the canal assume the closure of both gates. For consideration of failure of the protection, a failure of the gate closure is assumed. The Maeslant barrier, a closure structure part of the Dutch coastline is considered as a reference (Duits & Thonus, 2007). Studies have determined the estimate of the failure probability of the Maeslant barrier to be 1/100. Considering the results of such a study, and that the level of redundancy in design is at a greater level than that of New Orleans, an assumption of a 1/50 failure probability could be assigned to the gate closure.

Figure 4-11 Maeslant Barrier (left) of the Netherlands is estimated to have a failure probability of 1/100. The Seabrook barrier (right) is assumed to have a comparable failure probability.

As the protection levels in the canal assume the closure of both gates, the failure of a gate to close would allow water to rise above design levels of the current flood protection system. As such, the current protection is assumed to fail at a 1/100 year event given the failure of a gate. The gate failure probability combined with a probability of an extreme storm event lead to an assumed failure probability in the canal of **1/500**.

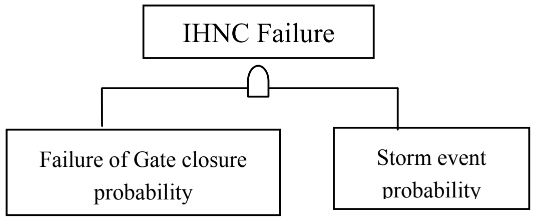


Figure 4-12 Fault tree for failure of IHNC protection

River protection breach assumption
Thus the existing river protection elevation at the city is roughly 1m (3 ft) above the 1/100 hurricane protection design elevations. Thus, the breach probability due to a storm surge is still assumed to be low. The results of the storm surge frequency analysis show that water levels for the 1/1000 year return period are comparable to the river protection design height. Thus, it is concluded that a **1/1000** year failure probability is a best estimate for breaching due to storm surge.

Multiple breach assumption
Finally, a multi breach situation is incorporated into the overall failure probability discussion. Dependency of failures is assumed in multi breach probability assumption. The multiple breach scenario is assumed to have a failure probability of 1/5000. As the scenarios are assumed to be independent, the multiple breach is assumed to have a smaller probability by an order of magnitude.

Lake boundary assumption
As the entire hurricane failure probability should equate to 1/100, the lake boundary condition is assigned the remaining portion of the failure probability to result in this assumption.

Additional considerations for this assumption might include analysis of overtopping rates determined for protection along the lake front can be examined to provide a probability estimate for breaching of the lake protection.

High discharge breach assumption
Finally, the failure probability for the high discharge will be added to the probability of failure of the hurricane system (1/100) to determine the overall failure probability. Based on the high discharge flowline data discussed, the failure probability due to river flooding is considered to be **1/1000**. This is thought to be a conservative assumption considering the uncertainty in the design guidelines.

Overall failure probability
Based on these assumptions, the overall system failure probability can be estimated to be the sum of the elements in the system:

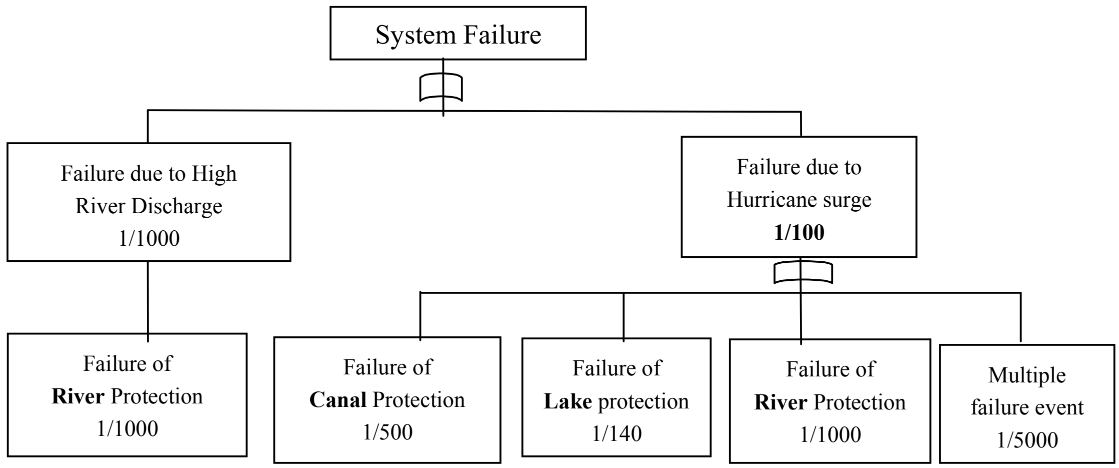


Figure 4-13 Fault tree represents failure modes that lead to system failure;

Boundary / Element	Failure Probability
Lake	1/140
IHNC	1/500
River, Surge	1/1000
River, High discharge	1/1000
Multi breach	1/5000

The assumed probabilities enable the simulated flood scenarios to be assigned an event probability. The next step is to define these simulations.

4.2 Step 3: Simulate event scenarios

A set of selected flood scenarios are defined and simulated to represent the various ways flooding occurs. These few scenarios serve as a simplification of a full probabilistic analysis where all possible parts of the protection system would be considered with every possible flood scenario.

Definition of scenarios

The defined scenarios simulate the failure of the various defined system elements due to the multiple hydraulic systems, resulting in a deterministic result of the progression of flooding. These scenarios are based upon several assumptions and best estimations. Scenarios with the largest contribution to the overall flooding probability are selected, allowing the most relevant contributions to the overall risk estimate are included (Jonkman, et al, 2008).

Catastrophic breaches of the updated protection system are simulated. When developing the scenarios, the following considerations are followed:

- Scenarios should result in a variation of consequences
- Scenarios should model the various hydraulic loads that can affect the system
- Scenarios should model the behaviour of the various protection elements of the system.

In theory, a breach is simulated for each reach of protection that would result in **similar consequences** given a failure. If this is the case, the exact location of a breach within the reach does not significantly affect the consequences (Aalberts, 2008).

Two breaches have been simulated along the lake, at the far 'west end' and in the middle of the lake boundary referred to as 'St. John'. A scenario is modelled on the IHNC at the east of the metro polder as well. At the river boundary, scenarios initiated both by hurricane surge and high river discharge are simulated. Lastly, a 'multiple breach' simulates a simultaneous breach event, similar to what occurred during Katrina. This breach consists of the simultaneous occurrence of the simulated lake and IHNC breaches.

Figure 4-14 Breach locations simulated (Graphic: Maaskant 2007)

The chosen breach points can be compared with the location of breaches caused by Hurricane Katrina. Breaches during Katrina occurred in the canals stretching into the city , but are no longer considered due to improvements made to the system.

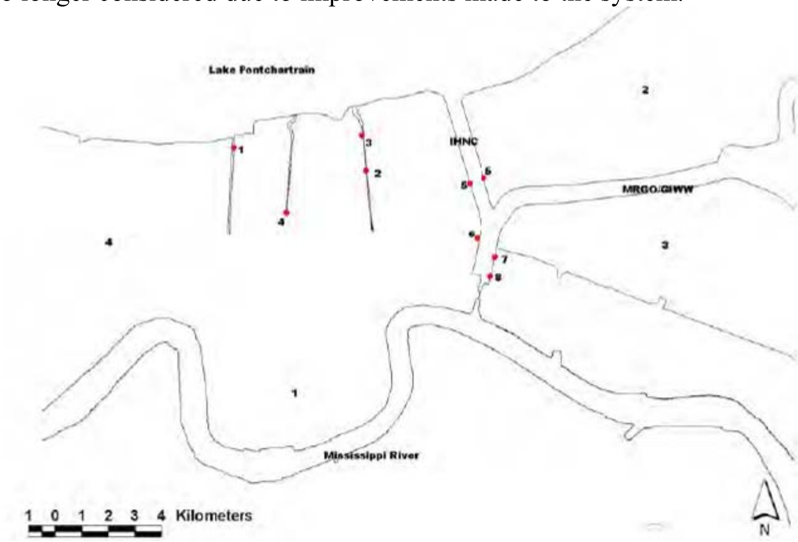


Figure 4-15 Locations of the most important breaches and overflow events (de Bruijn, 2006)

4.2.1 SOBEK flood model

To model the scenarios, the SOBEK 1D-2D overland flow model, developed by the hydraulic research centre Deltares, is used. SOBEK utilizes the ‘Delft scheme’ of solving the St. Venant fluid flow equations to compute flow. The program is both 1D and 2D as a 1D flow boundary condition is connected to a 2D overland grid (de Bruijn, 2006). The model computes the water levels and current velocities within the flooded area as a function of time so the progression of flood waters and depth is simulated.

The model of the New Orleans metro area was previously developed in SOBEK by K.M de Bruijn in 2006. The digital elevation model (DEM) of the New Orleans metro bowl is taken from the National Elevation Dataset of the United States Geological Survey (USGS, 2006). The horizontal and vertical datum of NAD83 and NAVD88 have been adjusted to UTM15, which lead to a cell size of 28.74m. GIS Data from various sources were used to define the elevation data of the protection in the model.

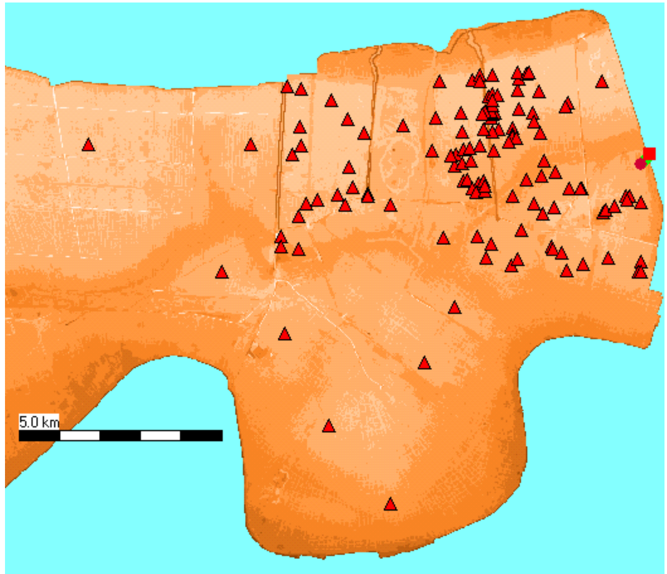


Figure 4-16 Sobek model for the Orleans Metro Bowl

4.2.2 Model Input

In this simplified assessment, deterministic scenario parameters are estimated as input. These parameters are discussed in the following categories:

1) Roughness
2) Hydraulic boundaries
3) Breach characteristics.

1) **Roughness**

Within the model, the roughness can be specified as Manning, Chezy, or White-colebrook (Nikuradse value). Here roughness of the terrain is taken to have a uniform Nikuradse value of .3m. Variation in roughness due to buildings or internal waterways has been neglected. A sensitivity analysis by de Bruijn found that variation in roughness slightly affects the temporal development of the progression of flooding, but does not significantly affect the maximum water depths.

2) **Hydraulic Boundaries**

SOBEK allows for boundary conditions, including water level or discharge, to be input as a function of time. Water level hydrographs are developed for each of the simulated scenarios. Breaching (failure) is assumed to occur under design water levels specified in the discussed guidelines. The input hydrograph for each simulation is characterized by a peak surge value of a return period which most closely represents the estimated failure probability of the event. The frequency curved developed by the joint probability model (discussed in section 4.1.2) provide the return values for the required return periods. Event characteristics are provided up to a 1/500 return interval for the lake and IHNC boundaries, and up to the 1/2000 year return period for the river boundary.

Orleans Parish Metro Lakefront Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
NO06	NO Marina	Structure/Wall	Future	16.0	12.8	0.244
NO10	Topaz St. Levee	Levee	Existing	15.0	11.3	0.314
NO10	Topaz St. Levee	Levee	Future	17.5	12.8	0.323
NO15	Type II Floodgate similar to Canal Blvd	Structure/Wall	Future	16.0	12.8	0.074
NO13	17th St. Outfall Canal Closure	Structure/Wall	Future	16.0	12.8	0.211
NO12	Orleans Ave Outfall Canal Closure	Structure/Wall	Future	16.0	13.1	0.076
NO14	Type I Floodgate Similar to	Structure/Wall	Future	16.0	13.1	0.156

Table 4-17 Surge levels as calculated for specified reaches.

Note: Surge levels are characterized as being either future or existing, for protection design purposes. Surge levels used as boundary conditions in this study are those designated as ‘existing’.

Breach Location	Peak Surge level	Surge Return Period
Lake	11.3 ft (3.5m)	1/500
River due to storm surge	19.5 ft (6m)	1/1000
IHNC	8.2 ft (2.7m) (level with gate open)	1/100
River due to high discharge	Constant stage of 19.5 ft (6m)	1/1000

Table 4-18 Peak hydrograph levels taken from result frequency analysis.

Note that along the lake, multiple breaches are simulated. The sum of the breach probabilities of the lake breaches equates to the total estimated failure probability of the lake boundary. The 1/500 year surge value is conservatively used as the boundary condition for the modelled lake breaches.

Also, note that in the case of the IHNC breach scenario, it has been assumed a gate has failed to close and that failure occurs at **a 1/100 year** surge event. The effects of this assumption will result in a less catestrophic breach than breaches modelled at lower frequency events.

Hydrograph width
The modelling effort resulted in a suite of 152 storm hydrographs available at several points surrounding the metro bowl. The 152 storms were selected from output of all the storms simulated by the Joint probability model. An innovative sampling method allowed for the optimal and efficient selection of fewer storms to provide a proper characterization of the storm population. The statistical approach is thereby referred to as JPM-OS, (Joint Probability Method with Optimal Sampling). For more information on the JPM-OS, the reader is directed to Resio, 2007.

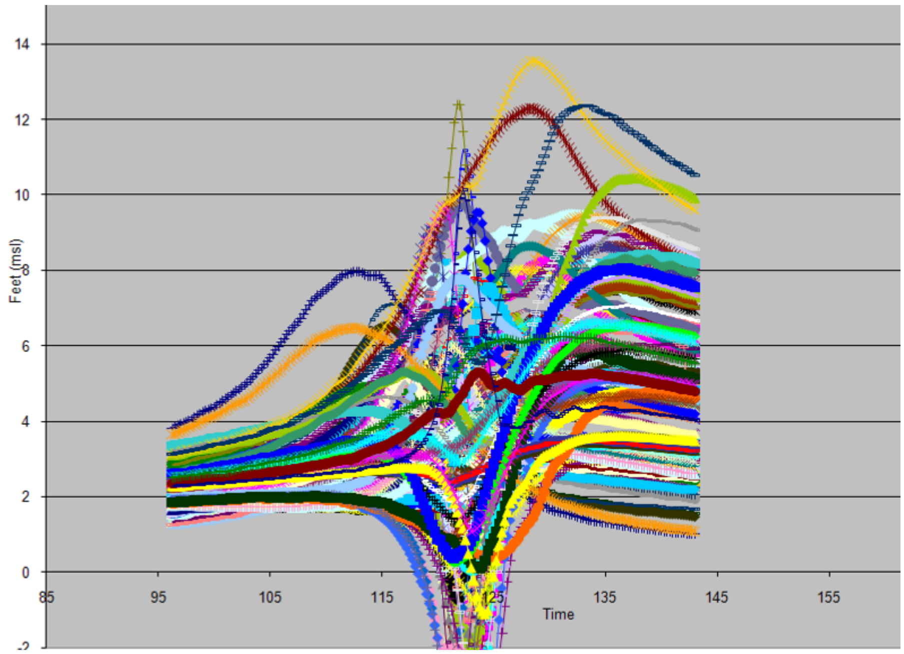


Figure 4-19 Storm suite of 152 storms are provided at various locations surrounding the New Orleans metrobowl.

Initially for input hydrograph widths, the correlation between the peak and width was roughly estimated by plotting the relationship between hydrograph peaks and width. This relationship was estimated for a given water level for at several water levels and resulted in a rough correlation for the river surge hydrograph. Initially, the width of the hydrograph was plotted against the peak surge values. This resulted in a reasonable correlation for the river boundary hydrographs, however correlation between the overall width and peak at the **lake front** boundary were less clear. The lake surge hydrograph is unique due to topography of the area and the effects that hurricanes induce on the lake water levels. As the sensitivity analysis will show, the hydrograph width has a relatively important influence on the breach characteristics. Thus the hydrographs were examined further.

The next approach plotted the slope of the falling side of the hydrograph against the hydrograph peak. This resulted in a reasonable correlation for the river boundary, and an average slope of roughly .2 is used in the boundary condition assumption.

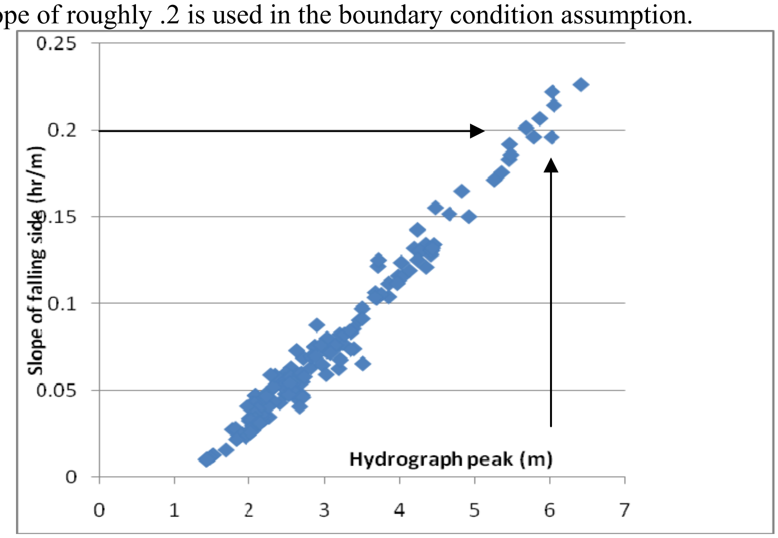


Figure 4-20 Slope of falling side of the hydrogrpah as dependent on the peak of the hydrograph.

For the lake breach, the falling hydrograph slope varied based on water level, so the slope was estimated for upper and lower meters of the hydrograph. An average falling slope is estimated.

(For the lower water levels of the hydrographs, hydrographs provided with peaks in the order of magnitude of the desired return period of the event did not provide data. Thus data from the hydrographs of higher peak surge levels were examined for the lower falling water levels of the hydrograph slope.)

As the hydrograph characteristics impact the results, a more accurate analysis is recommended in future work.

3) **Breach characteristics**

The simulated breach characteristics have an important effect on the progression of flooding and final results, but field evidence for these characteristics is limited (Van Ledden, 2007). Therefore, several breach characteristics must be assumed.

As the flood protection consists of various structures, the failure of various structures could be modelled. However for this work, breaches of similar growth characteristics are modelled. The final **breach width** estimations and **final breach depth** along the Lake and IHNC are based upon documented evidence of breaches caused by Katrina. Below is a summary table of breach characteristics that took place during Katrina.

Initial level embankment (m + msl)	lowest level breach (m +msl)	Width (m)
3.78 (12.4 ft)	-1.40	142
3.93 (12.9 ft)	-1.33	91
3.93 (12.9 ft)	-1.44	91
2.90	2.90	-
3.96 (13 ft)	1.96	20
3.96 (13 ft)	-0.72	50

Figure 4-21 Data based on IPET data, except lowest level breach, which are assumed ground elevations (de Bruin, 2006).

The final depths of the river breach are roughly assumed to be equal to the ground levels. Breach width estimates for river protection are based upon past work which simulated river breaches (Asselman and Heynert, 2003). For the river breach width, other bases may be found by investigation into breach growth models, which could be included in future work.

Location	Initial level Embankment	Lowest level breach	Final Breach width	Initial Width
River levee	6.25m	0m	180m	50m
Lake breach	4.9m	-1.5m	100m	50m
IHNC	3.2m	-1m	100m	50m

Figure 4-22 Summary of breach assumptions for this study. All elevations are in MSL datum (mean sea level).

User input regarding **breach growth** consists of an initial breach width, final breach elevation as well as a time clock of the breach area. For the earthen levee breaches, a simple approach for the breach growth is assumed, where the breach grows to its final depth in the first few hours (2-3) and to its final width after 3 more hours (de Bruin, 2006).

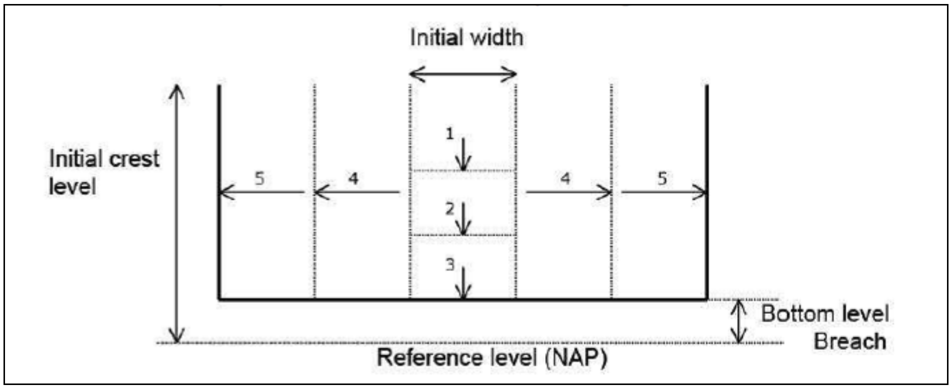


Figure 4-23 Graphical depiction of growth rate assumed for scenarios

However growth rate can be affected by many factors. With increasing water slope of the boundary conditions, the differential between boundary condition and elevation of protected area, the growth rate would be expected to increase. This assumption could also be investigated further.

4.2.3 Further limiting assumptions

Limiting factors of note in the simulations include the following:

- No pumping or internal drainage is assumed as the model does not accommodate for this. In general, for the catastrophic effects expected to result due to breaching, it may be considered that this effect may only play a minor role. For overtopping considerations, effects of this limitation could be investigated further.
- Rainfall is not included in the final water depths. This may affect water depths in the initial time steps of the model (de Bruijn, 2006), but is not anticipated to affect final depths significantly as final water levels would be equal to the lake level.
- It is assumed that there is no debris blockage of breach and no effects of emergency closures of breaches are considered.
- ‘Point’ breaches are used for the simulations, where the inflow is via a single point in the model. In reality, the breach has an actual width through which the water flows. This assumption may lead to a change in flow characteristics near the breach. The effects of this assumption have not been investigated, however the river protection has been modelled in three points to account for the wider breach.

The scenarios modelled are based heavily on uncertain assumptions. These assumptions imply that the results of this study cannot be used in an absolute sense but do give insight into the order of magnitude of results.

4.2.4 Simulation Results

In total 6 scenarios are simulated. Two along the lake, one along the canal, one river breach due to river discharge and one due to storm surge, and finally a multi breach.

Scenario	Location (and Mechanism)
1	River breach due to high discharge
2	River breach due to storm surge
3	Lake breach at West End
4	Lake breach at St. John
5	IHNC breach
6	Multiple breach

The event simulations in SOBEK create output files describing flood characteristics. Inundation maps of the various characteristics can be plotted that display visually the effects of flooding and give insight to the progression of flooding. Such maps can also be used to demonstrate the sensitivity of the flood extent to flood parameters such as breach width and depth. They can serve as a visual representation and assist in communicating information about the flood risk to the public and decision makers.

The following SOBEK output characteristics are possible:

- Maximum depth over run time (m).
- Maximum velocities over run time (m/s).
- Arrival time of water post breach (hr).
- Rise rate of water (m/hr)

A brief discussion of the modelled flood characteristics are described here. For each simulation, maximum depth and arrival time maps are found in the appendix.

Scenario 1 - River Breach due to high discharge
The River breach has been modelled using three breach points in an attempt to more accurately model a large breach width. This breach is simulated at the IHNC fore bank, a location of a relatively low earthen protection reach.

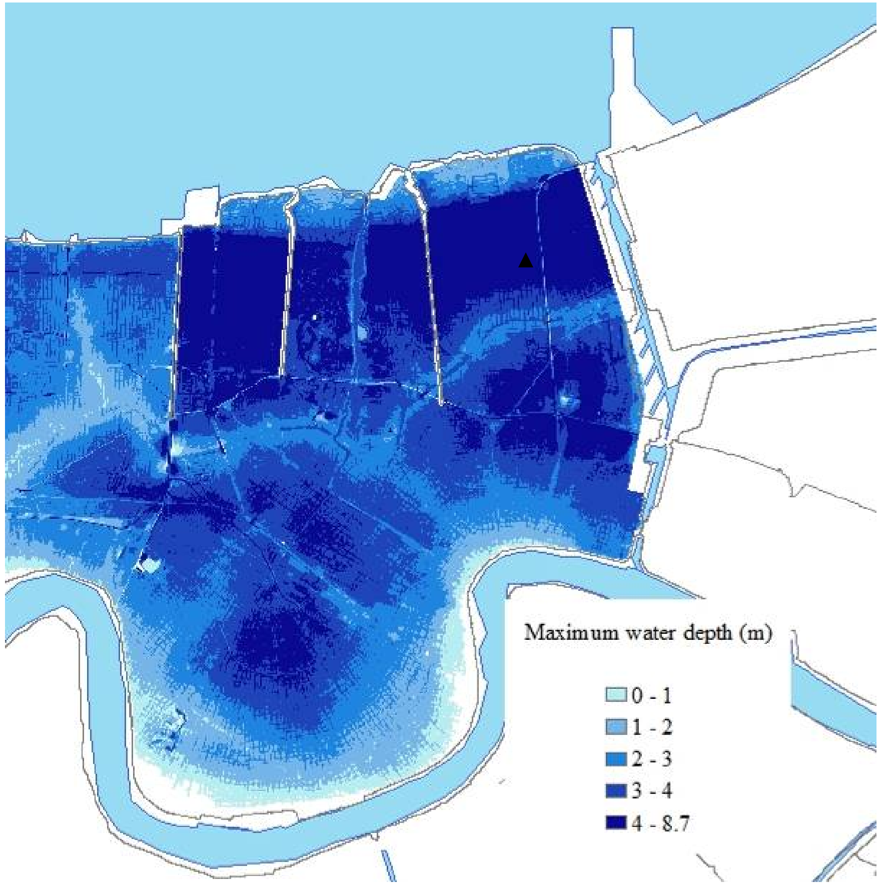


Figure 4-24 High discharge breach, maximum depth (m)

The time of inundation is seen in figure 4.8. It can be seen from the time of arrival that the majority of the metro bowl has filled within the first 30 hours. After the first two days, the flow in is over 500,000,000m^3 implying an average inflow of almost 3000 m^3/s. Flooding due to a river discharge event is especially devastating, as the duration of the flood wave can be up to several weeks, versus a hurricane event where the water rises and falls within a matter of days or hours. A simulation period of 4 days is run, and it is anticipated the metro bowl will continue to fill with water until the breach is closed.

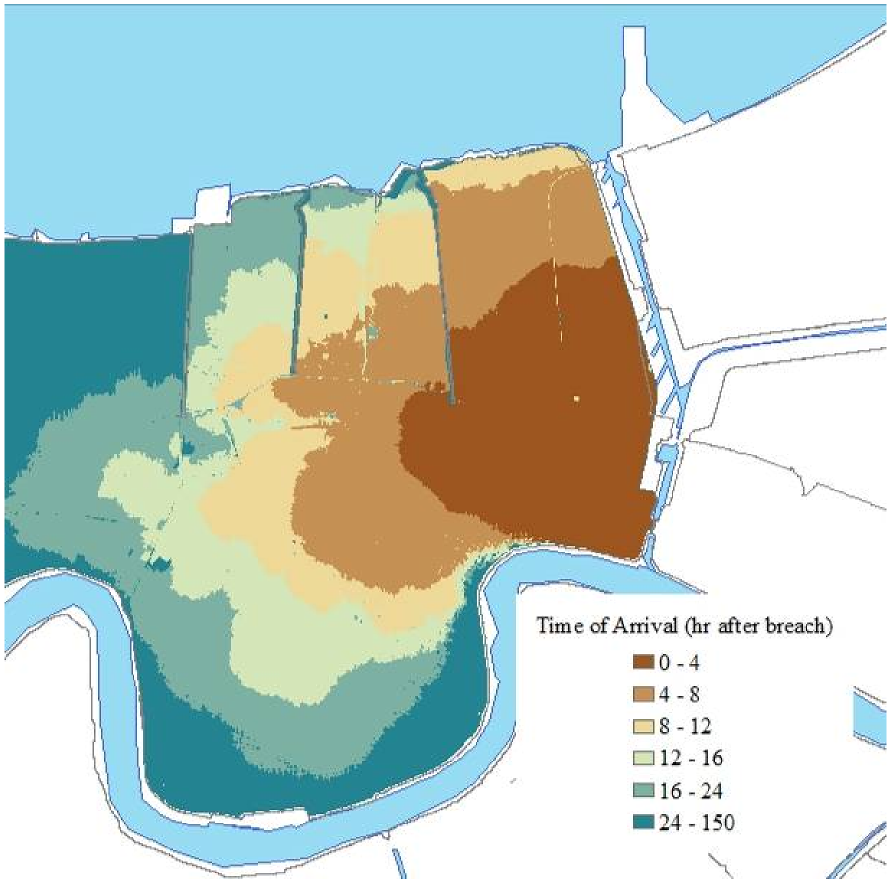


Figure 4-25 River breach due to high discharge, arrival time (hr)

Also noted is that the flood waters will inundate the populated area to the west of Orleans Parish. However these consequences are not considered in this thesis. This assumption could be examined further in future work.

Scenario 2- River breach due to storm surge
The breach caused by river surge up the river is simulated. The breach has been modelled using three breach points in order to better model a large breach width, as was the river breach due to high discharge.

The flow velocities at the breach location are comparable in both river breach scenarios with roughly a maximum of 3m/s. These are relatively low due to the shallow slope of the river bank. Also the time of arrival in the first 24 hours is relatively comparable in both river breach scenarios (see figure 4-25). However, in the breach due to surge, the majority of inundation extents occurred in the first day, as the surge has risen and fallen. It is assumed that had the scenario been run to steady state the bowl would fill until it reached the steady state river water level.

The total volume of storage is 70,000,000m^3, equivalent to the storage due to the high discharge breach in the first 6 hours. It is noted that inflow may be overestimated, as it is modelled as though an infinite volume of water is available to flow into the polder. In actuality the volume of water available for inflow is limited by the capacity of the river, as the river has a finite amount of water volume available. Therefore the flow to the breach may be less than modelled as the inflow to the breach is overestimated (De Bruin 2006).

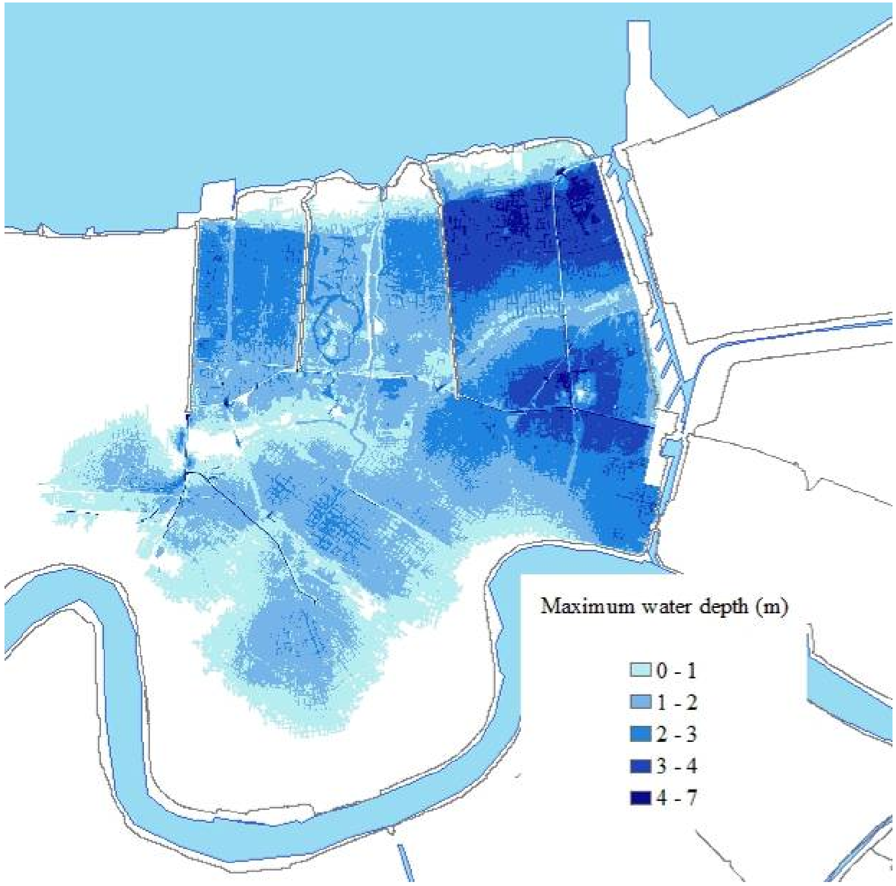


Figure 4-26 River breach due to storm surge, maximum depth (m)

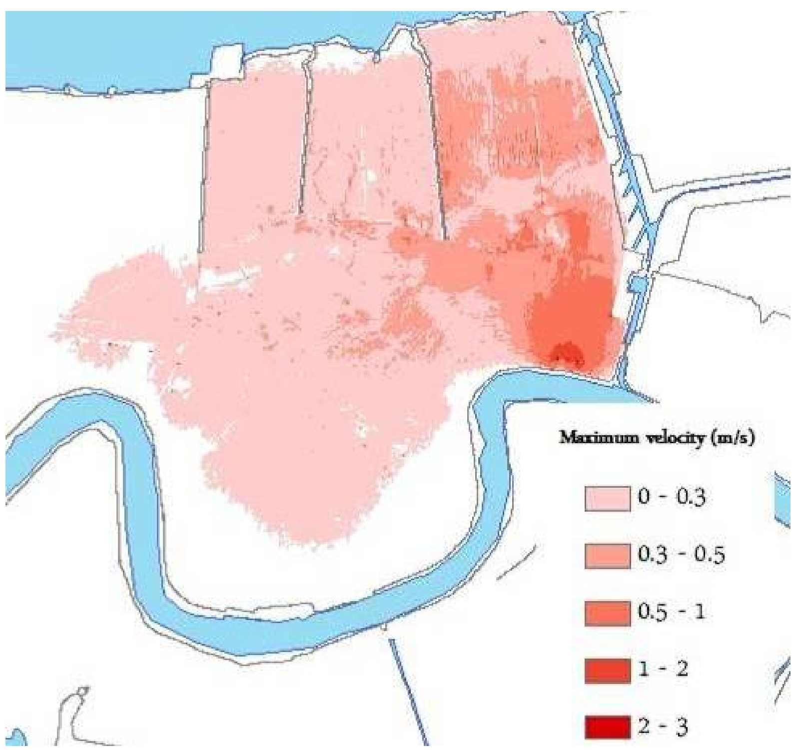


Figure 4-27 River breach due to storm surge, maximum velocity (m/s)

Scenario 3,4 - Lake breaches
Two breaches are simulated at the lake. One breach simulates flooding at the far ‘west end’ and another more to the centre of the metro bowl, referred to as ‘St. John’. The breaches are modelled to fail at the 1/500 year water level.

The west end breach occurs in an area of low elevation, and is assumed to be the breach of an earthen levee. The second lake breach is simulated at the center of the metro bowl. It has comparable flood extents to the west end breach. One difference is the topography where the breach is simulated. The ridge along the lake is of a higher elevation, and the modelled breach in SOBEK had to insure the length of the breach stretched to sufficiently low elevation where water would flow into the bowl.

The water storage due to the west end breach is roughly 135,000,000m^3 and 140,000,000m^3 for the St. John breach. As seen in the river surge where the hydrograph rises and falls, the majority of the flood extents (and therefore majority of the inflow) occur in the first day. Again the inflow into the bowl is expected to occur until the boundary condition of the lake is equal to the level in the polder.

Flow velocities are relatively low in the lake breaches with maximum values below 3m/s.

Scenario 5 - IHNC breach
Next the breach in the IHNC, the canal to the east, is discussed. Compared to the pre-katrina situation, upgraded protection limites the boundary levels in the canal. During Katrina, canal levels were much higher than experienced in the lake. Now, the presence of the Lake Borgne barrier at the east end of the canal prevents the funnel effect discussed in chapter two from occuring, reducing the porbability of high water levels in the canla.

In the modelled scenario, failure in the IHNC breach assumes a gate has failed to close. Therefore during failure the canal water level is assumed comprable to the lake level during the event.

However, as the desing heights of the canal assume both gates close, failure of the canal wall is assumed to occur at a 1/100 year event. This assumption leads to a reduced surge level, and flood depths due to the breach are less than is seen in the lake and river flooding. The resulting storage inflow is only 45.000.000m^3.

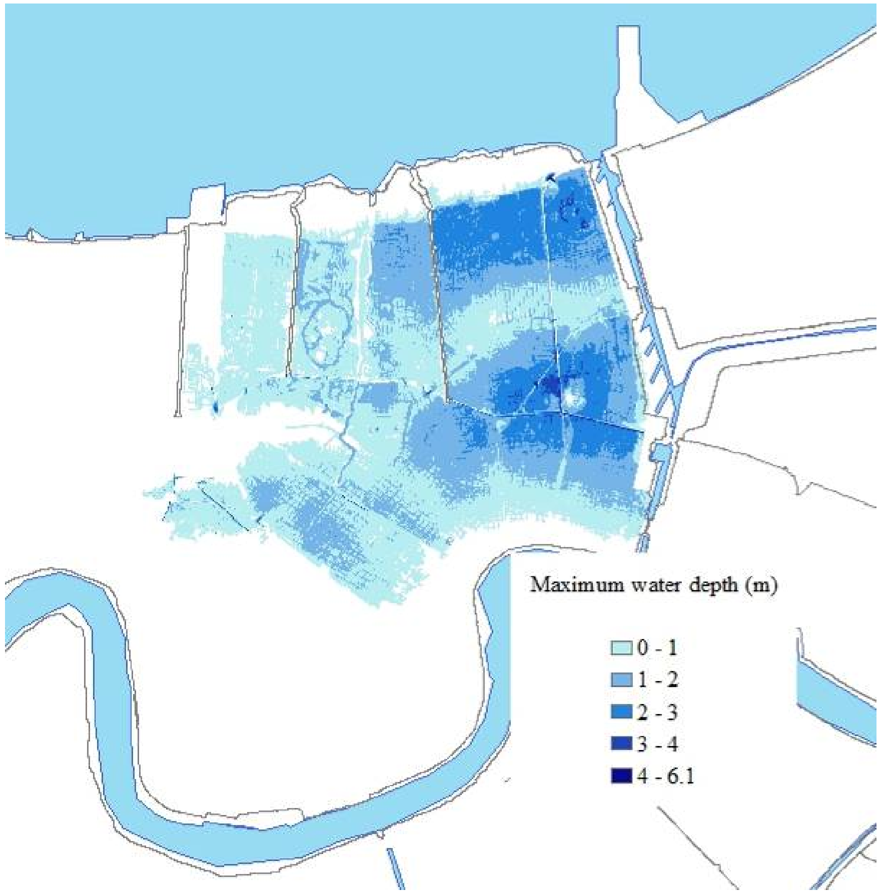


Figure 4-28 IHNC breach, maximum water depth (m).

It is also the case that the canal breach is simulated to have an infinite volume of water available to flow into the polder. In reality, flooding flows are limited by the capacity of the canal. Thus, the actual flow to the breach may be less than modelled and the inflow to the breach is overestimated (de Bruijn, 2006)

Scenario 6 - Multiple Breaches
Finally, a multiple breach scenario depicts flooding due to an extreme catastrophic flood event. This breach has been characterized by the simultaneous simulations of the two lake and IHNC breach already simulated. As velocities are higher near the breach locations, it is assumed velocities of the multiple breach are the same as the single breach scenarios simulated. However, the occurrence of multiple breaches leads to increased arrival time and larger maximum flood depths.

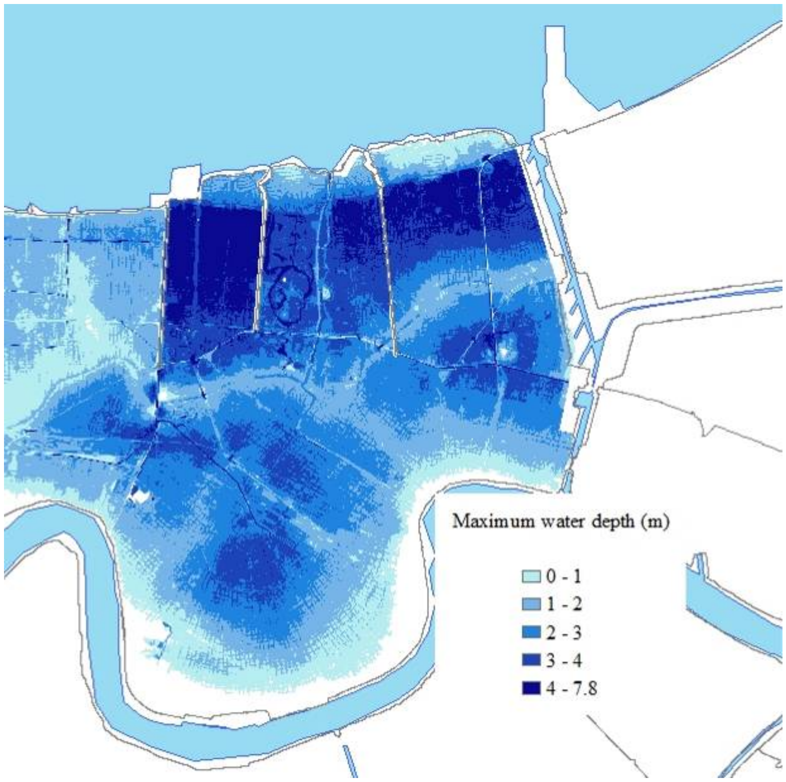


Figure 4-29 Multiple breach, maximum water depth

Katrina flood characteristics

The results of the simulations can be compared with the flood extents during Katrina. The scenarios simulated here vary from the events that led to the metro bowl flooding during Katrina. However, it can be seen that flood characteristics of Katrina are similar to the breaches simulated here. Velocities are in the same order of magnitude, with maximum velocities being around 3m/s. Time of arrival was also similar, as the majority of the flood extents occurred in the first 24 hours. Differences from Katrina considered are that flooding no longer occurs at the canals that stretch into the heart of the city. Therefore high flow velocities occur only at the perimeter of the polder.

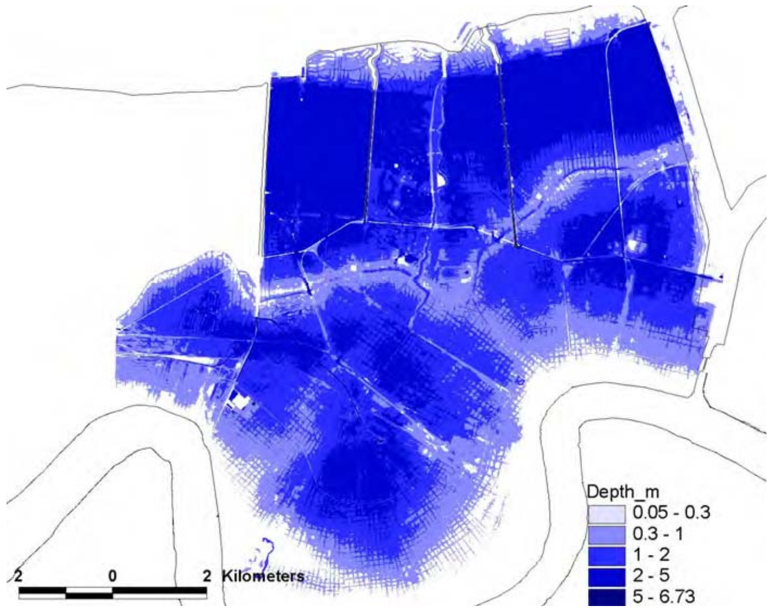


Figure 4-30 Flood depths during Katrina (Source: Louisiana State University, based on satellite images) (de Bruijn 2006).

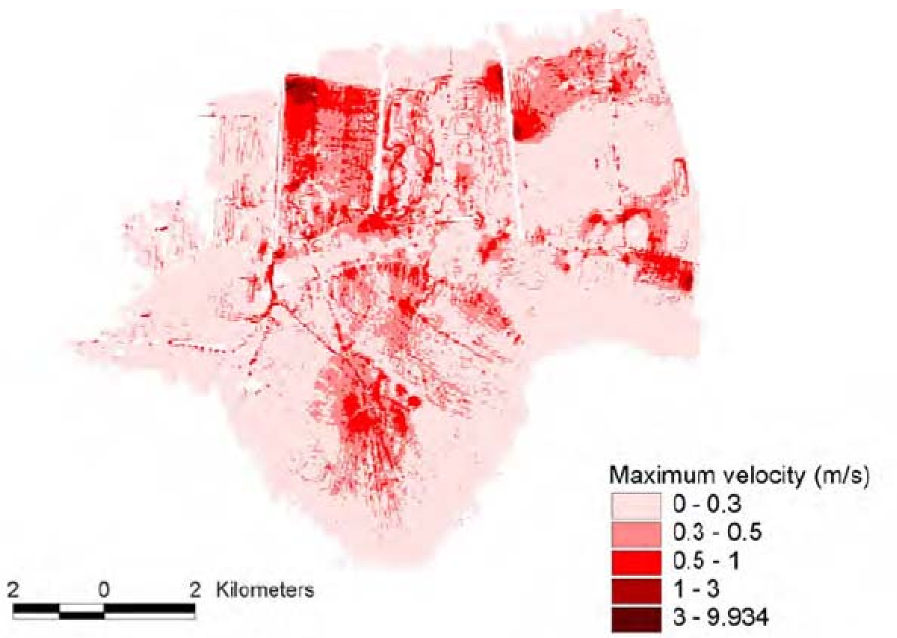


Figure 4-31 Katrina Event, Maximum flow velocities (m/s)

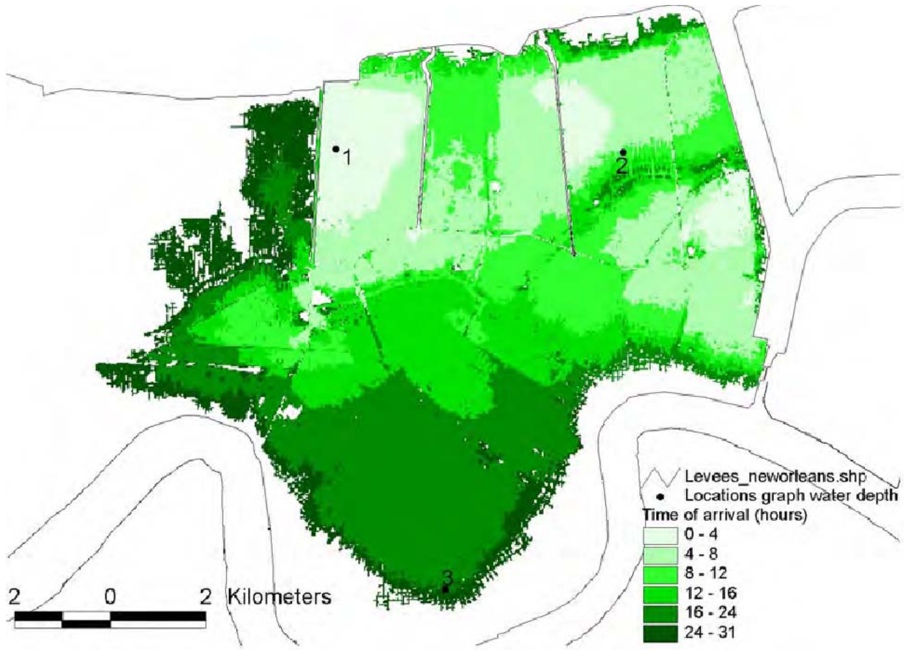


Figure 4-32 Time of flooding during Katrina (De Bruin, 2006)

Conclusions of flood simulations
The breaches simulated provide insight into the ways the city can flood considering the upgraded flood protection system. The flood characteristics of the various breaches show how inundation occurs due to various breaches. Overall, it can be seen that the occurrence of any of the scenarios modelled would lead to devastating consequences.

The majority of the flood extents and therefore water inflow occurred during the first few days (12-24hrs). After this, the inflow levels off but does not reach steady state. It is anticipated that each scenario would ultimately end in the polder filling up to water levels equal to the boundary conditions. Therefore had the scenarios achieved steady state, eventually the polder water level would match the boundary water level, roughly 1 meter above sea level (de Bruin, 2006).

The results presented here are based on the elaboration of a limited number of flood scenarios. It can be seen resulting flood characteristics are dependent on the choice of flood scenarios, specifically the number of breaches and the breach locations. A more complete set of scenarios could be included in a more detailed analysis in order to better represent potential load situations, breach combinations and flood conditions.

4.3 Step 3: Determine consequences

A life loss estimate for each scenario is next determined. GIS tools correlate population data to spatially determined flood characteristics. The flood characteristics resulting from the simulations are analysed to determine a deterministic consequence estimate for each scenario. A life loss model proposed by Jonkman (2004) provides an empirical method that relates lives lost to flood characteristics.

In the model the expected fatalities (N) are defined by:

$$N = F_D * N_{EXP} \qquad (eq\ 5\text{-}1)$$

Where:
$\mathbf{F_D}$ is the '**mortality**' (or mortality rate), defined here as the number of fatalities divided by people exposed to the flooding.
$\mathbf{N_{EXP}}$ is the total exposed population affected by the flooding.

The exposed population can be further broken down into:

$$N_{\text{EXP}} = N_{\text{PAR}}(1 - F_E). \qquad \textit{(eq 5.2)}$$

Where:
$\mathbf{N_{PAR}}$ is the original population at risk
$\mathbf{F_E}$ is the evacuated and sheltered percentage of the original population.

It is seen the model takes into account of the **characteristics of a flood event**, the **population at risk** and **percentage of those evacuated** or sheltered. The actual number of fatalities due to a flood event is of course dependent on many other factors such as the vulnerability of the population, the type of housing exposed to flooding, temperature, time of day, behavior of victims. For the purposes of this simplified evaluation, only the three parameters listed will be considered:

4.3.1 Mortality, F_D

As a basis for developing mortality functions, observations of historical flood events are analyzed. Jonkman expounded upon existing work which examined data from the North Sea flooding of 1953. This coastal event inundated the coastal region of the Netherlands and other nations including the UK and Denmark. In the Netherlands alone, it flooded more than 200,000 hectares and took 1800 lives.

The result of Jonkman's analysis determined relationships between flood parameters and fatalities. This allows a mortality rate that is a function of the characteristics of the flooding. In the approach, the characteristics of flooding are divided into zones based upon the way life loss can occur. A similar approach dividing flood characteristics into three zones, had been previously developed (Waarts, 1992). Therefore each zone is characterized by a **specific criterion** and results in a mortality rate for that zone.

The original model by Jonkman results in three 'mortality functions' relating the probability of drowning to specified hydraulic circumstances. (Jonkman and Asselman, 2003).

1. A 'breach zone, 'near the breach where flow velocities are high. The conditions in this zone lead to the collapse of buildings and instability of people standing in the flow. Velocity and water depth are used to characterize this zone.
2. A 'rise rate' zone, characterized by rapidly rising waters. People in this zone may have difficulties getting to shelter or higher ground. The rate of rise, the rate at which the water depth increases during flooding, characterizes this zone.
3. A 'remaining zone,' in which the flooding occurs slower. In the remaining zone, the flow velocity is the most important factor (Maaskant, 2007).

New Orleans mortality functions
After the events of Katrina, Jonkman's approach was then applied to the situation of New Orleans. Preliminary data of exposed population, consequential flooding fatalities and estimated flooding characteristics were used to determine mortality functions for New Orleans.

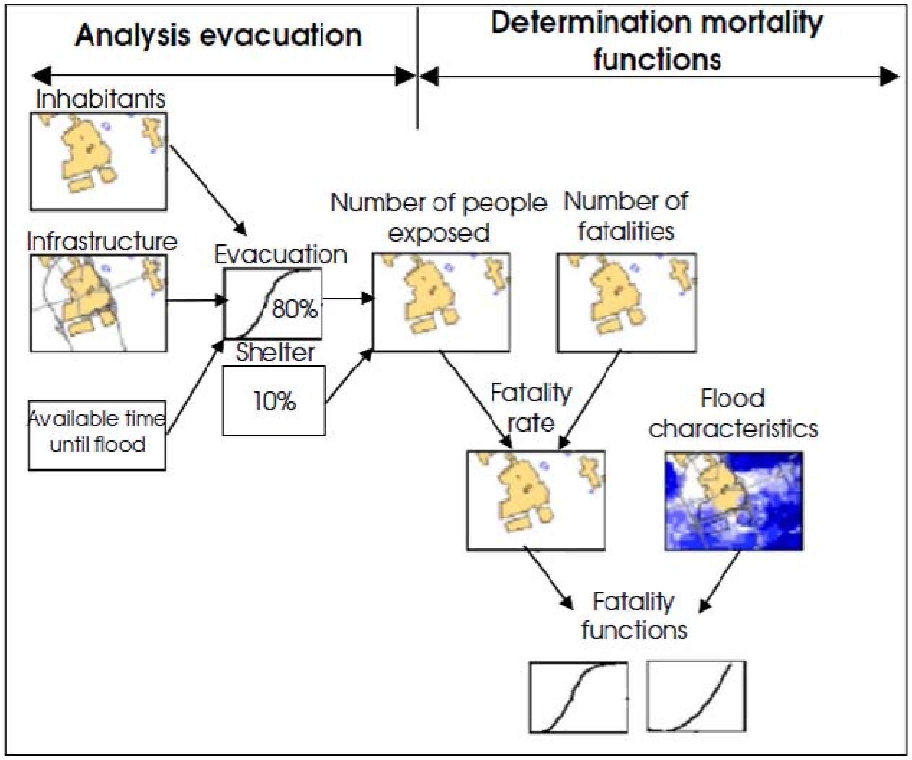


Figure 4-33 Approach used to determine mortality functions (Asselman & Jonkman, 2003, Maaskant, 2007).

Flood characteristics resulting from the storm were simulated by de Bruijn and combined with spatially observed population and fatality data. For example, mortality rates per census tract were calculated below and associated with a depth of flooding resulting from Katrina.

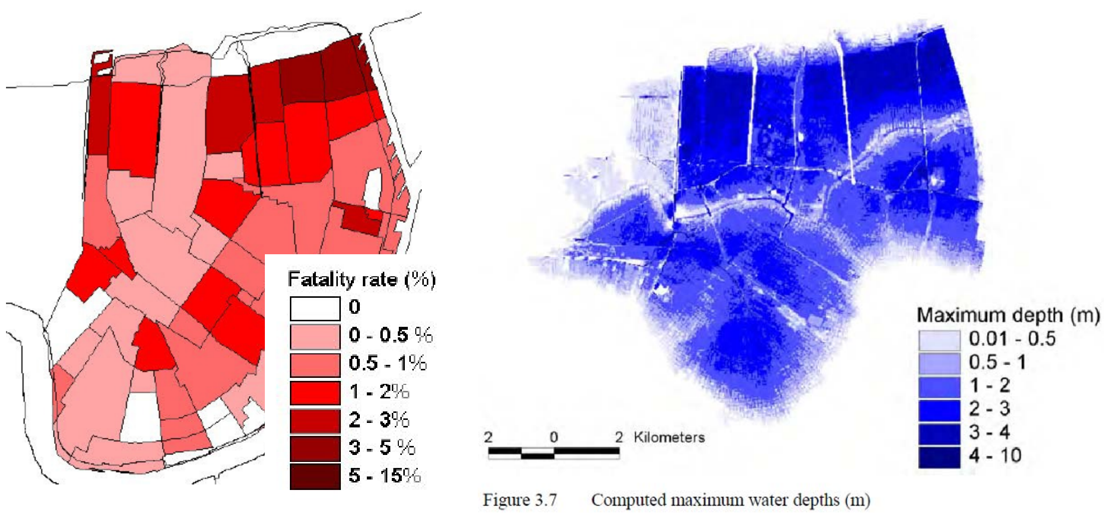
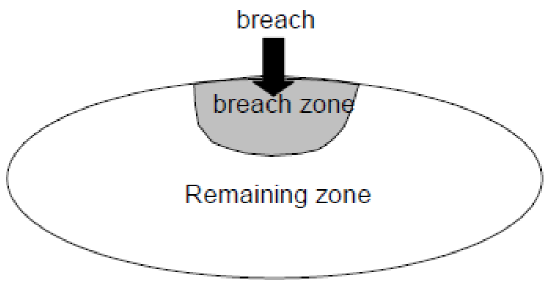


Figure 4-34 Spatially observed mortality rate as estimated by Maaskant (2007) is combined with flood characteristics as calculated by de Bruijn to determine mortality functions for New Orleans.

The analysis resulted in updated mortality functions.



Breach zone: $hv > 5m^2/s$
⇨ $F_D = 0{,}053$
Remaining zone ($hv < 5m^2/s$):
⇨ $F_D(h) = \Phi_N((\ln(h) - 5{,}20)/2{,}00)$

Figure 4-35 Mortality functions derived from data for New Orleans flooding (Maaskant, 2007).

The analysis of the New Orleans data using Jonkman's model found that the 'rise rate' flood characteristic had little influence on the number of fatalities. Therefore two 'zones' remain in the flooded area for which relationships between flood characteristics and mortality rate can be derived (Maaskant, 2007) .

1. Breach zone: the zone near the breach with high velocities. There is building damage and the flood conditions are severe. Both flow velocity and the water depth are the characteristics that determine this zone. Breach zone is defined as area where max velocity and max depth are greater than $5m^2/s$.

2. Remaining zone: For New Orleans, all other areas outside the breach zone are considered the remaining zone. Here loss of life is considered to be only water depth dependent.

The mortality functions are utilized to determine life loss for each scenario. The hydraulic characteristics of the simulated flood scenarios are analyzed using the geo referencing software ARCGIS. The SOBEK output of each simulation is spatially plotted to determine which criteria (and therefore zone) the characteristics meet. This provides information for determining the spatially varying mortality rate.

The mortality rate however does not estimate the number of fatalities, but is solely a characteristic of a location. To determine fatalities the number of people exposed must be assumed and therefore two further considerations must be made: the distribution and quantity of the initial population in the area and the percentage of the initial population exposed to the flood event.

4.3.2 Population at Risk, N_{PAR}

The population at risk takes into consideration the distribution and quantity of the initial population of the area. To estimate this, the 2010 US census data for the 'Orleans Metro bowl' area considered is roughly 221,200 people, (65% of the entire population of the Orleans parish.) The distribution of population is plotted by census tract.

A comparison of population at risk to that of the 2000 US census data per tract depicts the change in circumstances from the pre-Katrina situation. A total population of roughly 312,000 is the pre-Katrina population at risk. Additionally, a general decrease in population can be seen in the areas flooded by Katrina, which are largely low lying areas. Reasons for this redistribution are generally the slow repopulation of the flooded region, which was mostly, but not entirely below sea level. Comparing the population census of 2000 to 2010, population **above** sea level is estimated to have increased from 39% to 45% (Campenella, 2011). This would result in a slightly lower mortality than had the population from the pre-Katrina situation been run.

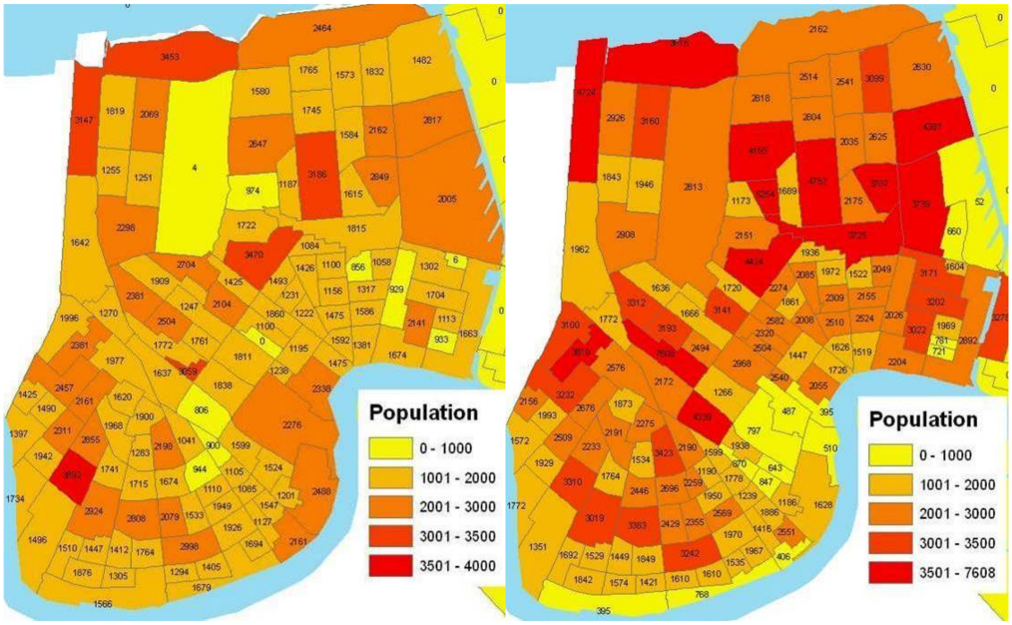


Figure 4-36 A comparison of population distributions. 2010 can be seen on the left compared with the population distribution of 2000 on the right. There are some discrepancies due to tract boundary changes in the two graphs.

4.3.3 Exposed population, N_{EXP}

The final parameter involves the fraction of the at risk population that is assumed to be directly exposed to flooding. Several considerations can be taken into account. Opportunities for shelter, opportunities for evacuation, and warning time are important assumptions for the 'exposed' population. For simplification, here the exposed population is assumed to be defined by the following function:

$$N_{\mathrm{EXP}} = N_{\mathrm{PAR}}(1 - F_E). \qquad (eq\ 5.2)$$

Where F_e is the percentage of N_{PAR} evacuated or sheltered during the storm.

For a hurricane event, a relatively long warning time is available. The opportunity for evacuation has improved since Katrina and is readily available to entire population at risk. When Hurricane Katina hit landfall it is found that roughly 20% of the overall population of the New Orleans metro bowl remained (Jonkman, 2009, Boyd 2011) and of that 20%, another 5 % were 'sheltered' (Boyd, 2011). If the assumption is made that awareness is improved, the evacuation or sheltering of 90% of the total population can be assumed for the hurricane event.

For a breach due to a high river discharge, a worst case scenario could consider that the breaches occur relatively unexpectedly. However is assumed that there is a long warning time, and the possibility for evacuation is readily available. Also, it could also be assumed that the opportunity for shelter is still an option, for example moving to a higher elevation, or to the third story of homes. Therefore 90% of the population is assumed to be unexposed.

4.3.4 Consequence Results

The life loss model is applied to each scenario. As mentioned, two mortality functions can be applied, depending on the hydraulic characteristics.

Breach Zone
According to Jonkman's criteria, to be within the 'breach zone,' the product of the maximum velocity and maximum depth characteristic of a given location is higher than $5 m^2/s$.

During Katrina, a 'breach zone' occurred in the St Bernard parish east of the IHNC. A floodwall failed when a 400 foot section of wall collapsed simultaneously due to a barge collision. The water in the IHNC had reached very high levels (14ft, 4.6m). The building damage was extensive and the breach resulted in many fatalities. The product of the water depth and the flow velocity is for almost the entire lower 'ninth ward' larger was estimated to be larger $5 m^2/s$.

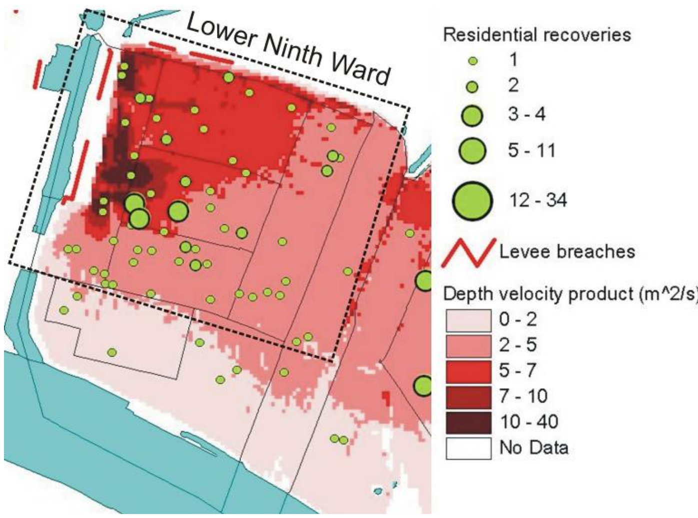


Figure 4-37 Spatial distribution of the recovered fatalities and the depth-velocity product determined in flooded areas of the ninth ward (Jonkman, 2007).

For each scenario simulated in this thesis, the product of these characteristics has been analyzed spatiality. The results of the flood simulations show that in the metro bowl, areas that meet the criterion of a 'breach zone' are rather small. Flow velocities are not significantly high in the New Orleans metro bowl. As such, it is assumed they do not contribute significantly to the life loss estimates. Therefore the analysis of flood characteristics shows that for the area studied, the breach zone is not a determinant of fatalities.

Remaining Zone
The second zone, which considers fatalities due to other conditions, is the 'remaining zone '. This zone applies the water depth dependent function. The metro bowl is aggregated into sections over which the maximum water depth characteristic is averaged. This choice of area is important, as choosing too large an area results in a large variation of characteristics; while too small of an area can lead to too low a value of exposed population per area. The population census tract is utilized as similar studies have used areas of comparable size. An average mortality for each tract is then calculated resulting in a spatial distribution of mortality rate.

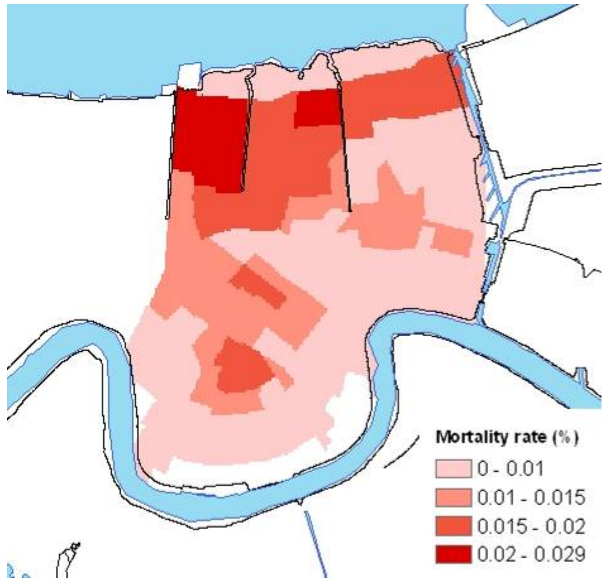


Figure 4-38 Mortality rate per tract for 'West End' simulation of this study.

Mortality rate results
When combined with the estimated exposed population of each tract, the number of fatalities estimated from each scenario is evaluated. When the spatially distribution of mortality is associated with the spatial distribution of population, an expected fatality value is determined.

Scenario	Location	Resulting fatalities	Exposed population (assuming 90% evacuation)	Area Flooded (km^2)	Overall Mortality rate (%)
1	River breach, high discharge	450	22118	102 km^2	2%
2	River breach, storm surge	150	18160	91 km^2	0.8%
3	Lake, West End	170	16975	95.7 km^2	0.9%
4	Lake, St, John	167	19977	89 km^2	0.8%
5	IHNC	55	12662	51 km^2	0.4%
6	Multiple Breach	280	22118	102 km^2	1.3%

Table 4-39 Mortality rate results

The mortality rates resulting from the breaches vary from .4% for the breach at the IHNC, simulated to occur during an event of a higher return period, to 2% for the scenario of a breach due to a high river discharge simulation.

Sensitivity analysis
As these results are based on assumptions and estimates of the modelled breaches, a rough sensitivity analysis is useful for showing the variation in mortality rate due to scenario parameters. For a given scenario, selected parameters adjusted and the mortality rate

determined. Adjusted parameters are final breach width, hydrograph width and moment in the hydrograph when breach occurs.

West End Scenario		Final breach width		Input Hydrograph		Breach Pre-peak of hydro.
	Base case	100% Narrower	100% Wider	100% Longer	100% Shorter	
Loss of life	63	50	65	130	50	64
Mortality	0.42%	.40%	.48%	.69%	.40%	.42%

Table 4-40 Results of sensitivity analysis

Note that the sensitivity model runs were run for a slightly shorter time period (after two days of flooding) to reduce model run time. However, it is expected that the effects on mortality rate seen here indicate the effects of breach characteristics on scenarios of longer duration.

The results of the analysis show that mortality rate is not very sensitive to final breach dimensions or point in time when breaching occurs. However, the mortality rate was sensitive to the width of the hydrograph. In this case greater maximum water depths are seen. Due to this parameters impact on the results, the width of the hydrograph was analyzed further in an effort to provide a more accurate estimate.

Additional parameters could be included a more robust analysis, such as the initial breach width, rate of breach growth, and final breach depth. However, the initial breach width and rate of growth are anticipated to have more of an effect on the initial velocities rather than final flood depths and extents. As the damage estimates are based on the depth dependent function, these parameters are not expected to overly affect the mortality results. The final breach depth parameter, however, could be investigated more thoroughly as it would affect flood depths and extents.

Discussion of mortality rates

The flooding of the metro bowl during Katrina was characterized by multiple breach events, wall failures, overtopping and other events. The flood characteristics and therefore mortality rate resulting from Katrina varied from the scenarios modelled here. For comparison, preliminary post- Katrina mortality rates as found in detailed analysis resulted in mortality rates of .6% for the metro bowl (Boyd 2011).

	Flood Deaths	Flood Exposed	Flood Fatality Rate (Deaths per Exposed Person)
Central New Orleans	250	39,913	0.00626
Lower 9th / St. Bernard	158	6,914	0.02169
New Orleans East	54	12,800	0.00007
Plaquemines Parish (*)	6	250	0.024
Total	468	59,877	0.00782

(*) Data for Plaquemines, which actually consists of two polders separated by the Mississippi River, provided by the parish emergency director (St. Amant 2006) and included for means of comparison.

Figure 4-41 Rate of mortality caused by Katrina (Boyd 2011).

The mortality results presented in this study for the various scenarios are within the same order of magnitude as seen during Katrina.

Conclusions of consequence determination
Given the above sensitivities and the lack of knowledge of the exact course of flood events, the resulting fatality values are thought to be indicative and to provide insight in the **magnitude** of consequences. Mortality rates for resulting scenarios depend on many assumptions that would affect the resulting mortality. However the following conclusions can be drawn:

- The updated protection leads to a considerable reduction of load on the IHNC therefore mortality due to this breach is expected to have been reduced.
- The upgraded protection system has shortened the protection along the lake, resulting in the reduction of flooding at the interior of city.
- Results show that for a breach due to a high river event, the mortality is roughly 2% due to large flood depths. The consequences due to river flooding in the specific case of New Orleans are very disastrous.
- The Multiple breach scenario results in higher mortality rates than the single breaches increased depths.
- The ultimate fatalities resulting from each scenario depend on the population distribution throughout the city as well as breach circumstances.
- The resulting fatality values can be roughly validated by mortality rates determined during Katrina, roughly 6% for the metro bowl polder.
- Evacuation assumptions for a river scenario can be researched further.

The final step in the quantitative risk evaluation is to combine the results of the consequence and probability estimations.

4.4 Step 4: Quantify risk

Finally, an estimation of risk to life is determined for each scenario by combining the probability of each event and the resulting consequences for the event. The risk result is represented in risk metrics, such as individual risk, and societal risk and expected value (number of fatalities /yr). General analytical formulas used to quantify these metrics are described here.

4.4.1 Individual risk

Individual risk (IR) is defined as the probability of an individual residing in a given area perishing as a consequence of flooding. Therefore it is a characteristic of a location, and defines the risk level spatially. It is calculated as the product of the probability of the event and the mortality rate at a certain location given the flood event.

$$IR(x,y) = \Sigma P_i, F_{D|i(x,y)} \qquad \text{(eq 5.3)}$$

Where:
IR(x,y) is Individual risk at location (x,y) [yr-1];
Pi is the probability of occurrence of flood scenario i [yr-1];
FD|i(x,y) is the mortality at location (x,y) given flood scenario i [-]

Individual risk is not a characteristic of the exposed population, but of a location. Thus it is calculated for a permanently present individual. To include the opportunity that a person has evacuated or sought shelter, the IR value is multiplied by the exposed percentage of the population.

$$IR_E(x,y) = \Sigma P_i, F_{D|i(x,y)} \quad (1-Fe) \qquad \text{(eq 5.4)}$$

To determine the IR for each scenario, mortality as per the fatality functions is determined spatially for each scenario and then multiplied with the probability of the event's occurrence. The result is a spatial representation of the individual risk for each scenario. The same is done for each of the scenarios and the spatial representations of the individual risk are added, producing a representation of the individual risk of the area. Thus, the location-specific individual risk takes into account the contribution of the failure of each element.

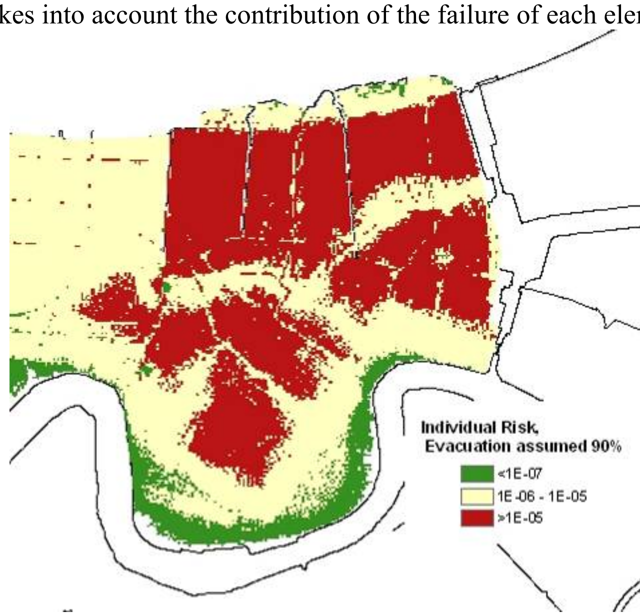


Figure 4-42 Map of individual risk assuming 90% evacuation

The IR values vary from a maximum value of **4.97 x 10^{-5}** to 0. As the IR is a function of the depth dependent mortality function, it corresponds roughly with the area topography. Also it is seen that the location of breaches around the perimeter of the region affects the IR result as well. High individual risk areas are seen to roughly correlate to higher mortality rates as a result of Katrina.

Figure 4-43 Mortality rates resulting from Hurricane Katrina (Boyd 2011).

Individual risk maps indicate the distribution of risk over the population of the exposed area. Thus it is a spatial measure useful for planning and residential zoning. Also the maps can be used in assisting risk communication or increase the dissemination of information to the public.

4.4.2 Societal Risk

Societal risk, defined as the probability of exceedance (in a year) of a certain number of fatalities due to one event in a given population (Jonkman, 2007). It describes the probability of a large, multi-fatality event.

Expected value

Societal risk can be measured most simply as the expected value of the number of fatalities per year.

$$E(N) = \int n \, f_{N(}n) \, dn \quad \text{(eq 5.5)}$$

Where $F_N(n)$ is the probability density function of the number of fatalities per year. The expected value is the weighted average of all possible outcomes where the probabilities of the resulting fatality estimates are the weights.

In this study, the expected value is estimated with the scenarios where the E(N) for each scenario is the product of the annual probability with the estimated fatalities. As the consequence estimate and therefore expected value is based on the population exposed, the effect of evacuation is assumed in the societal risk quantification.

Scenario		Event probability	Resulting fatalities	Expected fatalities E(n)
1	River breach due to high discharge	0.0001	450	0.45
2	River breach due to storm surge	0.005	150	0.17
3	Lake, west end	0.0035	181	0.905
4	Lake breach St. John	0.002	167	0.835
5	IHNC breach	0.0035	55	0.1
6	Multiple breach	0.0002	280	0.056
		∑0.0112		∑2.0

Table 4-44 Overview of expected values and mortality rate for each scenario

The sum of the probabilities of the flood scenarios equals the flooding probability of the system. Note that it is assumed that the two lake scenarios have the same probability of occurrence adding to be the failure probability assigned to failure of lake protection. Had more scenarios been included, the corresponding probability of the lake scenarios would be reduced to maintain the failure probability assumed for the lake protection.

The expected value determined from each scenario can give insight into the risk contribution of the individual protection elements to the overall risk of the system. It can be seen that the IHNC risk is smaller. With the inclusion of the barrier, it is assumed this is the most reduced element in the system. The lake breaches provide the greatest risk to the city. However, it can be concluded that no specific element of the protection system is overly contributing to the risk. Was this found to be case, risk reduction measures in the final chapter could be focused on reducing risk at a specific area of the protection.

FN curve
The societal risk is commonly depicted with a frequency- number curve, or ‘FN’ curve which plots the probability of exceedance (in a year) as a function of the number of fatalities, and is usually shown on a double logarithmic scale.

To determine the FN curve, the following steps are followed:

Using the probability and fatality estimates from each scenario, the probability density function of the scenarios was determined (fn(n)). From this, the probability distribution function is found as the integral of the density function.

$F_{N(} n) = \int f_{N(} n)\, dn$. (eq 5.6)

And the probability of exceedance of the fatalities is determined by subtracting the probability distribution from 1.

$P(N > n) = 1 - F_N(n)$. (eq 5.7)

This is the probability that a certain number of fatalities will be exceeded. This probability is plotted against the fatalities estimate for each scenario.

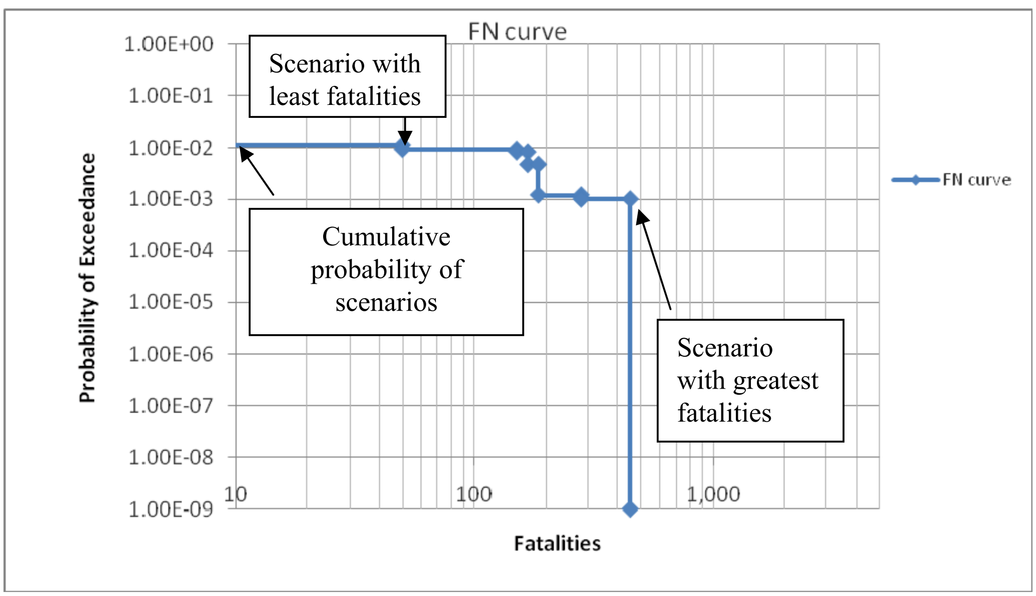


Figure 4-45 FN Curve for New Orleans Metro Bowl (based off Jonkman 2007).

The intersection with the y-axis is the cumulative probability of the scenarios; the metro bowl’s overall flooding probability of (1.12×10^{-2} yr^{-1}). The intersection with the x-axis is the consequence result of the scenario with the largest consequences, roughly 450 fatalities.

The probabilities and fatality estimates for the scenarios modelled are combined to determine **expected number** of fatalities. The expected value can also be found by integrating the area under the FN curve. Again, this yields a risk estimate in terms of the ‘expected value’, $E(N)$ of 2 fatalities/year.

Thus the FN curve serves as a useful depiction of the probability with respect to the magnitude of consequences and represents the societal flood risk in an easily understandable way. Also, it can be compared numerically with other risk events (Reidstra, 2010).

4.4.3 Uncertainties

The scenario results provide a deterministic estimation of fatalities for each scenario. As the fatality results are first order estimates, they are uncertain and will lead to uncertain risk estimates. To attempt to quantify the uncertainty associated with the **consequence** estimates, given an assumed data set of scenario results, the following sources of major uncertainty in the estimates are considered:

- Exposed population (assumption of peoples evacuated)
- Model uncertainty
- Other

Uncertainty in consequence estimates can be depicted in an FN curve. For example, to account for the uncertainty in the assumption of exposed population, bounds are estimated for the percentage of population evacuated.

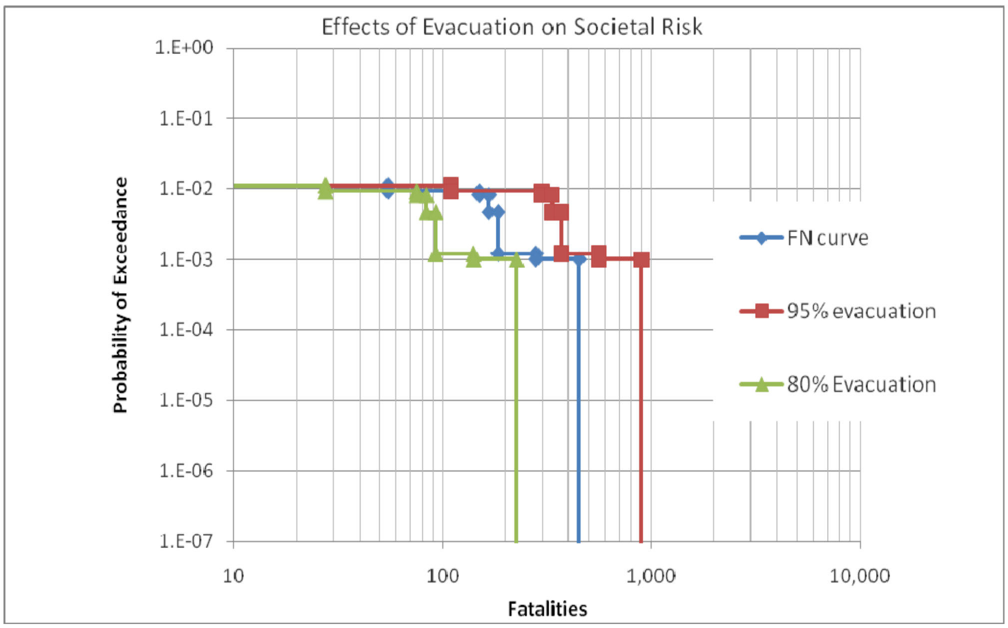


Figure 4-46 FN curve representing the effect of evacuation on societal risk .

To account for model uncertainty or the variation in observations with the mortality function fitted to the observations, bounds proposed for the original model developed by Jonkman (2001) are used.

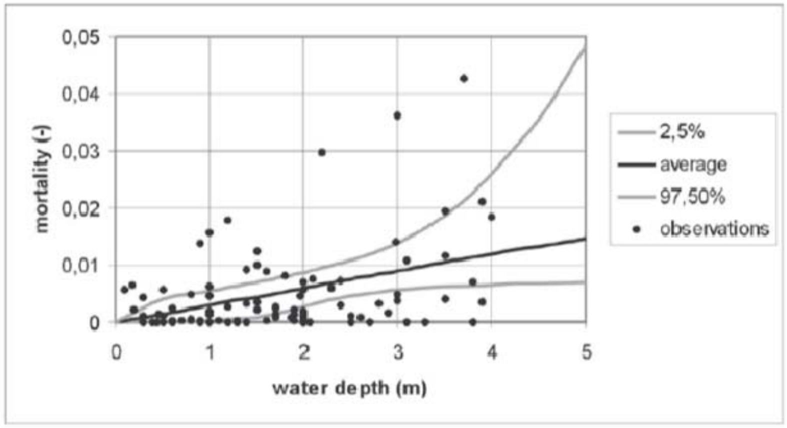


Figure 4-47 2.5 % and 97.5 % confidence intervals showing model uncertainty for mortality function (Jonkman, 2008).

By combining uncertainty bounds for the mortality functions and uncertainty of evacuation percentages, an overall uncertainty evaluation is calculated for the consequence estimates. The uncertainties are assumed to be independent (Jonkman, 2008). A rough estimation for the overall consequence uncertainty can be depicted:

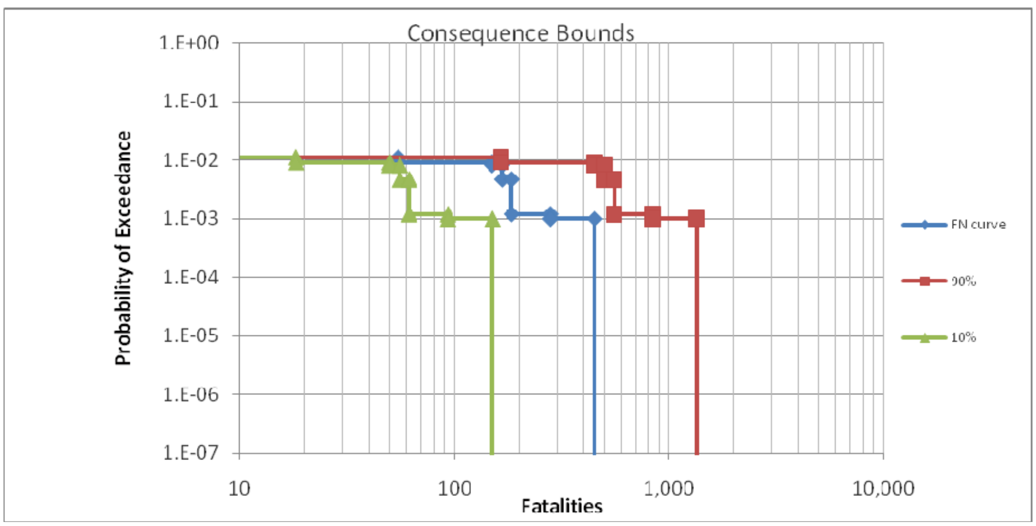


Figure 4-48 Consequence uncertainties including major uncertainties (evacuation and model uncertainty).

Therefore while there is much uncertainty involved in this analysis, the results provide a rough order of magnitude of fatalities. The five steps described in this chapter outline a detailed analysis resulting in the quantitative risk to life estimate for New Orleans. The next step in the overall risk management approach is to the evaluation of risk to determine if risk reduction is necessary.

5 Risk Evaluation

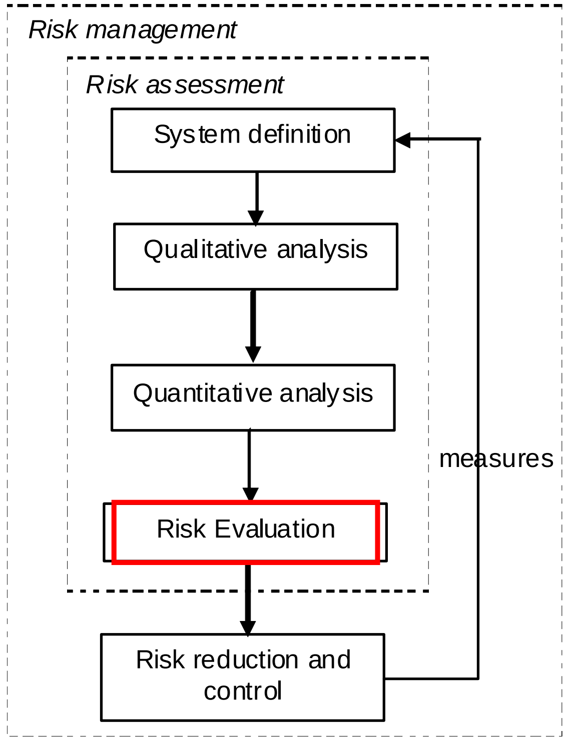


Figure 5-1 Steps of a Risk Assessment (Jonkman, 2007).

With the risk having been assessed, it can now be evaluated. To do so, it must be compared against tolerable risk criteria. Various frameworks and acceptable risk criterion found in literature or currently employed in engineering applications are discussed. The risk is compared with societal and individual risks found for other common civil infrastructure such as dam safety or nuclear effects.

5.1 Individual Risk Criterion

Criterion of Vrijling

An acceptable risk framework developed by Vrijling (1995) provides a basis for comparing of individual risk (IR). In this framework, the tolerable individual risk limit is based on the analysis of fatality statistics. The analysis results in a minimized 'death rate' of 10^{-4}, which was decided upon by examining the various ways in which people die. It is then decided that an event that puts a person at risk should 'add less than 1%' to the already existing probability of death.

Thus Vrijling's criterion is described by the following equation:

$$IR<\beta*10\text{-}4 \qquad \text{(eq 5-7)}$$

Where β is a' policy factor', reflecting an individual's voluntariness to the event. The higher the degree of voluntariness, the higher the β value used, and the higher the tolerable risk. The beta value allows the idea of 'preference' with regard to risk to be incorporated into this function. Proposed values for this factor are between .01 for involuntary activities and 100 of voluntary activities for personal benefit.

Beta	Voluntary	Benefit	Example
100	Completely voluntary	Direct benefit	Mountaineering
10	Voluntary	Direct benefit	Motorbiking
1	Neutral	Direct benefit	Driving a car
0.1	Involuntary	Some benefit	Factory
0.01	Involuntary	No benefit	LPG station

Figure 5-2 Policy factor values (LPG: Liquefied propane gas).

In the case of dikes that protect from flooding urban areas, a relatively involuntary risk, the β factor usually is between 1 and .01. If a β value of **.1** is used, describing an 'involuntary with some benefit' activity is used, the result is an IR limit of $\mathbf{10^{-5}}$.

This framework can be applied in other fields, such as **industrial safety** to determine tolerable individual risk levels. For example, a tolerable risk level of $\mathbf{10^{-6}}$ is determined to be appropriate considering hazardous installations near residential areas with **no direct benefit** to the individual (Vrijling, 2011).

USACE criterion

Another source for IR criterion proposed for flood defences is proposed in the updated guidelines for dam and levee safety by the US Army corps of Engineers. Within these guidelines, tolerable risk is defined as the risk that society is willing to live with so as to secure certain benefits' (USACE 2010). As a basis for their tolerable risk criterion, the USACE looked to existing limits currently used in industry (dam design, chemical contamination and land use planning) (USACE Feb 2010). The consensus of this discussion resulted in a limit of 10^{-4}, criterion found in design criteria for dam design (USBR, 2003). It was agreed upon that this limit value had achieved consensus among government and private sectors institutions engaged in safety management. This limit, like the one proposed by Vrijling, is derived from fatality rate studies and adjusted to reasonable limits.

An additional consideration discussed in the USACE guidelines is the use of a buffer zone to characterize between 'tolerable' and 'acceptable' risk criterion. This buffer zone falls between 10^{-6} and 10^{-4}, and is known as the 'ALARP' zone, the 'As Low As Reasonably Practical' zone. This buffer zone represents the fact that the level of risk is very much a decision made based on several considerations such as cost effectiveness and societal concern to enter the decision of acceptability of a risk.

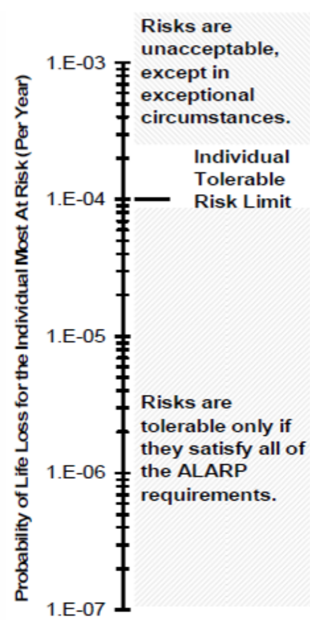


Figure 5-3 Proposed IR for levee systems as proposed by updated USACE guidelines for Dam and Levee safety.

The concept of voluntariness to the risk is not without consideration within the USACE discussion of IR. Specifically in the dam and levee guidelines, the concept of **informed consent** is discussed. This idea, that people choose to live in a risky areas, could justify higher limits of individual and societal life risk for those who decide to live in a flood plain. Following this reasoning, those exposed to the risk of flooding also receive a benefit from the project that is posing the risk. However while the idea is advocated by some, it is not taken into consideration in the updated guidelines.

Evaluation of Individual Risk of New Orleans Metro Bowl
To summarize the two approaches discussed, an acceptable IR value of 10^{-5} is determined from the approach of Vrijling (considering an involuntary activity with little benefit), while an acceptable value of 10^{-4} is assumed by the USACE guidelines. Below regions that exceed the IR value of 10^{-6} and 10^{-5} for the New Orleans metro bowl are plotted. These maps include the same assumptions as used previously, that 90% of the population has evacuated.

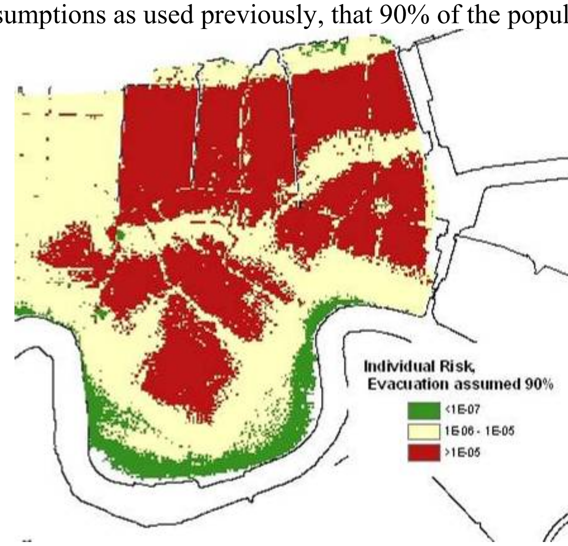


Figure 5-4 Area with an IR value higher than than 10^{-5} is seen in red and higher than 10^{-6} in yellow.

It is found that while the individual risk for the New Orleans metro bowl is relatively low in comparison with the probability of driving a car for example (10^{-4}), the majority of the metro bowl results in IR values higher than 10^{-6} , the acceptable level of individual risk proposed by Vrijling. Comparing this to the USACE tolerable risk guidelines for levee systems, the risk would fall within the ALARP zone, between the tolerable and acceptable risk limit.

5.2 Societal risk Criterion

To determine if societal risk is acceptable, the FN curve representing the societal risk is plotted against an acceptable 'limit line'. The FN curve should not exceed the limit line.

In general for societal risk, a lower level of tolerability is accepted than for individual risk. This is because society as a whole is more averse to large, multi- fatality events than it is from several individual loss accidents which may involve only a few people. Large loss events will initiate more of a social political response (Baecher, 2009), provoking government involvement or regulation. It may therefore be the case that the individual risk is acceptable, but that the societal risk is not.

When discussing societal risk criterion, the scale of the proposed criterion must be considered. It will be seen that criterion for both local and national scale events is proposed.

As the risk in this thesis quantified for a specified location, or installation, national criterion would not directly reflect acceptable risk for the installation. In actuality, to determine criterion for acceptable risk for the installation, the national scale criterion should be adapted according to the relative size of the installation at a national scale (Jonkman, 2007). This concept is not well examined in this paper and could be discussed in future work.

Criterion of Vrijling

The framework of Vrijling also proposes an approach to determining a societal risk in the context of flood protection .The basis for this approach was built upon the societal risk quantification approach of (VROM)(the Dutch ministry of housing, land use planning and environment), who quantified the contamination risk due to an LPG (liquefied propane gas) plant. The risk was evaluated against a proposed limit of 10^{-5} as an acceptable probability for the occurrence of 10 fatalities (This parameter can be described as the **base point** for the limit line.) This limit is described by the following equation:

$$1-F_{Ndj}(x) < 10^{-3}/x^2 \quad \text{(eq 5-8)}$$

Where F_{Ndj} is the cumulative distribution function of the number of deaths resulting in place j in one year for each activity. VROM's development of the equation is not clearly known, although it is implicitly understood that the basis for the criteria is the result of political process (Vrijling, 1995).

The criteria proposed by VROM was elaborated upon (Vrijling 1995), to be applicable for use at a **national** level for the Netherlands, (as the initial equation had been established for a singular **local** event). Again through the analysis of accident statistics, the threshold for a national, socially acceptable level is determined. Statistics show that death due to a non-voluntary activity (such as working in a factory) has a probability of 1.4×10^{-3}. For establishing a risk norm for the acceptable risk for engineering structures, a non voluntary activity is considered a sound basis for the probability of death. Upon this reasoning, and again using a β value for a non voluntary activity (β=0.1) the tolerable risk is limited by **$\beta * 1.4 \times 10^{-2}$.**

The elaboration of **VROM** yields:

$$1 - FN(n) < C_N/n^{\alpha} \quad \text{(eq 5-9)}$$

Where C_N is = $(\beta * 100/k)^2$, a constant that determines the vertical position of the limit line. The k is the risk aversion value and generally is set to 3, and again the policy factor b is found.

The exponent α determines the steepness of the limit curve. This is referred to as the risk aversion coefficient. In the applications of this approach, an α (and slope) of two is used, representing a risk averse perspective. Risk aversion is the notion that as the consequences increase, the probability of a given consequence should decrease more conservatively than linearly (an α of one is the linear relationship.) The level of risk aversion varies amongst approaches as there is not consensus regarding this parameter.

In order to apply this limit line to a single 'installation' the C value is reduced to represent the relative size of the impacted area/population at a national scale. However, this step is not examined here and thus the result of this equation will provide a value for a national scale for the Netherlands.

Therefore using a k value of 3, and α of 2 and a beta of .1 (involuntary activity with some benefit) a c value of 11 results.

USACE criterion
The USACE has recently begun to consider the use of societal risk that the use of the FN curve has been discussed in regard to flood protection. Recent guidelines for tolerable risk limits of dams and levee guidelines by the USACE are based upon past acceptable risk limits applied in other sectors.

The use an FN curve in risk applications was pioneered in the US by the USNCR, the US Nuclear Regulatory council for reactors. The cumulative probability distribution of man caused events was plotted for a reactor safety study which allowed the risks of nuclear power to be compared to other industrial risks and natural disasters (USNRC 1975).

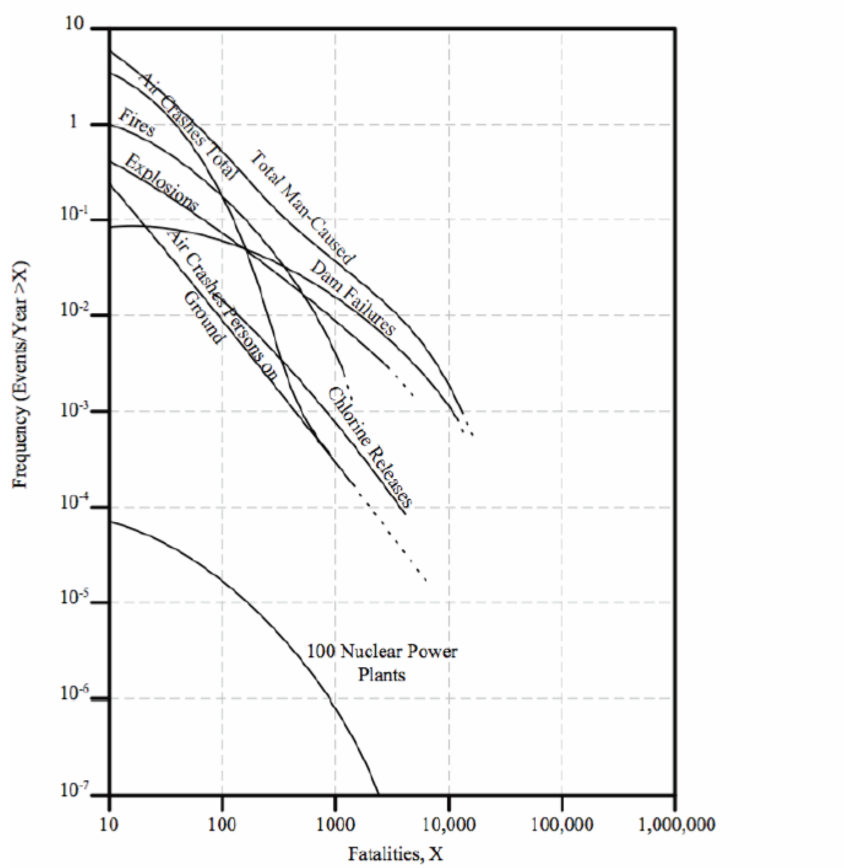


Figure 5-5 Frequency of man caused events involving fatalities (USNRC, 1975)

Probability consequence data was also produced for common risks in the field of civil infrastructure in a plot was published in a consulting report. The plot served as a useful tool for comparing risks.

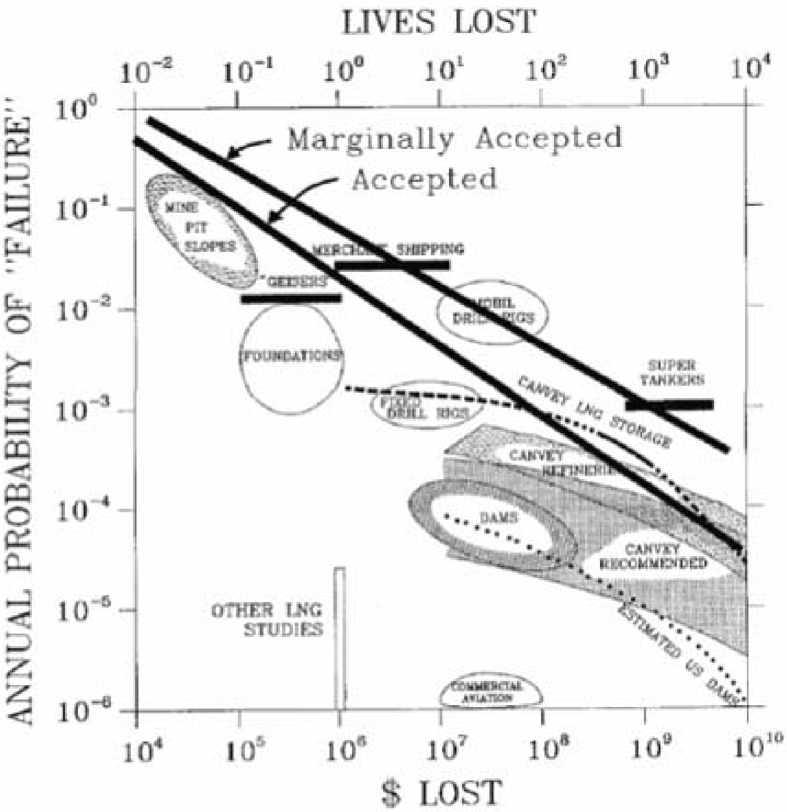


Figure 5-6 ' f-n' chart of common civil infrastructure risks based on approximate actuarial failure frequencies, with subjective interpretations of acceptability (T.W. Lambert and Associates, 1982).

(Note the above plot is not actually an FN curve, but rather a combination of F:N and f:N data in that it plots **failure frequencies**).

In 2003 The US bureau of reclamation (USBR, 2003) ranked safety upgrades to the dam systems by applying societal risk criteria. The base point for an acceptable limit line presented by the bureau is .001 lives/yr /load. This same base point is considered appropriate for the acceptable risk criteria for the levee safety and is proposed in the tolerable risk guidelines of the USACE and can be seen as well in proposed limit lines by Sielinski and Baecher (2008). The limits are once more characterized by the intermediate realm beween tolerable and acceptable risk, the so called ALARP zone.

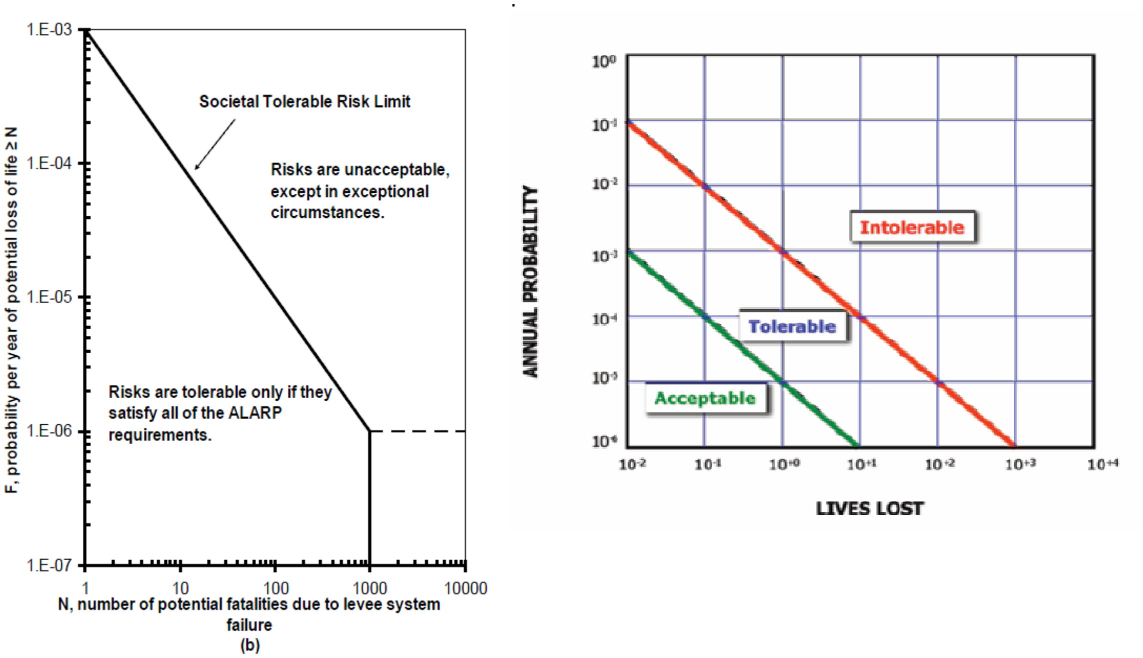


Figure 5-7 Tolerable and acceptable risk as found in the tolerable risk guidelines of the USACE (left). This tolerable risk limit is seen also in earlier work by Sielinski and Baecher 2008, (right).

Summary of societal risk criterion

Recall that in the approach prescribed by Vrijling, the vertical constant C_N (national scale) is represented using a β =.1, and thus a Cn value of 11. This is the tolerable risk limit on a national scale for the Netherlands. When comparing the approach of Vrijling to those of the USACE, the slope of the criteria line, the risk aversion coefficient α, varies. An α value of 2 reflects risk aversion toward large multiple fatality accidents, whereas a more liberal value of 1 places equal weight on exceedance probabilities and numbers of fatalities, resulting in a risk neutral approach. Discussion regarding this coefficient is ongoing. A summary of discussed parameters for the approaches is seen here.

Author and limit type	**Scale**	**C value (Vertical position)**	**α (steepness)**
Vrijling, Tolerable Risk	National (Netherlands)	11	2
VROM	Installation	10^{-3}	2
USACE, Tolerable Risk	National	10^{-3}	1
Lambert and Assoc., Acceptable Risk	National	10^{-1}	1

Table 5-8 Comparison of limit line parameters

Societal Risk of New Orleans Metro Bowl

The societal risk for the New Orleans metro bowl is plotted against the discussed criterion. it can be seen that the societal risk for New Orleans exceeds all of the proposed criterion. Most notably so when compared against criterion found in the updated USACE tolerable risk guidelines. (As the 'VROM' criteria is for a single installation it is not applied as a comparison criteria).

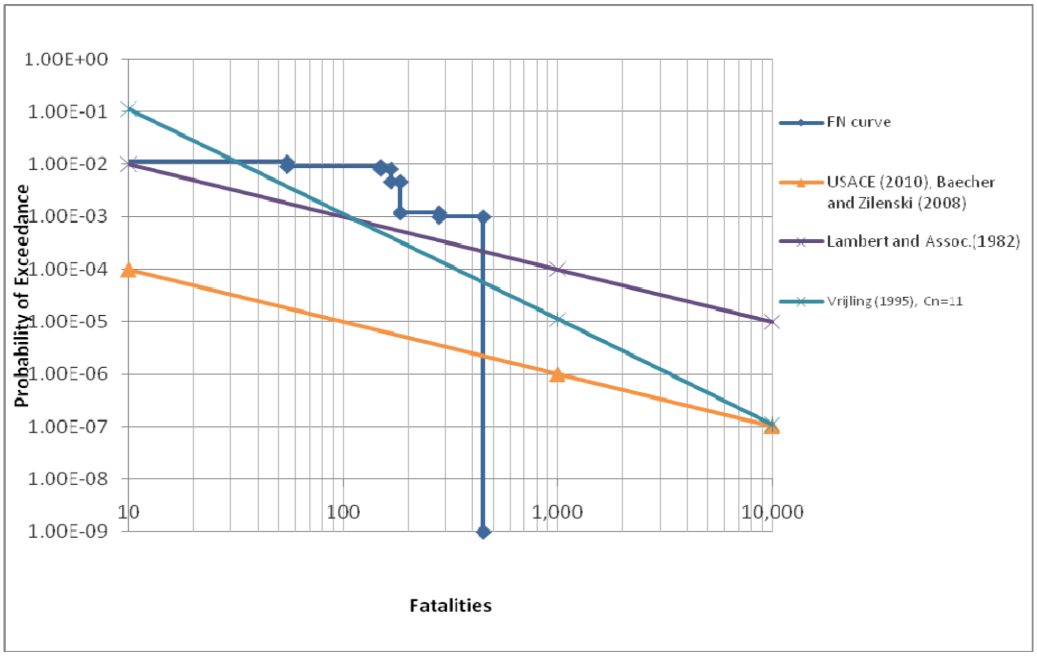


Figure 5-9 New Orleans Societal risk compared to limits proposed in literature.

Using the approach of Vrijling, a limit line that would correspond to the current risk for New Orleans equates roughly to a C value of 500:

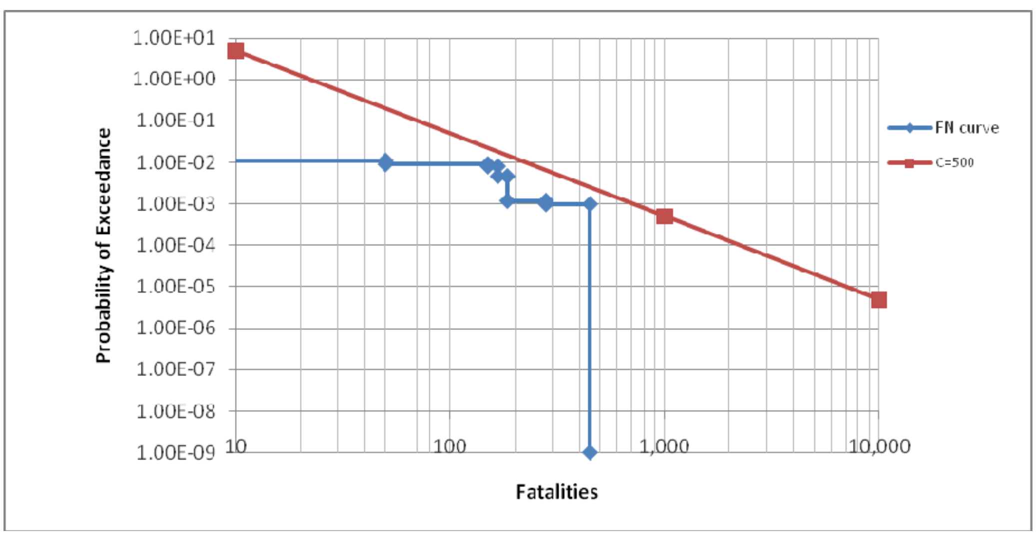


Figure 5-10 New Orleans Societal risk compared to limits proposed in literature (α of 2).

As a comparison, estimates for C values for below sea level protection rings in the Netherlands are depicted. This comparison can demonstrate the risk calculated for New Orleans could be compared to the c values in the Netherlands (Jonkman et al., 2011).

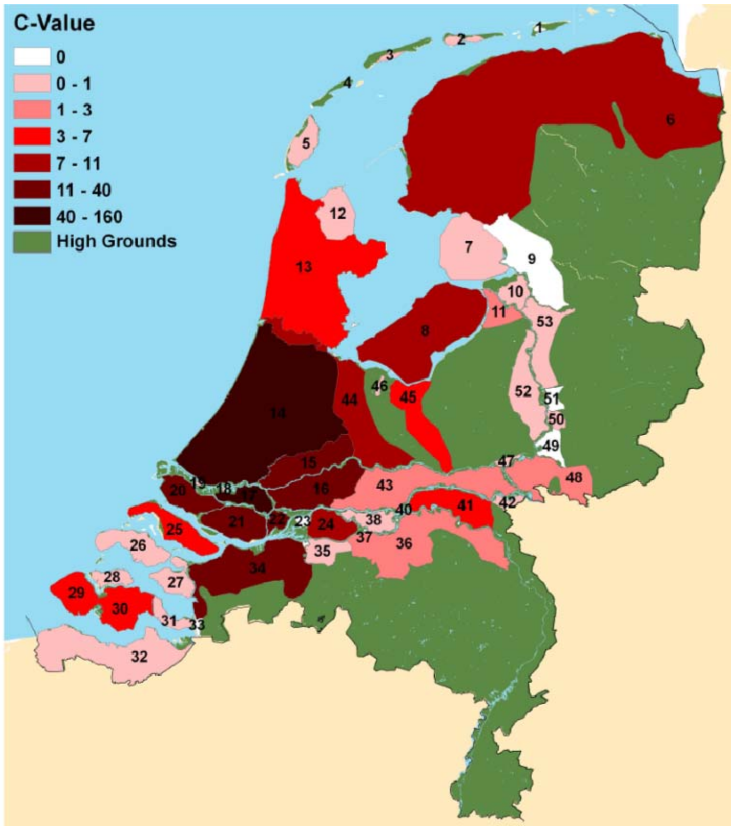


Figure 5-11 C values for dike rings in the Netherlands which indicate relative level of societal risk (Jonkman et al, 2011).

C values are found to be high for assessments of highly populated polders in the Netherlands (assuming a **low** evacuation percentage). Therefore societal risk for New Orleans, based off an assumed 'evacuated' population, is high when compared to societal risk in the Netherlands assuming little to no evacuation.

Comparison with societal risk of other sectors

Additionally, the use of the FN curve allows the quantitative comparison of societal risk with other sectors. Here the data from this study is plotted within the context of the Lambert and Associates consulting report results of 1982. Preliminary results of the IPET study, seen here in blue have also been plotted. While the risk approaches used by IPET vary from the approaches used here, results of this study could be considered comparable to preliminary IPET results.

Note that assumptions used in the IPET approach vary from those used in this thesis. A 'conservative assumption' with regard to evacuation is used (and therefore unknown to this author), and pre- Katrina population values used in fatality estimates. Also, the effects of river flooding have not been considered in their analysis.

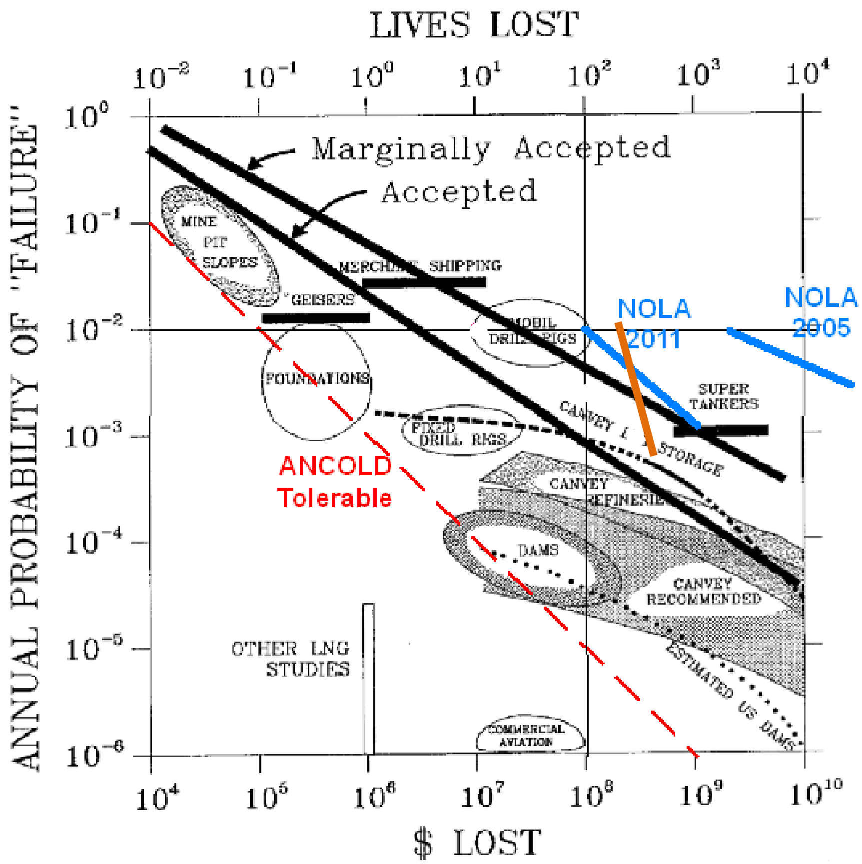


Figure 5-12 Results of this study plotted in orange. Data from IPET has been plotted in blue.

The IPET results have been plotted against the results of this thesis below as well as an estimated of the event of Katrina. In this way the risk reducing effect of the upgraded protection system can be seen

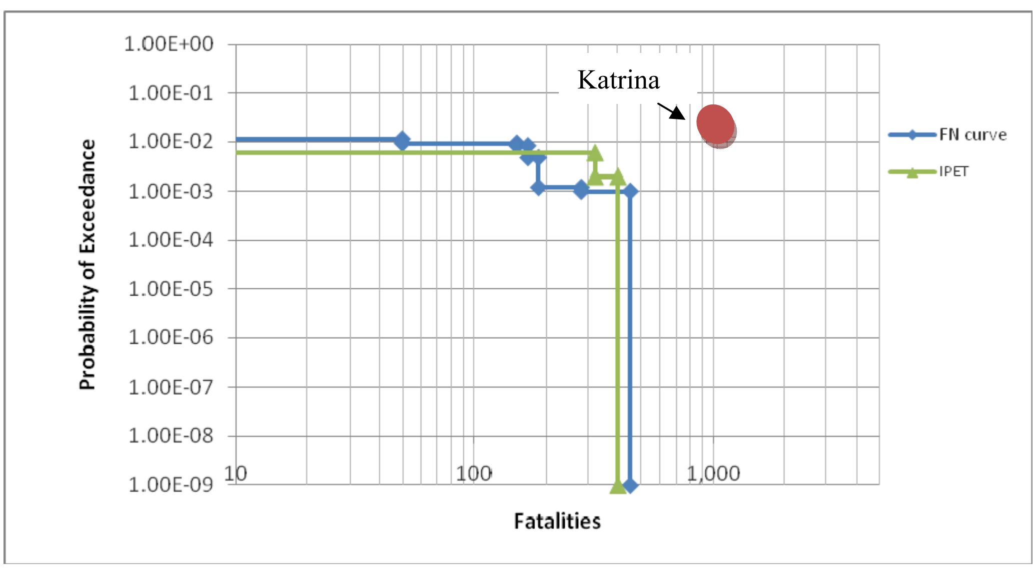


Figure 5-13 Results of this study plotted against resutls of IPET work. The event of Hurricane Katrina is plotted in red.

5.3 Conclusion

Therefore, two common life loss risk metrics have been expressed. These results have been evaluated through comparison against existing limits found in literature and government guidelines. Comparisons against such limits show the risk to life of New Orleans exceeds criterion proposed in literature and industry.

Using the above criterion discussed, a risk assessment can be carried out upon which to decide acceptable protection levels. The final step in the risk assessment will touch on the application of a risk based approach can be used to quantify and evaluated the effectiveness of risk reduction measures.

6 Risk Reduction

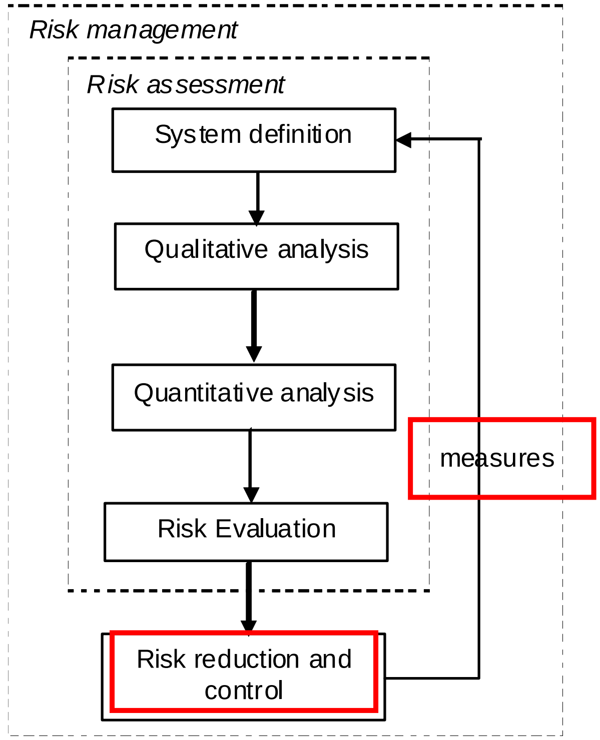


Figure 6-1 Steps of a Risk Assessment (Jonkman, 2007)

The combined risk of flooding from hurricane surge and river discharge for New Orleans exceeded several discussed acceptable risk criterion. The evaluation of risk can be applied to determine the effects of various risk reduction strategies. Having a quantified risk estimate allows the reductive effects to be analyzed in a systematic and consistent way.

6.1 Risk reduction measures

As the risk is a function of the probability and the consequences, reduction measures can be categorized into measures that reduce event **probability** and measures that reduce the **consequences.** Effects of risk reducing measures can be depicted with various metrics such as and FN curve or individual risk map. Such metrics serve as tools to visualize the effects of measures and facilitate the evaluation of different risk management strategies.

It was seen that the risk measures FN curve and maps depict the effects of the reduction measures. The reduction of probability shifted the FN curve down to meet a proposed criterion. Measures that reduce consequences of flooding will cause the FN curve to move to the left.

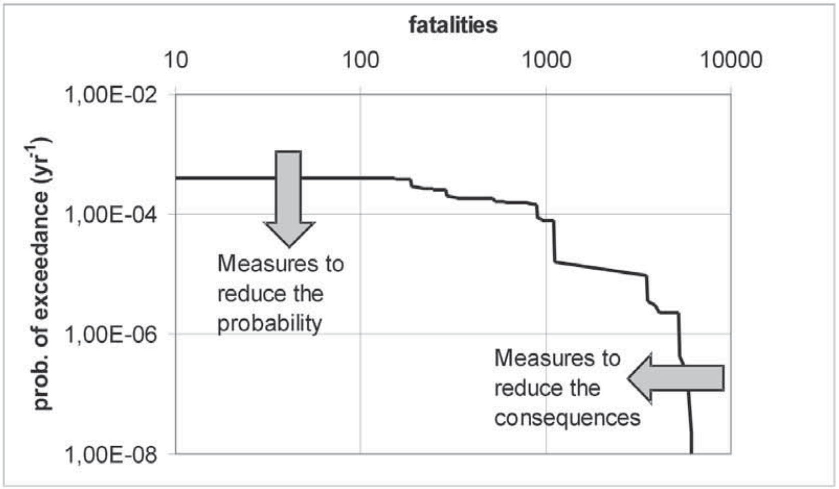


Figure 6-2 FN curve indicating the effects of two types of measures (Jonkman et al., 2008)

6.1.1 Probability reducing measures: Protection level

One probability reducing measures is to increase the protection level, in general accomplished by increasing the elevation or strength of the protection.

Thus the risk estimation provides a basis for the design of protection, as a protection level can be calculated to meet the discussed acceptable risk criterion. The effects of increased protection levels can be depicted in the various risk measures discussed. For example, analysis through the use of societal risk will show an increase in protection level will reduce event probabilities, shifting the FN curve down so that limit criterion may be met.

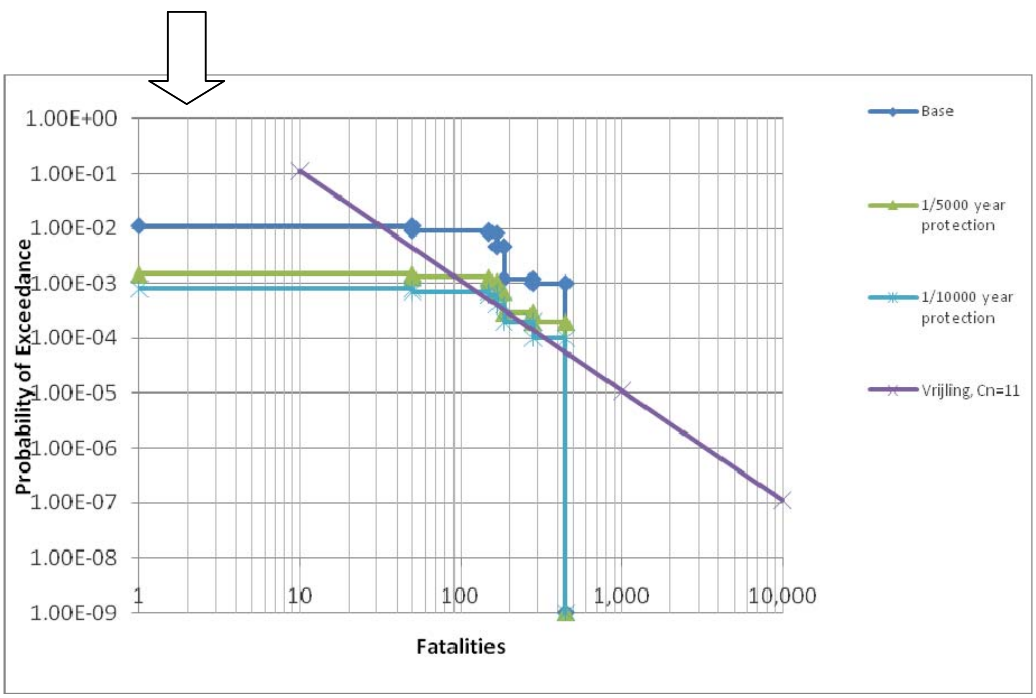


Figure 6-3 An appropirate protection level can be proposed to meet limit lines.

The effects of increased protection levels show that a level of protection of 1/10,000 year would fail to meet the national risk limit line as proposed by Vrijling (This line is a liberal criteria in this context).

Using the IR maps, a level of protection that meets IR acceptable criteria is estimated. By calculating the maximum individual risk and comparing it with proposed acceptable criterion, allowing levels of flood protection that would provide a certain IR. Below the effects on the individual risk due to increased protection levels are depicted:

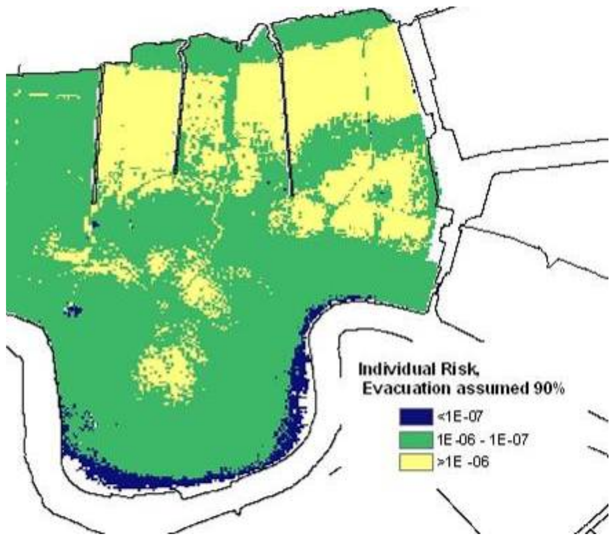


Figure 6-4 Map of IR, 90% evacuation, with effect of increased protection (1/5000) yr.

The IR map above illustrates the risk associated with a 1/5000 year protection level. The risk is reduced such that no areas exceed the 10^{-5} threshold (areas that exceed 10^{-6} are in yellow).

The inclusion of protection levels based on other criterion, such as economic considerations can be useful for comparison. By balancing the investment in flood protection with the risk reduction benefits provided by the protection, an economically optimized protection level can be determined by. The optimal level of protection is the point at which the total costs in the system are minimized. As mentioned in chapter one, such an approach was used to make decisions regarding flood protection in the Netherlands after flood events in 1953. For the city of New Orleans, a similar analysis using this optimization was carried out. Results showed that protection in the order of a 1/1000 to a 1/5000 year level could be economically justified (Jonkman et al., 2008).

Protection levels based on varying risk criterion show the following appropriate levels of flood protection:

Criterion	Resulting Pf`
Individual risk <10-5	1/4000
Individual risk <10-6	1/10,000
Nationally Acceptable Risk, Vrijling	1/25,000
FN criterion, USACE	<1/50,000
FN 1982 Plot	1/10,000
Economic optimization	1/1000-1/5000

Table 6-5 Protection levels that would meet proposed risk criterion.

Thus a risk based 'protection level' could be designed based on acceptable economic and life loss criterion. It can be seen that the increase of protection may not be practical. Also it may be less cost effective, or not preferred. In this case, other risk reduction techniques must be looked at for the management of risk.

6.1.2 Probability reducing measures: Load reduction

Further measures to consider that would reduce the failure probability include measures that **reduce the load** on the system. Such measures can include:

- The development of barrier islands.
- Wetland restoration.
- Diversion of Mississippi river via other routes (ie by means of the Atchafalaya basin)
- Implementation of multiple lines of defence , consisting of coastal habitat restoration and engineered flood protection to impede storm surge (Lopez, 2007)
- Barrier at river mouth or spillways along river to reduce surge up the river.

Quantifying the effects of load reducing measures can be difficult. Extensive time frames associated with implementation of such measures make their effects difficult to quantify. Such discussion is outside this research scope; however it is anticipated such measures play an important role in the future of flood protection for the city and discussion and analysis of such measures must be carried out in the future.

6.1.3 Consequence reducing measures

Secondly, measures which minimize event consequences are considered. Recall that this study quantified consequences, the number of fatalities (N) with the equation:

$N= F_D\ (1-F_E)*N_{PAR}$

Where:
F_D - mortality rate
F_E - fraction of the population evacuated
N_{PAR} - total population at risk
As such, reduction measures for each of these terms are discussed.

Reduction of the exposed population, F_E
Reduction of the population exposed to the flood waters can be accomplished by improving evacuation. There are various ways of encouraging evacuation such as increasing inventive to evacuate with regulatory measures or by furthering government assistance in evacuation, such as providing bus services. An important component to this is a well prepared plan for timely evacuation and well communicated evacuation strategies.

To quantify the effect of a reduced population on the consequences, the value entered fro exposed is adjusted.

Reduction of the population 'at risk', N_{PAR}
Limiting the number of people allowed to live behind a certain level of flood protection reduces N_{PAR}. Governmental zoning laws could limit the population behind the 1/100 year protection. It is estimated 3% of citizens in America currently live behind 1/100 year protection (Crowell et al, 2010). On a more local scale in the New Orleans metro bowl, residential zones in higher risk areas could be discouraged through zoning measures. Limits

on residential building below sea level would have similar effects. Residing in high risk areas can be discouraged through the use of insurance practices correlating higher premiums to high risk areas (this is done to some extent already).

To quantify the effect of the relocation population to reduce consequences, a new initial population per tract was used in the calculation. The initial population located within tracts having and individual risk value above 10^{-5} was redistributed amongst remaining tracts. The overall initial population was unchanged. However, as most of the scenarios result in the flooding of the majority of the bowl, the reductive effect is minimized.

Reduction of mortality rate, F_D

A final consideration in the reduction of fatalities, is the reduction of the mortality rate. In this study, the loss of life is a function of water depth. By adapting or **raising homes** and buildings to a specified elevation, the (effective) water depth is decreased, and mortality reduced. In the same way, if homes were required to install an accessible **third story**, so that all homes have a floor above the maximum possible water depth, the mortality may be further reduced.

The effect of elevating homes is analyzed to reduce the consequences. To do so, a 5 foot (1.52 m) elevation of homes is represented by reducing the averaged effective water depth of the census tracts by 5 feet (1.52 m) for every scenario. The original mortality function is then reapplied to result in an updated fatality estimate.

It is noted that this may result in an over estimation of the reduction of fatalities. The assumption is that because their homes are raised, that people are no longer exposed to the flood effects. However this may not always be the case. People may not be in their homes during the flooding or may lose their life due to other causes such as dehydration.

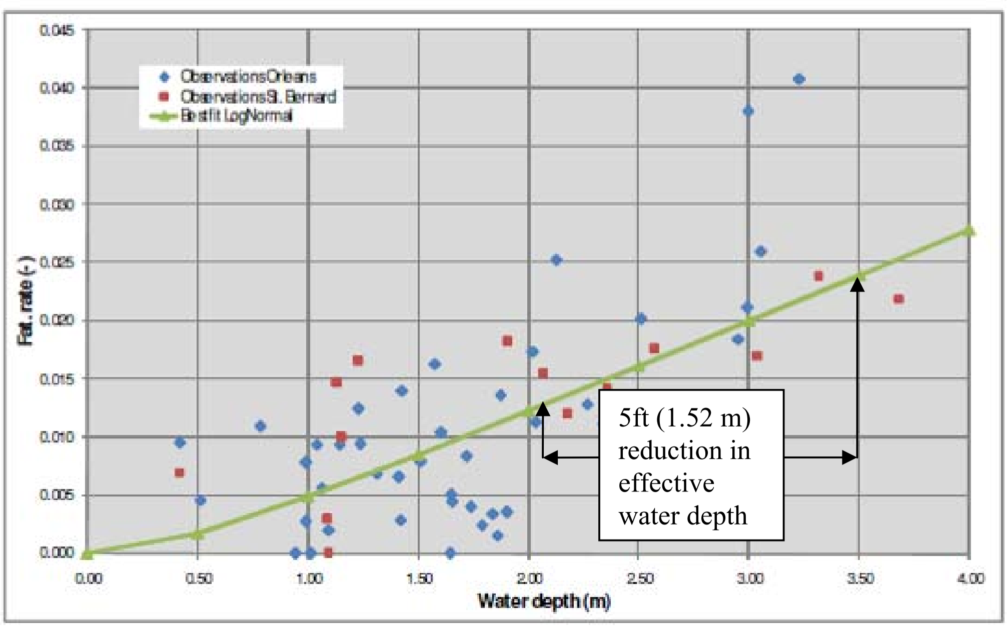


Figure 6-6 Relationship between water depth and mortality rate for New Orleans (Maaskaant, 2007). The assumed reduction in depth leads to a lower mortality value.

The reductive effects of the various measures are quantified using the FN curve below.

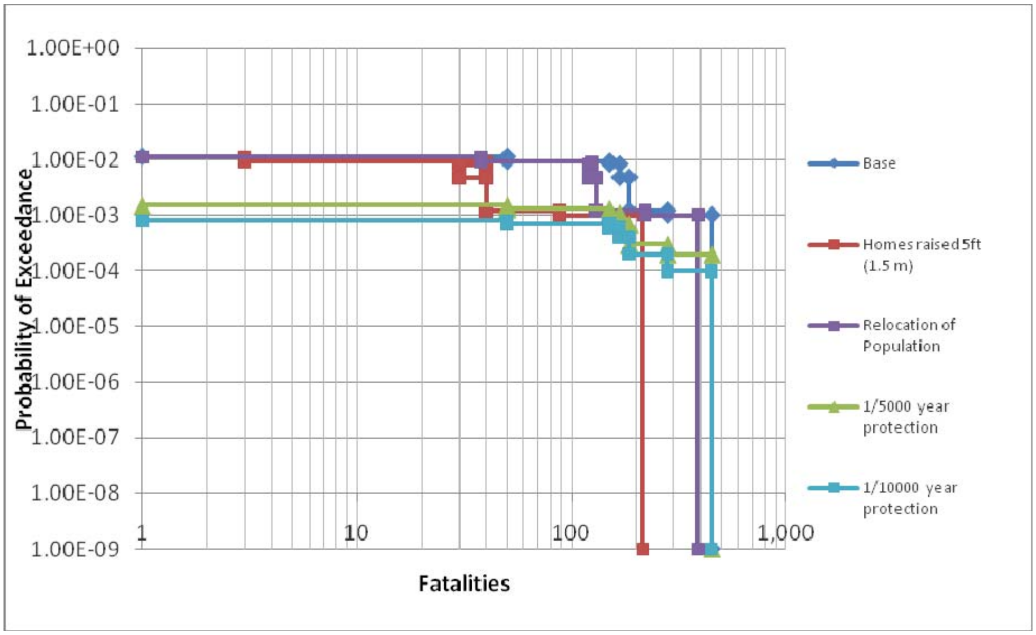


Figure 6-7 Effect of various strategies to reduce the risk.

The effects of other consequence reducing measures, such as evacuation, were discussed in section 5.2. It is seen this has a larger effect as it reduces the initial population. As mentioned, the reduction effect of the relocation of population is minimal due to the large flood extents of the scenarios modelled. The elevation of homes is seen to have more impact on the reduction of societal risk as the water depth from which fatality estimates are determined, has been significantly decreased.

6.2 Cost comparison of reduction measures

While the effects of various measures can be depicted using such risk measures to meet various criteria, the cost effectiveness of the various measures is an important consideration in decision making. Evaluating the risk also allows the cost effectiveness of measures to be quantified.

CSX values

As the effects of the measures have been determined, the costs associated with the protection can be related to the measures reductive effects. Expressing what is known as the 'cost of saving an extra statically life', or CSX, accomplishes this. The CSX value relates the investment in the reduction measure to the reduction of the expected fatalities and expressed in a unit of $\$/\Delta E(n)$.

$$CSX = I/\Delta\ E(N) \qquad \text{(eq 6.1)}$$

Where:
CSX is the cost of saving an extra statistical life per year (\$/Fat/Yr)
I = investment (\$)
E(N)= expected number of fatalities (fat/yr)

The lower the ratio between the investment and the saved lives, the more 'cost efficient' the measure is. This approach can be a cost benefit analysis whilst refraining from assigning an amount of money to a life.

CSX values are determined for individual reduction measures and the combination of measures. The effect of these measures to reduce solely the risk to life is analyzed here. Some measures, such as evacuation and relocation, do nothing to prevent risk to economy and culture. Therefore the conclusions drawn regarding reduction measures are solely ranked in the context of life loss analysis. Also to note is that measures that reduce consequences, such as relocation or residents, reduce risk locally, whereas probability reducing measures decrease all dimensions of risk in all locations.

Cost assumptions
The estimated cost for the proposed measures must be determined. Rough and indicative cost estimates are made. The following assumptions led to these estimates.

- Estimation for the investment in hurricane protection has been made based on contracts costs estimates provided by the USACE (Bos, 2007). In these contracts, the estimate to raise the New Orleans east polder to a level of 1/500 is roughly 3.3 $x10^8$. This cost is applied to the New Orleans metro bowl ring to result in the overall cost to raise protection.
- For the elevation of homes, a cost of \$50,000 -\$100,000 per home is estimated. According to Census data, the Orleans parish has roughly 114,000 homes. As the population of exposed persons considered in this study is 65% that of the Orleans parish, an estimate of 80,000 homes are located in the metro bowl. When elevation of the metro bowl are examined, it is determined that 90% of the metro bowl lies below 5ft, thus 90% of the homes would be raised. A cost estimate of 4 billion dollars is used.

Figure 6-8 Area of Orleans metro bowl with an elevation below 5 ft (1.25m) is seen in red. Areas above sea level are in pink.

- For a measure that would relocation people to higher ground, 50% of the population is an assumption of 50% of the population are relocated to higher ground. According to estimates by Hoss, a cost of \$50,000 per home is assumed for this cost.
- It is assumed evacuation costs are minimal and would consist of provision of bussing services.

The cost estimate is divided by the change in E(N), as was calculated earlier for the FN curve. The reduction in E(n) for each measure, found by subtracting the updated expected value from the original estimate of 2fat/yr, is seen in the far left column.

Measure	Description	Cost $ (10^6)	Source	Resulting E(n)	% E(n) Reduced
Elevate homes	5ft water depth reduction (1.52 m)	4000	Estimate	.52	73%
Increased Protection	1/500 year level of protection (for Hurricane protection only)	2000	Bos (2007)	1.03	48%
Evacuation	95% of population evacuated	10	Jonkman (2007)	.01	98%
Relocate population	100% of persons in high risk zone relocated (roughly 50,000 persons)	2000	Hoss (2010)	.52	73%

Table 6-9 Table of cost and Reduction in E(n) values for various risk reduction measures. In this table be suregt o include original expected value

The resulting CSX values are then calculated by dividing the cost by the change in expected value.

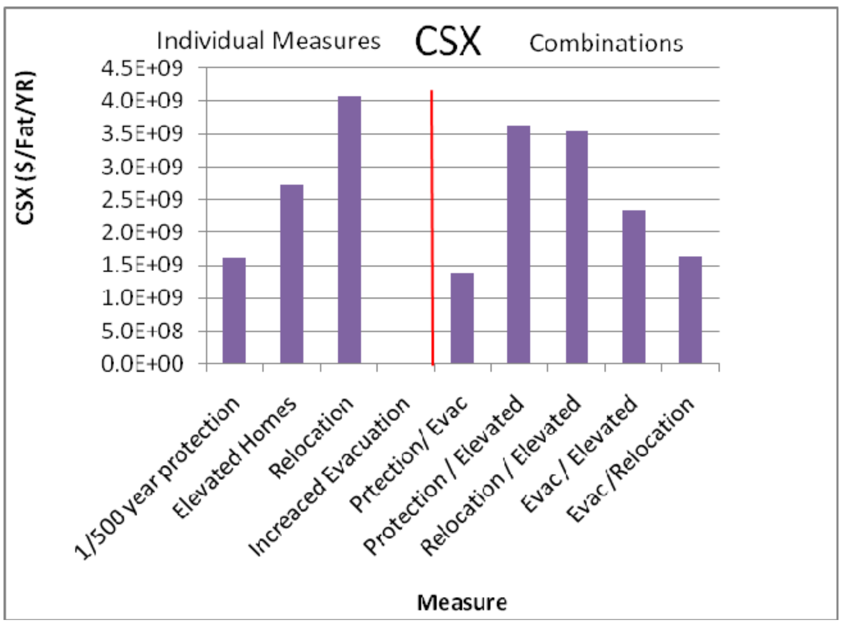


Figure 6-10 CSX values estimated for both the effects of individual measures and the combination of measures.

Discussion of CSX Results

In the analysis of CSX values, recall that the lower the CSX value, the more cost effective the measure is. It is seen with regard to individual measures, increasing evacuation is the most cost effective as it limits the exposed population at a relatively low cost. Comparing the remaining alternatives discussed, a 1/500 year protection level is most cost effective. This measure is less expensive than elevating homes and more effective than the relocation of people. The elevation of homes can be seen to be relatively cost effective as it reduces the depth of flooding having a significant reduction on the mortality rate.

The effects of a combination of reductive measures can also be analyzed. When combining measures, the reductive effects of measures may not linearly add up due to a correlation in the reductive impacts between measures (Hoss, 2010) It is seen that the combined measure of increased evacuation combined with the relocation of population in high risk areas or elevation of homes is the most cost effective. Combining protection and elevation of homes are both expensive alternatives with risk reducing effects that have diminishing effects when combined.

6.3 Conclusion

An important application of a risk evaluation is to provide a basis for decision making in the context of risk reduction measures. The comparison of the risk to various criteria allows an estimation of protection levels that would meet discussed criterion.

In the case that increased protection is not practicable or too expensive, the application of alternatives can be investigated. Risk measures such as FN curves and IR maps depict the effectiveness of proposed reduction measures such as relocation of population and the elevation of homes. To evaluate the cost effectiveness of measures, CSX values can be expressed, which show that evacuation is most cost effective, but that an increase in protection is still relatively cost effective compared to other examined measures. The next most effective of the alternatives discussed is the elevation of homes, which while costly, significantly reduced the mortality rates. In the future, more detailed and realistic numbers for investment costs and risk reduction can be used to assess the effects of various measures.

7 Conclusions and recommendations

In this thesis, an evaluation of the risk to life for the city of New Orleans has been carried out. To quantify the risk to life, an approach which considers potential consequences through the simulation of possible flood scenarios has been applied. The risk is quantified in several metrics, namely expected value, societal risk and individual risk. The individual risk is plotted spatially as it is a characteristic of location. The IR represented for an evacuation rate of 90% of the initial population, results in the majority of the study area, the New Orleans metro bowl to have an Individual risk level over 10^{-5}.

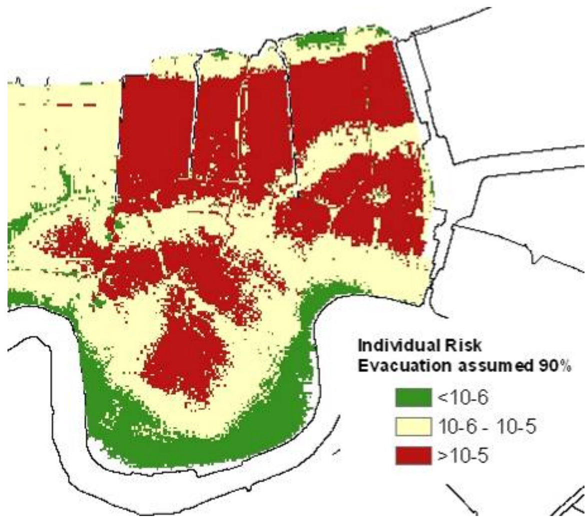


Figure 7-1 IR value results for New Orleans Metro bowl.

Societal risk is depicted as an FN curve, and plots the probability of exceedance of flooding of the area studied against estimated fatalities. The resulting FN curve of this study is seen below. It can be seen the river event scenario, the worst modelled scenario, leads to 450 fatalities and has an estimated flooding probability of 1/1000 per year. The other flood scenarios with smaller numbers of fatalities, ranging from 50 to 180, have a probability of exceedance of 1/100 to 1/1000 per year.

The area under the curve is also equal to the expected number of fatalities, a simple measure of societal risk. Or the average number of fatalities year, the result of which is 2 fat/year. Also plotted on the FN curve is a representation of the risk level of the hurricane Katrina event. In this way, the risk reduction effect of the upgraded protection is depicted.

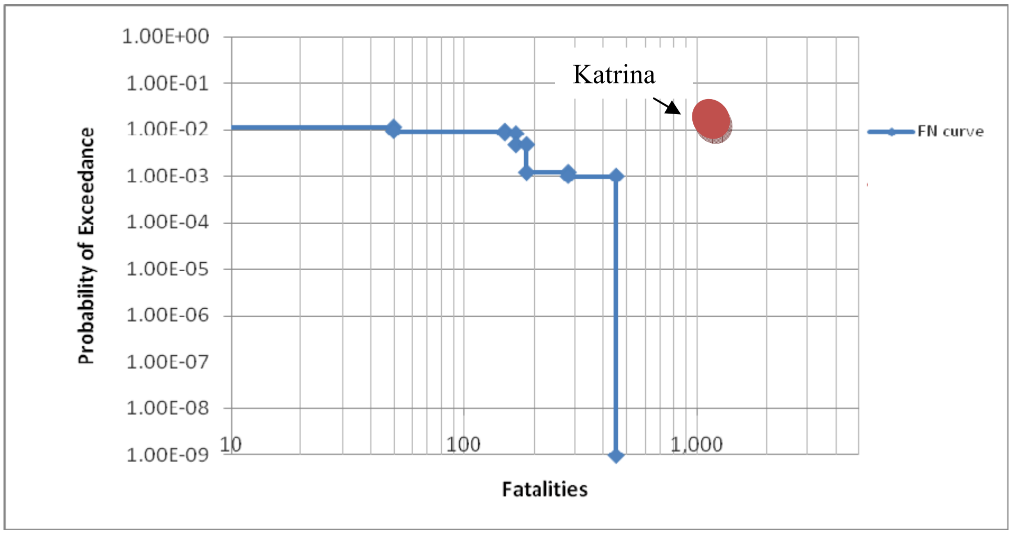


Figure 7-1 FN curve for New Orleans Metro Bowl in 2011, after improvements

The societal and individual risk dimensions have been evaluated based on various acceptable risk criteria found in literature and existing applications. Risk reduction measures have been discussed and their effects quantified through the use of the risk quantification measures. As an indicative analysis of the effectiveness of measures, the CSX, the cost of saving an extra statistical life, is presented. The cost of elevating the defence system against the reduction of the probability of flooding is compared, as well as other reductive measures. Based on this analysis, the following conclusions are made:

7.1 Overall Conclusions

- An evaluation of the risk to life of the New Orleans metro bowl indicates that the risk, while reduced from the pre-Katrina status, is still significant. And while there is no consensus on an acceptable risk to life in literature, the risk to life for New Orleans, both individual and societal, exceeds proposed acceptable risk limits.
- The quantified effects of mitigation strategies in the situation of New Orleans demonstrate that evacuation is the most cost effective strategy and should be encouraged. An increase in protection level is the next most cost effective followed by elevation of homes.
- The results of the study indicate the necessity of further societal discussion on the acceptable level of flood risk. The decision then must be made whether the current risks are acceptable or whether additional risk reduction measures necessary.
- The quantification of risk to the city of New Orleans has several applications in the context of safety and flood protection such as providing a basis for determining protection levels and describing the current level of safety of the residents.

7.2 Recommendations

Recommendations on simplified study

- Presented here have been results based on the elaboration of a limited number of flood scenarios. For a more complete evaluation, it is recommended to include a more complete set of scenarios to better represent possible load situations, breach combinations and flood conditions.
- The approach has considered solely a portion of the city, identified here as the metro bowl. The approach should also be applied to remaining metropolitan areas of New Orleans to get a more complete assessment of the risk.

- The western boundary of the analyzed area is assumed to have a small probability of flooding and therefore it is not taken into consideration. This assumption should be investigated further in future work.
- The approach to reduction measures can be carried out in greater detail. In the future risk, reduction analysis could include measures to reduce the load, such as the quantification of the effects of wetland restoration and barrier islands. Implementation of multiple lines of defence, combining coastal habitat restoration and engineered flood protection (Lopez, 2007) can impede storm surge, while the diversion of the river down the Atchafalaya basin could be discussed to reduce river load.
- A complete reliability analysis should be carried out to determine the event probabilities which have been estimated in this thesis. The reliability analysis as done by IPET could be a contributor to such a study.

General recommendations

- This study has considered risk to life. The inclusion of economic risk should be included to provide a more complete picture of the risk of the city and to be included in discussion of flood management strategies.
- The use of risk based analyses can be applied to describe the level of safety of the inhabitants, and propose appropriate protection levels. Evaluation of risk to other highly populated urban areas should consider the use of similar approaches to assist in flood management decisions.
- Currently, maps developed by FEMA used for the national flood insurance program show areas that are within the 1/100 year level. In the event IR maps are not easily developed, (due to lack of consequence data for example), maps can be extended to show the probability of selected water levels. These can be distributed to the public and used to increase risk communication.
- In the same way that a higher level of protection may be warranted for areas of dense population and high economic value, similar approaches can be used to evaluate protection measures for less populated regions with minimal economic value. In general, if improvements to protection are not cost effective or necessary to fulfil acceptable risk criterion, a higher probability of flooding should be accepted.
- Finally, as the risk to life for New Orleans exceeds proposed acceptable risk limits, further discussion on the acceptability of the current level of protection for New Orleans is recommended.

Therefore, the risk to life for New Orleans is still significant and predicted to increase in the future. Taking into account the risk based considerations for flood protection decisions provides valuable technical information on current and acceptable risk levels. Risk informed decision making will be important for the future of the city.

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Glossary:

Boundary: Hydraulic condition in the supplying water body at the border of the analyze system. Generally will be associated with a certain probability.

Correlation: The degree of influence that one variable has on another.

CSX: Cost of saving an extra statistical life. Computational measure to evaluate cost effectiveness of reduction measures.

Deterministic model: every set of variable states is uniquely determined by parameters in the model and by sets of previous states of these variables. Therefore, deterministic models perform the same way for a given set of initial conditions.

Expected value: (or expectation, or mathematical expectation, or mean, or the first moment) of a random variable is the weighted average of all possible values that this random variable can take on.

Exposed population, N_{EXP}: Number of people affected by flooding

Failure mechanisms: Mode of failure. Protection elements fail due to various mechanisms, which are often correlated.

Fatality: Lost human life as direct consequence of flooding. (vs exposed)

Fault tree: Model to analyse events or conditions that lead to and unwanted event (here inundation).

Flood characteristics : circumstances which quantify the flooding . Output of characteristics include velocity,(v) depth, rate of rise , time of arrival and others.

FN Curve: Exceedance frequency of number of fatalities plotted on double log scale.

Independent variable: If the value taken on by one variable has no influence upon the value assumed by another variable. Independence of random variables simplifies the representation and analysis of uncertainty.

Individual risk: probability of death of an average unprotected person constantly present at a certain location. (property of location).

Informed consent: The idea that people choose to live in a risky areas, thus they are aware of conditions and risk level.

Investment (I): Cost of risk reduction measures (direct material costs).

Mortality function: relationship between flood characteristics and life loss.

Mortality: Relationship between flood characteristics and fatalities

MSL (Mean sea Level) : the zero elevation for a local area.

Parish: county within Louisiana

Polder: Ring of protection

Probabilistic: model where variable states are not described by unique values, but rather by probability distributions. Stochastic variables, therefore randomness is present.

Probability Distribution: A probability density function (PDF) describes the *relative likelihood* that a random variable will assume a particular value in contrast to taking on other values. Area under a PDF is always unity.

Random Variable: Variable where the magnitude of variable is not exactly fixed, but rather may assume any of a number of values.

Reliability: Conditional probability of component or system performing intended function.

Resiliency: Ability of the defence to withstand the design load

Risk: Product of probability and consequence of event.

Risk evaluation: This involves a set of concepts and procedures, such as fault trees and event trees, for studying structures or facilities with many components and different modes of failure.

Risk measures: an expression or graph which quantifies or depicts risk as a mathematical function of the probabilities and consequences of a set of undesired events.

Safety factor: Ratio of allowable value of some quantity to the calculated /measures value of the quantity. (ratio of absolute element strength to applied load)

Schematization: simplification of complex system

Serial System: a system in which the failure of one component causes failure of the remainder of components.

Societal Risk: Risk dimension quantifying describing the probability of a large, multi fatality event.

Stochastic Variables: variables without a fixed value, but which can assume any number of values

System: overall network of components combining to provide an entire ring of protection.

Transitory zone: zone of the river where water levels are affected by riverine and marine effects.

9 Appendix

9.1 Flood maps, maximum depth and time of arrival

Maps are listed in the following order:

Scenario	Location (and Mechanism)
1	River breach due to high river
2	River breach due to storm surge
3	Lake breach at West End
4	Lake breach at St. John
5	IHNC breach
6	Multiple breach due to storm surge

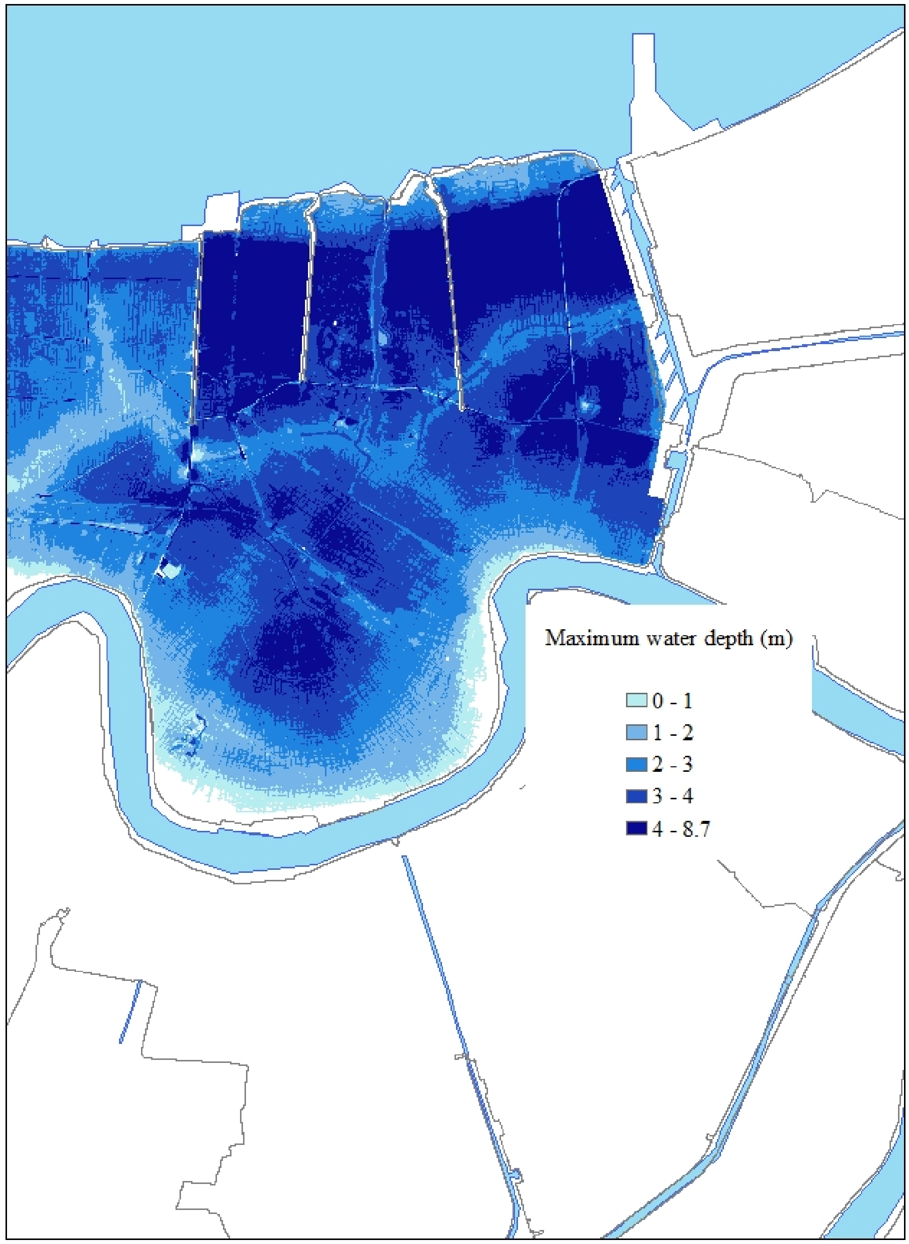

Figure 9-1 River breach due to high river, maximum water depth (m)

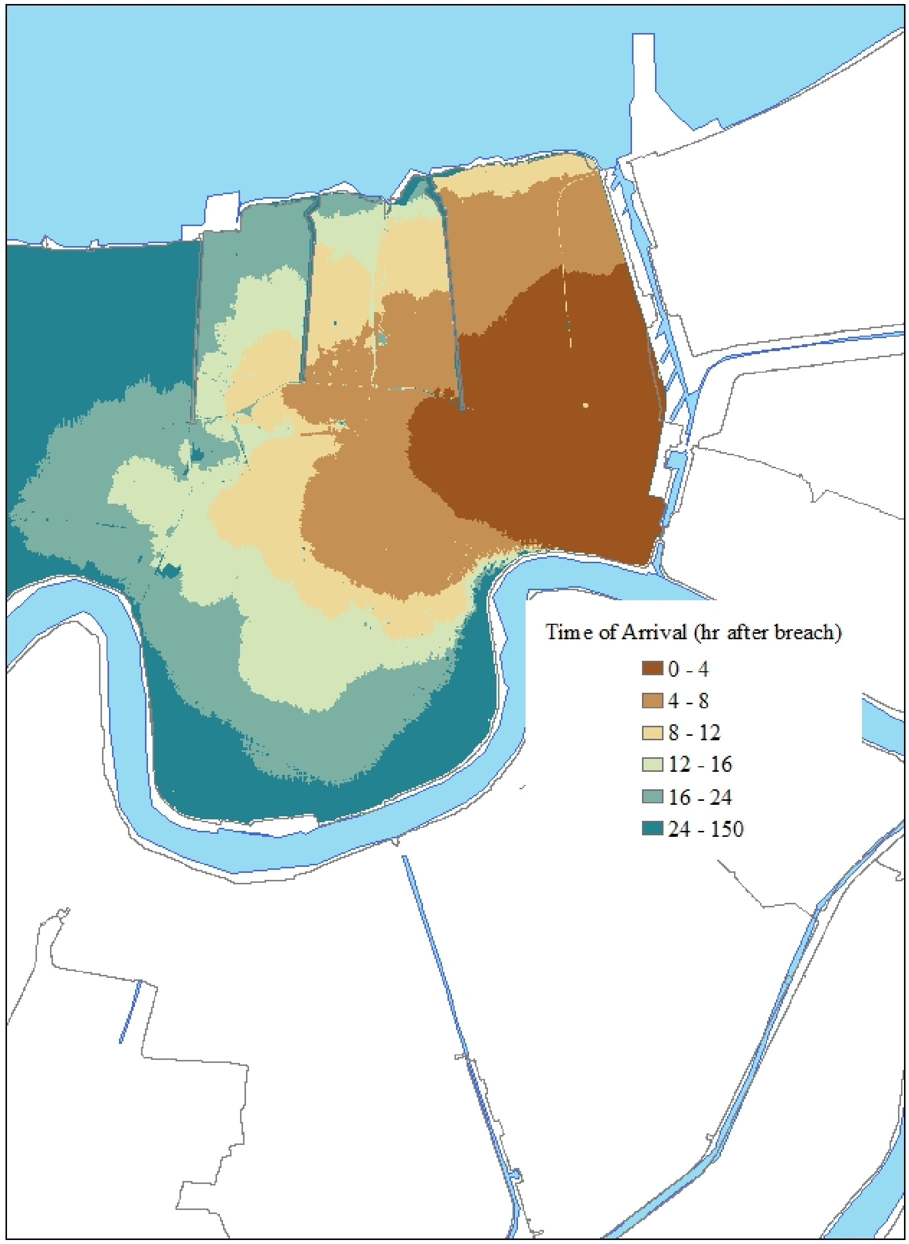

Figure 9-2 River breach due to high river, time of arrival (hr)

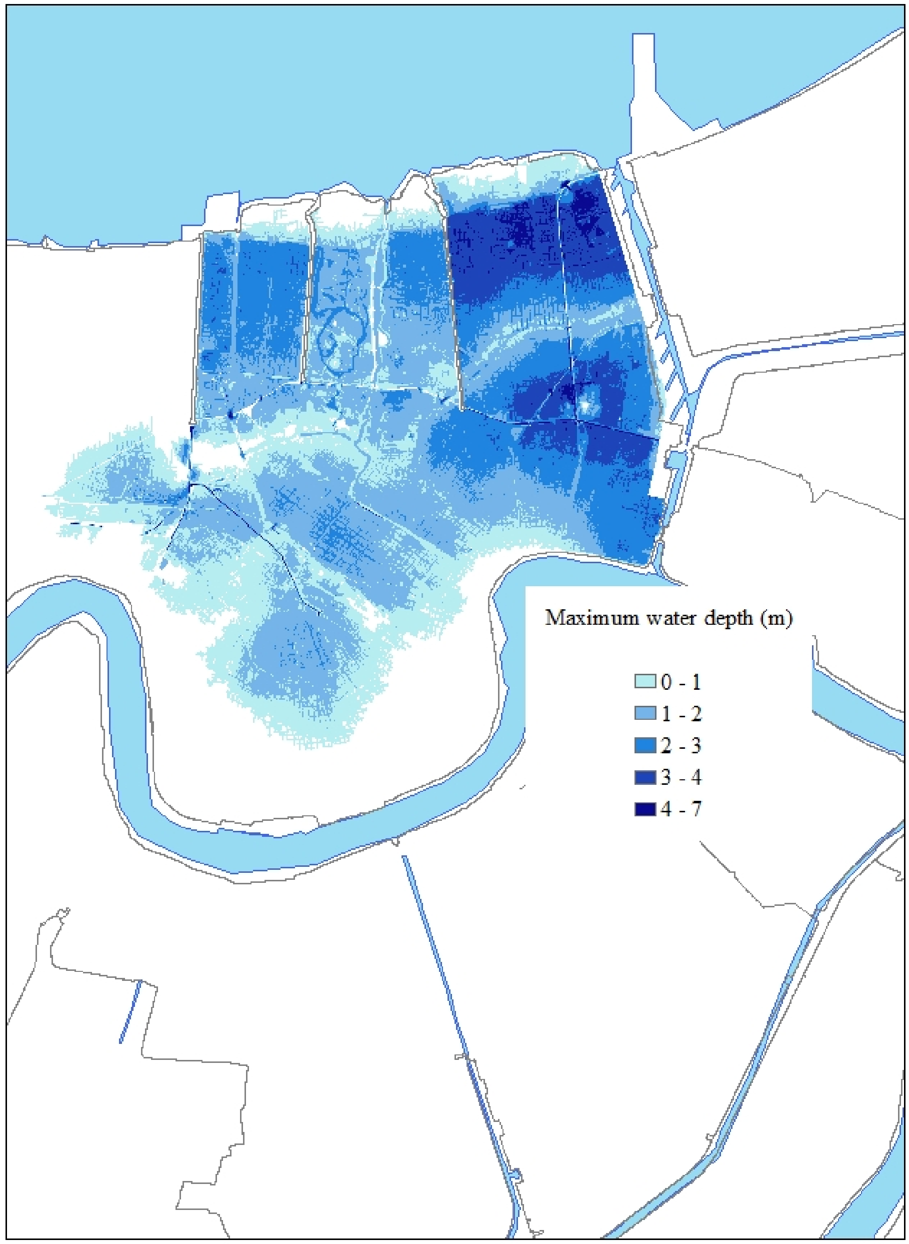

Figure 9-3 Breach due to high river surge, maximum water depth (m)

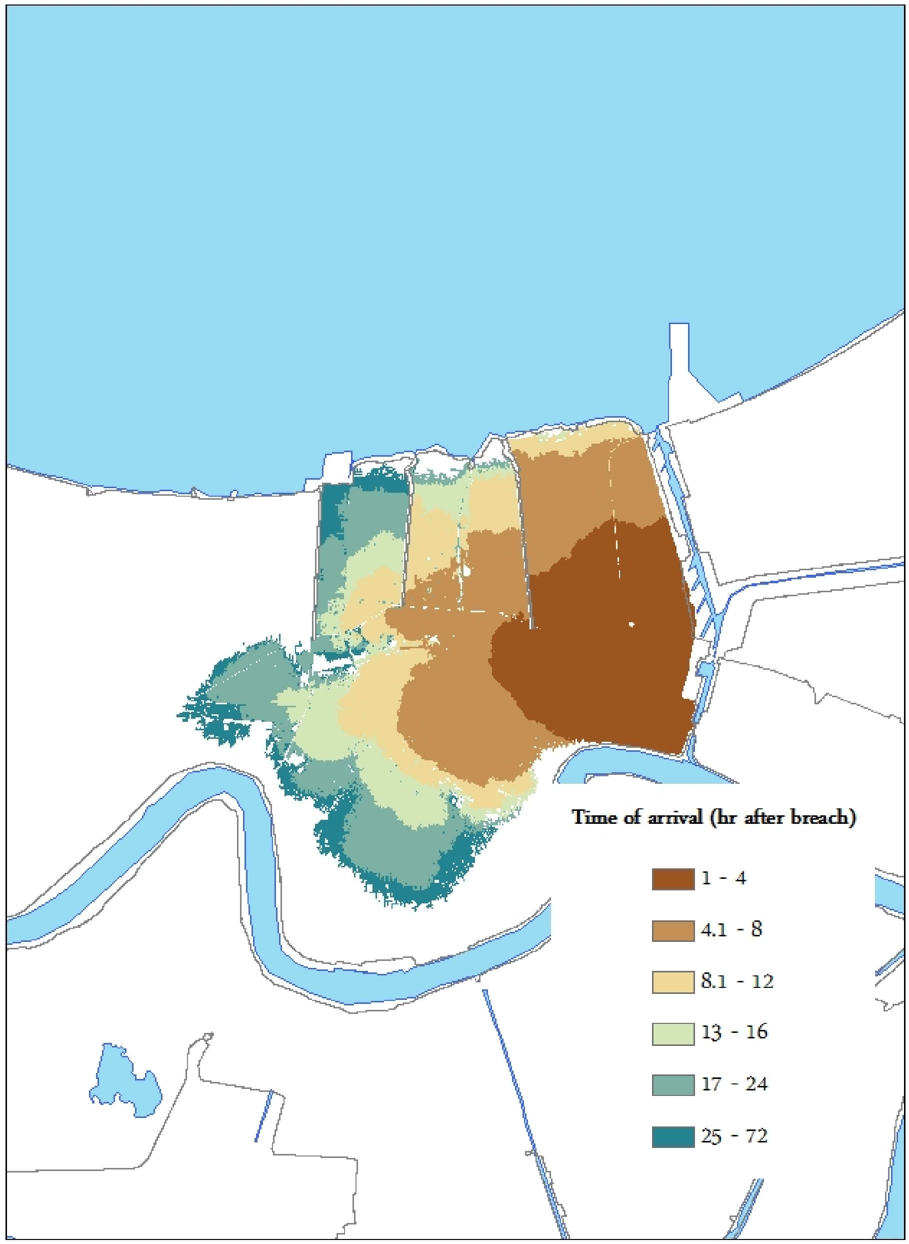

Figure 9-4 Breach due to high river surge, time of arrival (hr)

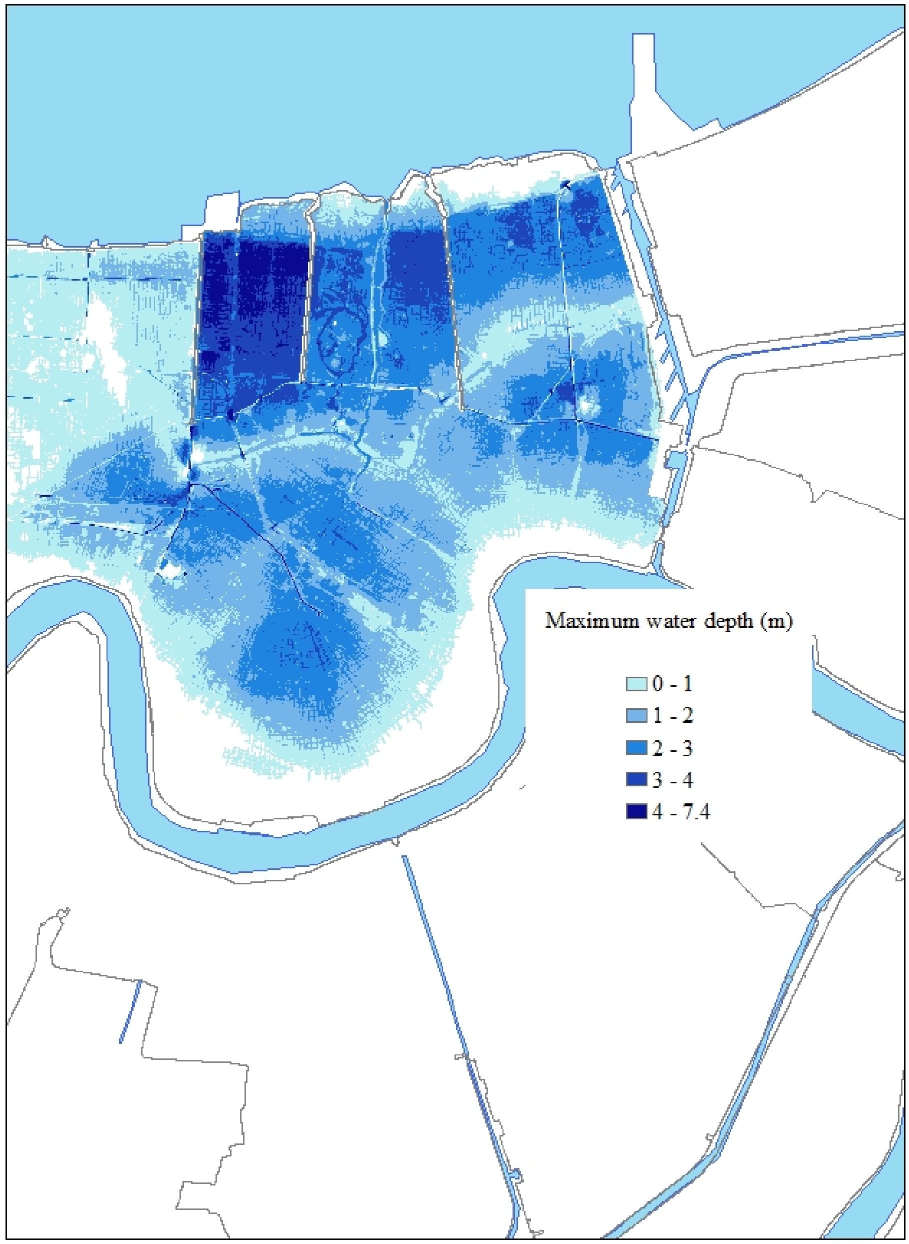

Figure 9-5 Lake breach at West End, maximum water depth (m)

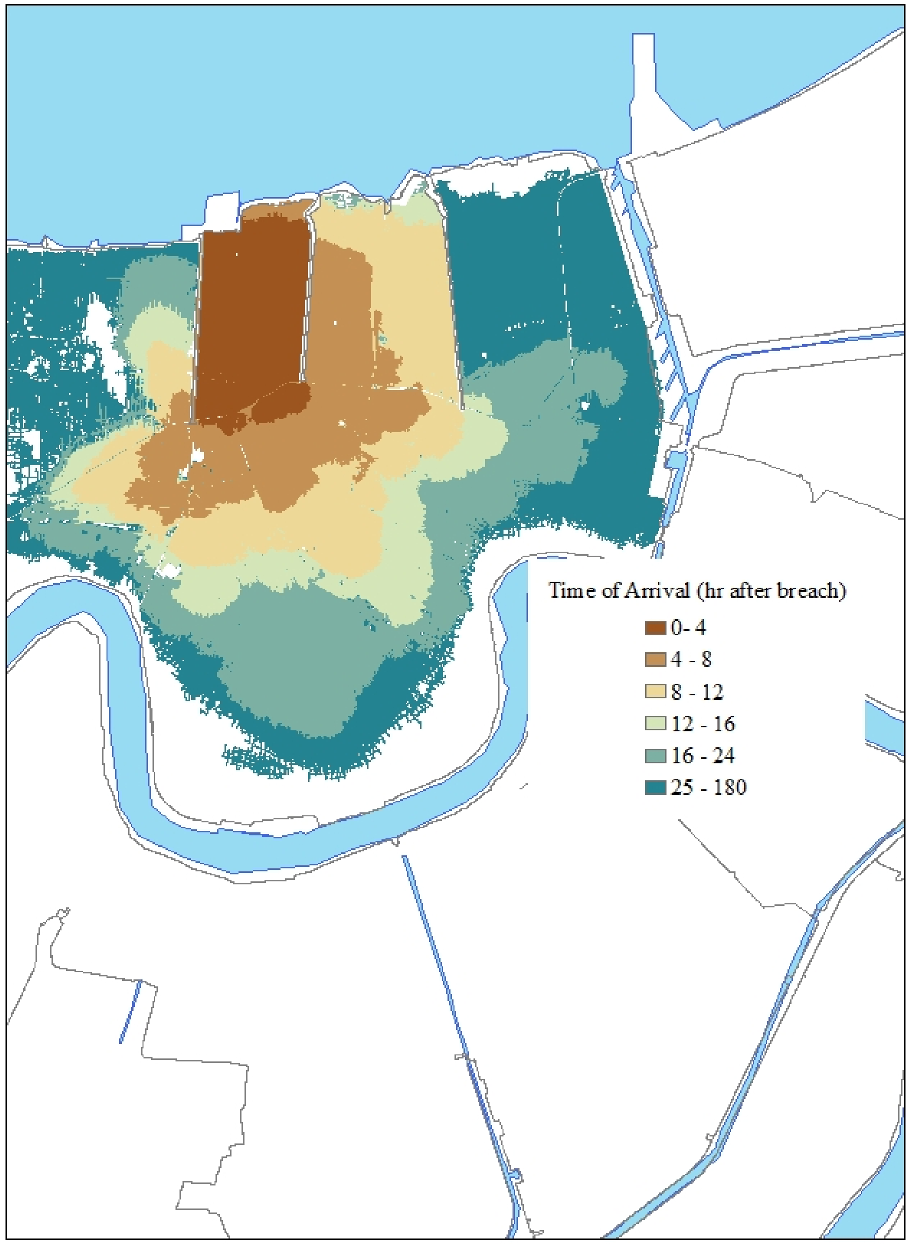

Figure 9-6 Lake breach at West End, time of arrival (hr)

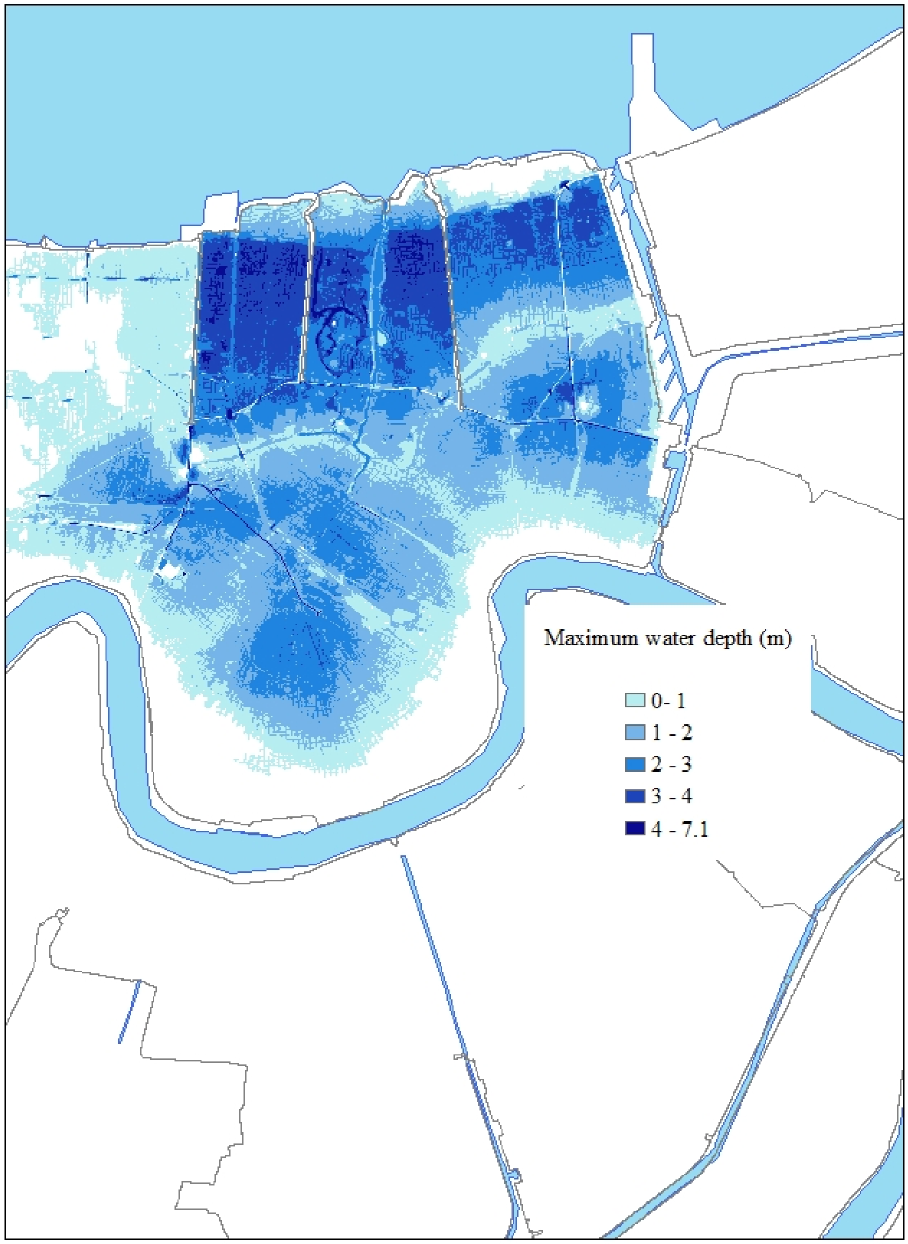

Figure 9-7 Lake breach at St. John, maximum water depth (m)

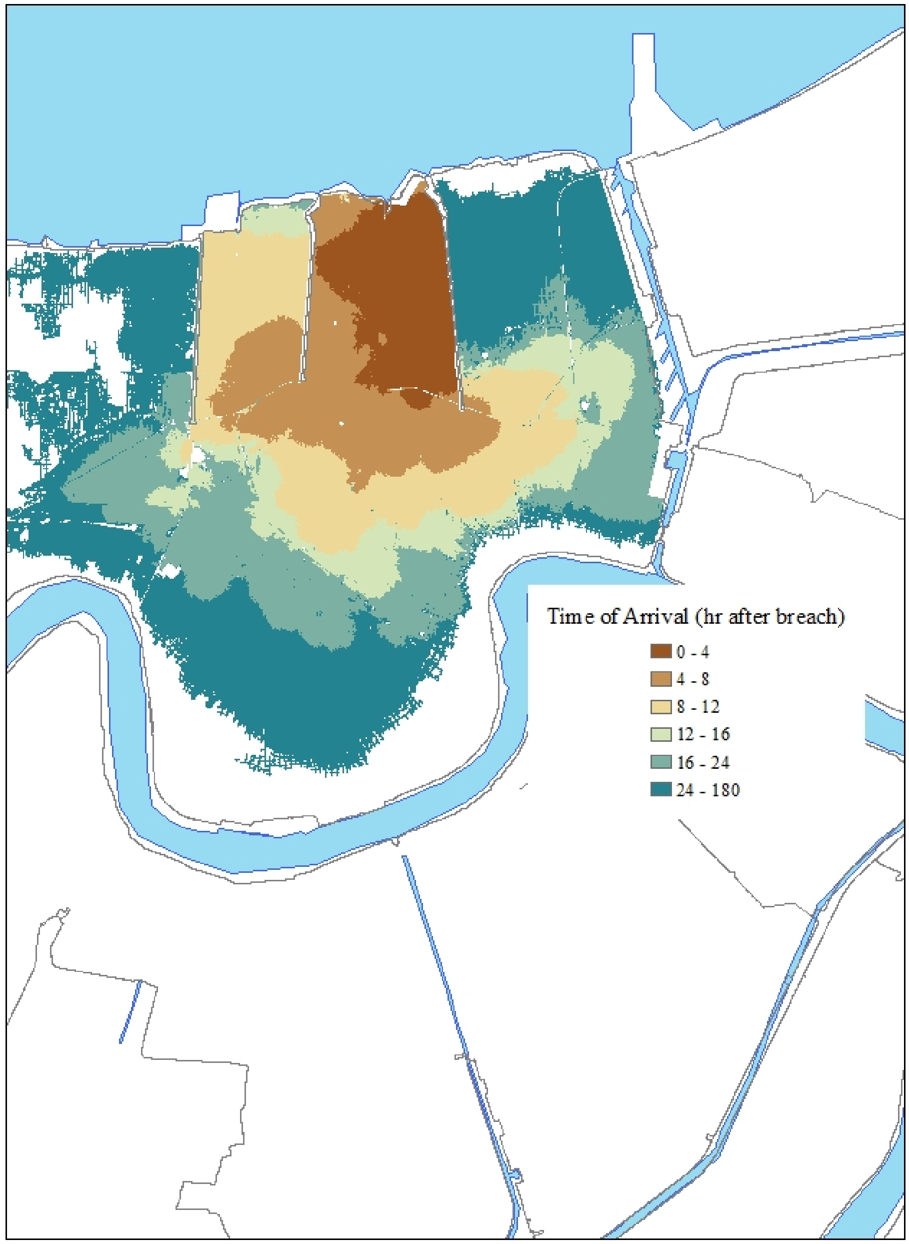

Figure 9-8 Lake breach at St. John, time of arrival (hr)

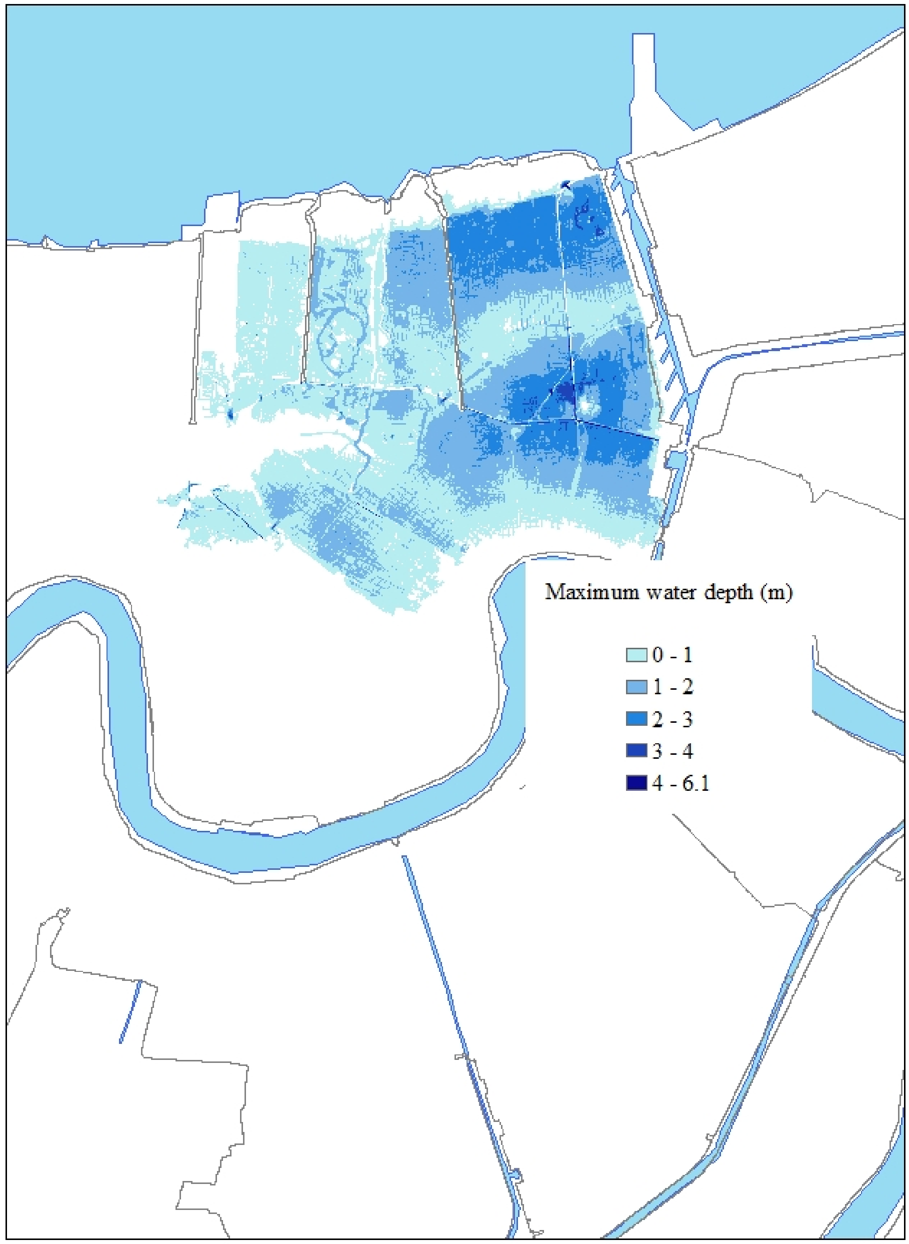

Figure 9-9 IHNC breach, maximum water depth (m)

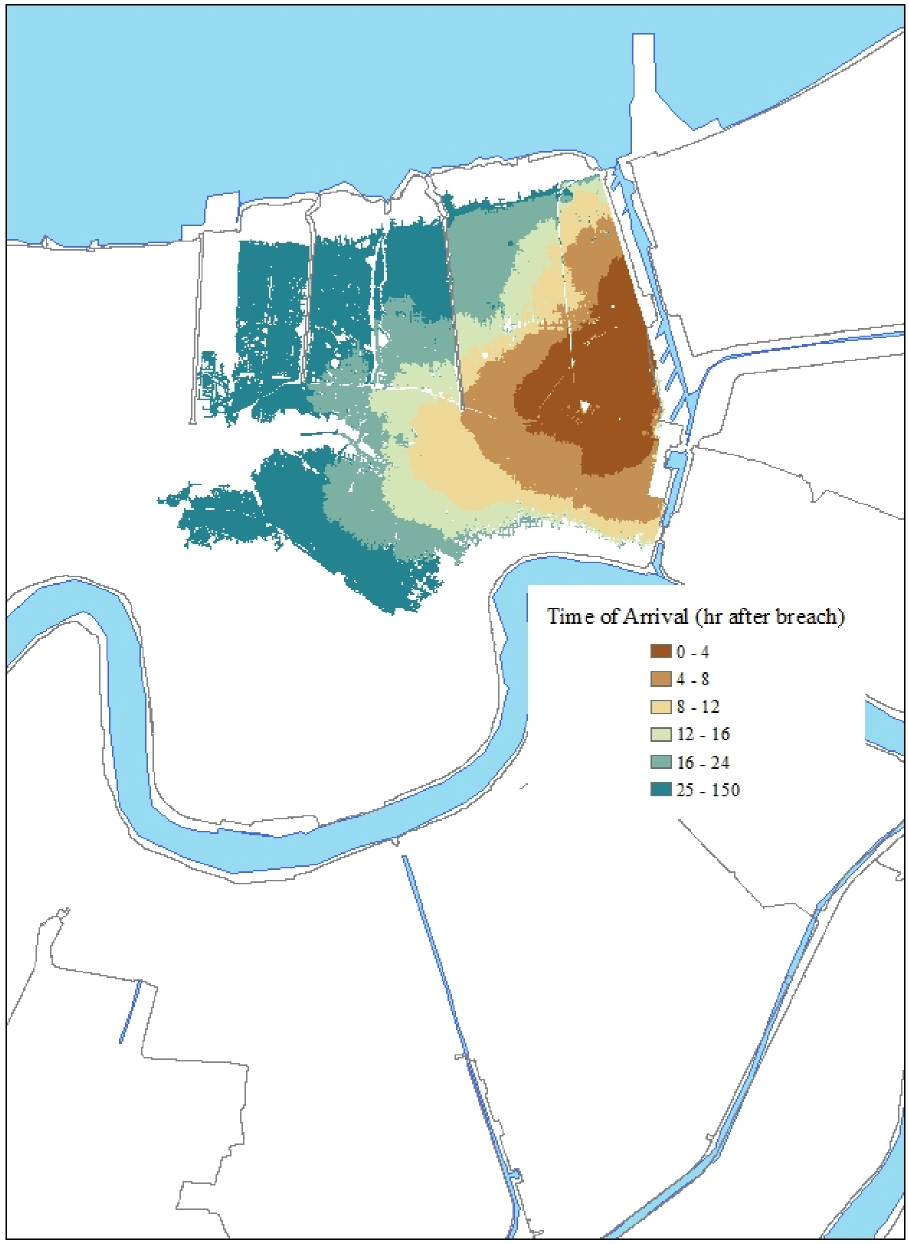

Figure 9-10 IHNC breach, time of arrival (hr)

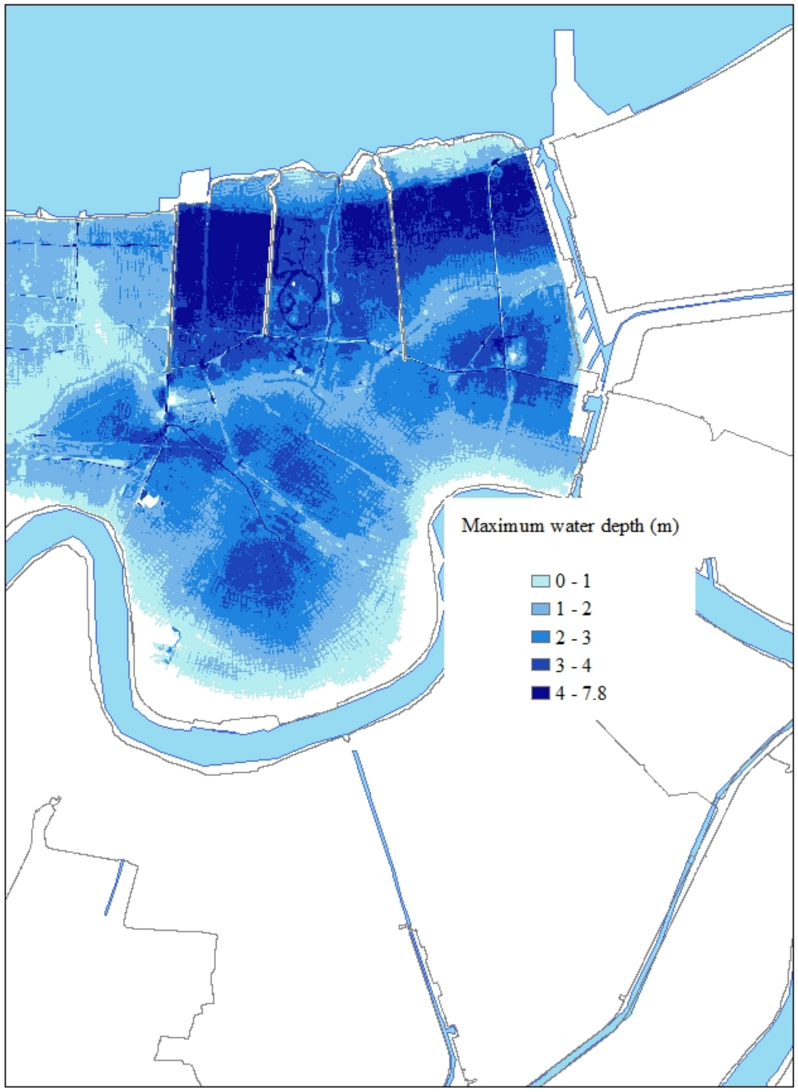

Figure 9-11 Multiple breach, maximum water depth (m)

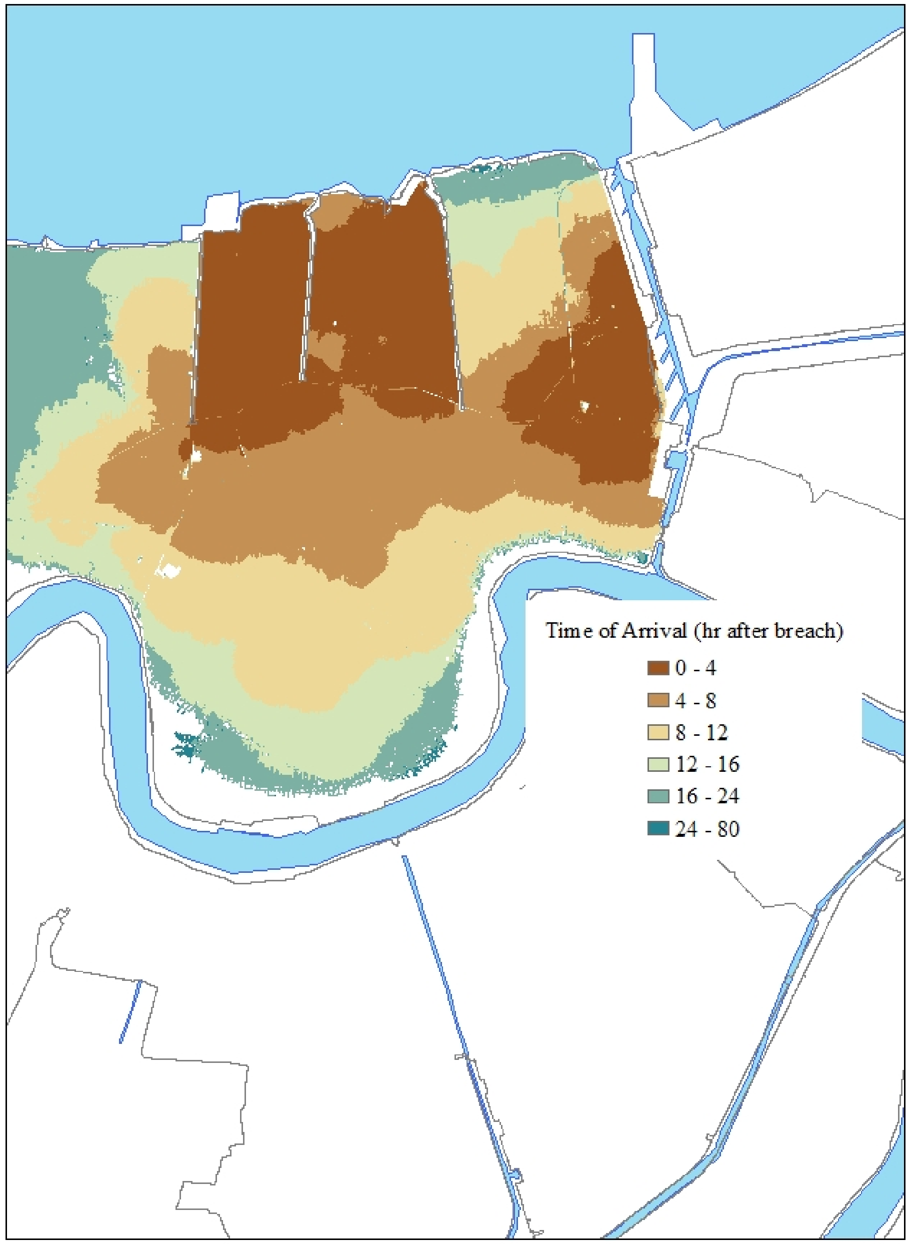

Figure 9-12 Multiple breach , time of arrival (hr)