Risk Analysis of a Protected Hurricane-Prone Region. II: Computations and Illustrations

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Abstract: This paper describes a case study implementing a methodology for assessing risks to protected hurricane-prone regions. A simple hurricane protection system is constructed to illustrate the required inputs for the system definition, computations, and hazard and risk profiles. The inputs include the required specifications for basin and subbasin reaches, transitions, and associated fragilities, closures, and storm parameters. Moreover, the case study produces elevation- and loss-exceedance probability and rate curves for each subbasin and the system as a whole, and demonstrates quantitative benefit-cost analysis using this risk information. The implementation of the risk model is packaged as the Flood Risk Analysis for Tropical Storm Environments tool currently in use by the U.S. Army Corps of Engineering Interagency Performance Evaluation Team charged with assessing hurricane risks to the New Orleans and Southeast Louisiana, and proposed changes to the hurricane protection system.

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Risk Analysis Framework

The severity of losses resulting from the impact of hurricane Katrina on the United States has prompted significant national investment in intellectual capital toward understanding and quantifying the risks to hurricane-prone regions. As noted by many leading risk researchers, quantitative risk analysis provides a means to *inform* the decision making process by communicating the probability and severity of potential risk scenarios (Ayyub 2003). When expressed in terms of clearly defined measures (e.g., dollars), knowledge of risk facilitates defensible benefit-cost analysis to determine the cost effectiveness of alternative risk mitigation strategies. While current national attention is on applying quantitative risk methods for the reconstruction effort in New Orleans (Ayyub et al. 2009), it can be anticipated that other regions will be more inclined to adopt risk methods to manage the risks associated with hurricanes and other natural and human-caused hazards (Ayyub et al. 2007).

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In the engineering community, *risk* is generally defined as the potential of losses for a system resulting from an uncertain exposure to a hazard or as a result of an uncertain event (Ayyub 2003). Risk is quantified as the rate (measured in events per unit time, such as years) that lives, economic, environmental, and social/cultural losses will occur due to the nonperformance of an engineered system or component. The nonperformance of the system or component can be quantified as the probability that specific loads (or demands) exceed respective strengths (or capacities) causing the system or component to fail, and losses are defined as the adverse impacts of that failure if it occurs. Risk can be viewed to be a multidimensional quantity that captures event-occurrence rate (or probability), event-occurrence consequences, consequence significance, and the population at risk. As a measure, risk is commonly represented as a pair of the rate (or probability) of occurrence of an event, and the outcomes or consequences associated with the event's occurrence that account for system weakness, i.e., vulnerabilities, and is commonly expressed as

$$\text{risk} = \text{event rate(or probability)} \times \text{vulnerability} \times \text{consequences of failure} \quad (1)$$

This equation not only defines risk but also offers strategies to control or manage risk: by making the system more reliable or by reducing the potential losses resulting from a failure through vulnerability reduction. Decisions concerning investments in systems designed to control natural hazards are best made by explicitly and quantitatively considering the risks that the systems pose to public safety and property.

Ayyub et al. (2009) applied probabilistic risk analysis to develop an overall methodology for assessing hurricane flood risk to a geographic region that considers the reliability of a hurricane protection system (HPS). The overall methodology is illustrated in Fig. 1. In general, a HPS consists of the following elements:

1. Basins, or areas of land protected against flooding from adjacent bodies of water by levees, dykes, or floodwalls;

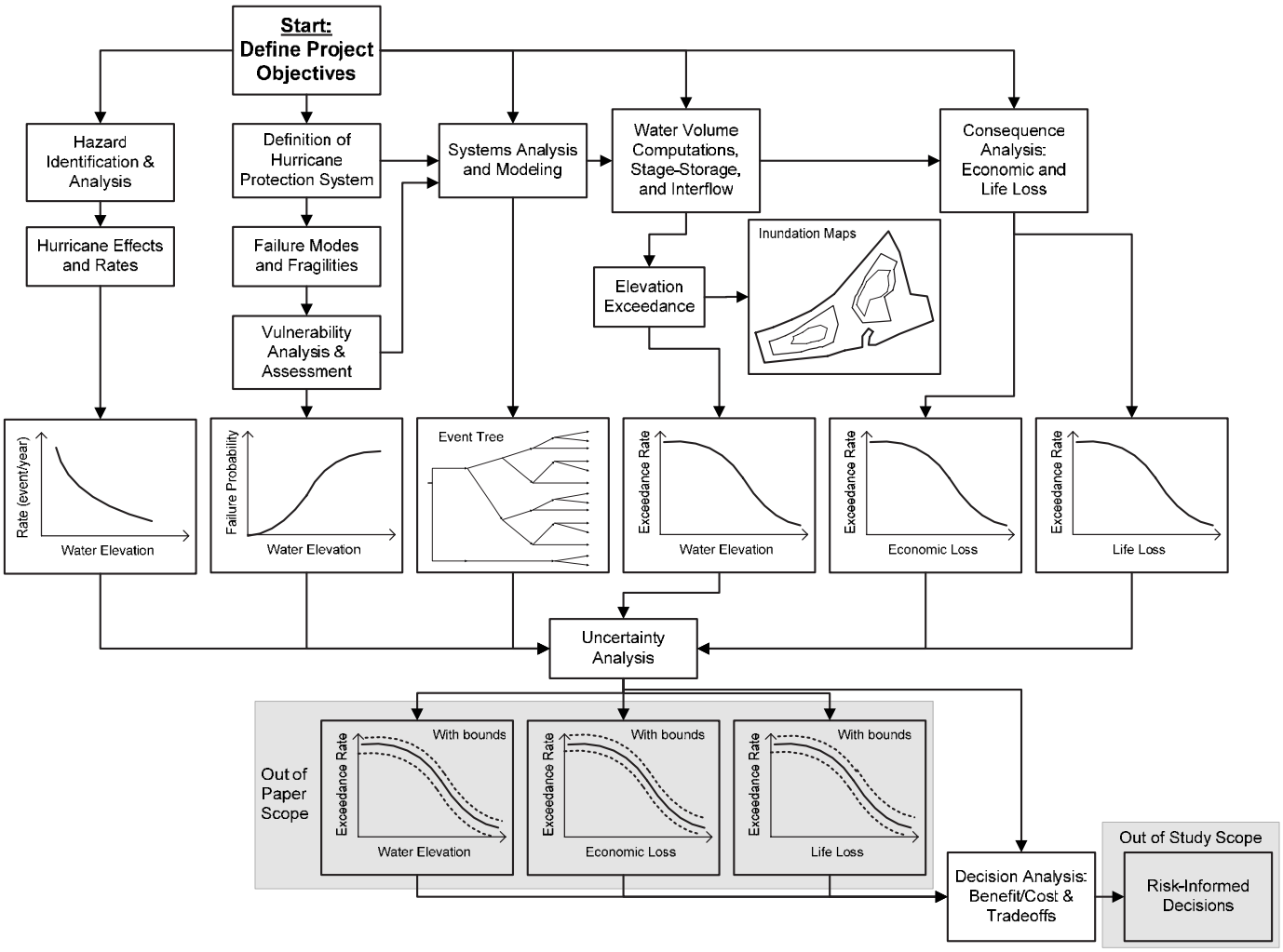

Fig. 1. Overall risk analysis methodology (Ayyub et al. 2007)

2. Subbasins, or divisions within a basin to isolate flooding given failure of a levee, dyke, or floodwall;
3. Reaches, or discrete stretches along the wetted perimeter of a basin that have similar characteristics along the length (e.g., floodwall or levee type and cross section, engineering parameters, jurisdiction);
4. Transitions, or short reaches adjoining two dissimilar reaches;
5. Closures within reaches that permit movement of people and goods during dry conditions; and
6. Pumps that actively drain water from the basins.

USACE (2006) and Ayyub et al. (2009) provide maps and descriptions of basins defining the HPS of New Orleans as examples.

A high-level expression for the risk associated with hurricanes due to the performance of a HPS considers the regional hurricane rate, λ, the probability $P(h_i)$ of realizing a hurricane event of type i given the occurrence of a hurricane, the probability $P(S_j|h_i)$ that the HPS will be left in state j given h_i, and the probability $P(L>l|h_i,S_j)$ with which a consequence measure L exceeds different levels l. Given this information, the loss-exceedance probability per event is evaluated as:

$$P(L>l)=\sum_i \sum_j P(h_i)P(S_j|h_i)P(L>l|h_i,S_j) \tag{2}$$

and the annual loss-exceedance rate was estimated as follows:

$$\lambda(L>l)=\sum_i \sum_j \lambda P(h_i)P(S_j|h_i)P(L>l|h_i,S_j) \tag{3}$$

The summation in Eqs. (2) and (3) is over all hurricane types i and all system states j using a suitable partition of the space of all possible hurricane and HPS failure scenarios (Kaplan et al. 2005). Simulation studies of hurricanes for risk analysis require the use of representative combinations of hurricane parameters and their respective probabilities. The outcome of this process is a set of hurricane simulation cases and their respective conditional rates $\lambda P(h_i)$. Additional information on this model is provided by Ayyub et al. (2009).

An event tree was constructed as shown in Fig. 2 to determine flooding elevations and displaying the results as inundation contours within the basins. The tree events are as follows:

1. Hurricane initiating event: each hurricane, h_i, defines a set of storm surge hydrographs (with waves) for the study area. Given an overall annual hurricane recurrence rate λ, each h_i represents a mutually exclusive partition of an exhaustive set of representative hurricanes. The probability of occurrence for a particular hurricane is denoted as $P(h_i)$, and the corresponding annual rate of occurrence $\lambda_i=\lambda P(h_i)$;
2. Closure structure and operations (C): this event models the

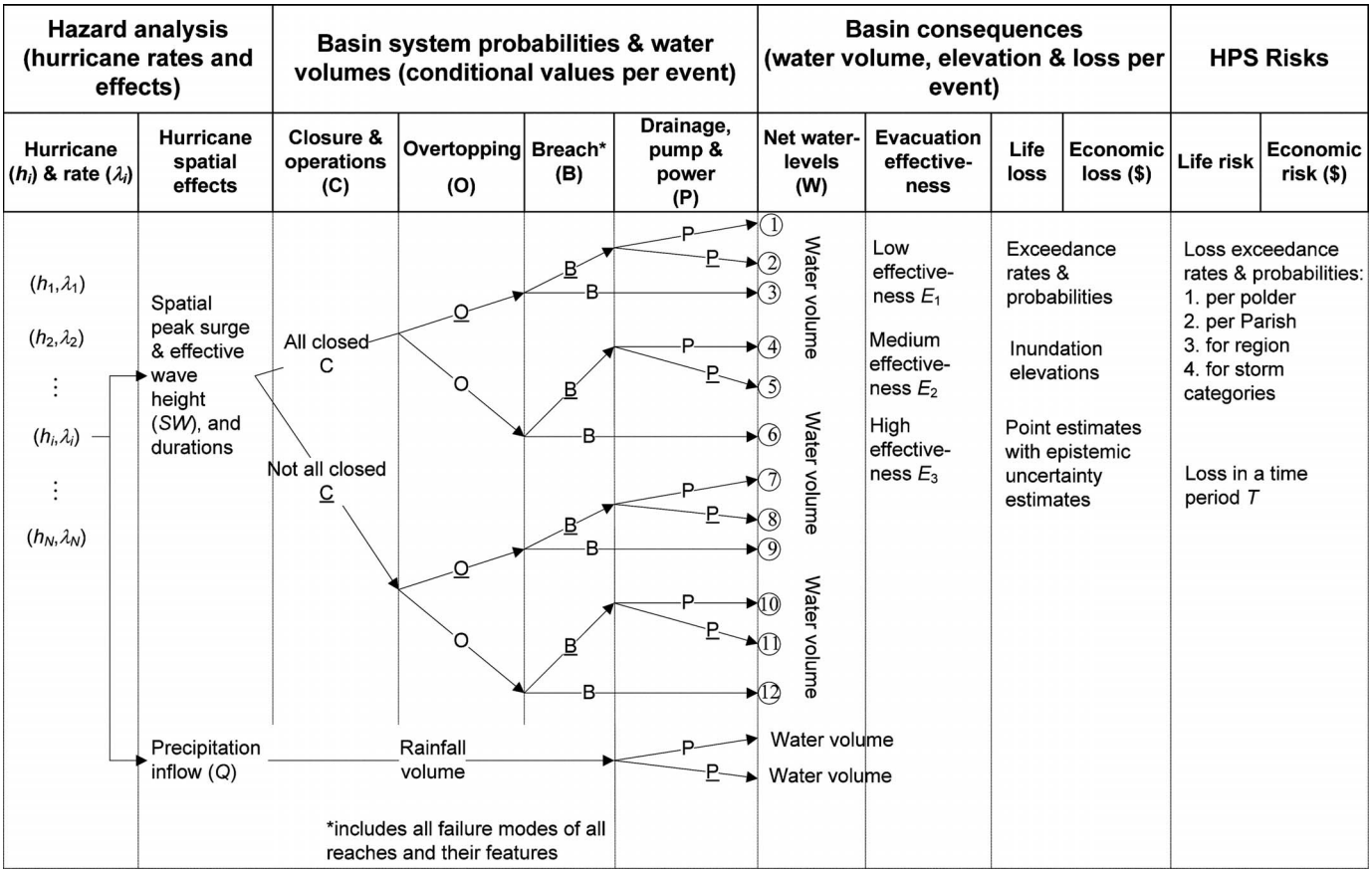

Fig. 2. Event tree for quantifying risk. Underlined events (i.e., $\underline{C}$, $\underline{P}$, $\underline{Q}$, and $\underline{B}$) are complements of respective events (i.e., C, P, O, and B) (Ayyub et al. 2009)

hurricane protection system closures, i.e., gates or ramps, by providing probabilities of sealing them prior to the hurricane;

3. Precipitation inflow (Q): this event corresponds to the rainfall that occurs during a hurricane event. The precipitation inflow per subbasin is treated as a random variable;
4. Drainage, pumping, and power (P): this event models the availability of the pumping systems;
5. Overtopping (O): this event models the failure of the protection system due to overtopping, given that failure has not occurred by some other (i.e., nonovertopping) failure mode. If breach failure does not occur, flooding due to overtopping could still happen; and
6. Breach (B): this event models the failure of the protection system (e.g., levees/floodwalls, closures) during the hurricane, exclusive of overtopping failures.

The computational details of all event probabilities and associated water volumes considering interflow among subbasins are provided by Ayyub et al. (2009). The goal of this paper is to demonstrate this model developed and implemented by the U.S. Army Corps of Engineers (USACE) Interagency Performance Evaluation Task Force (IPET) using a simple case study of a notional city. This example highlights the data inputs required to produce regional elevation- and loss-exceedance probability and rate curves, including parameters that define a HPS, its fragilities, and severity of consequence as a function of flood levels.

Case Study: City of Forteville

This section describes the input information (e.g., storm data, system description) and corresponding risk results for a notional city with a hurricane protection system. For more information on the mathematical details of this methodology, the reader is referred to the companion paper by Ayyub et al. (2009).

Hurricane Protection System Definition

Consider the notional City of Forteville, a city in a hurricane-prone region with a hurricane protection system as shown in Fig. 3. The Forteville hurricane protection system (FHPS) can be divided into four geographically defined basins comprised of one or more subbasins with reaches, transitions, and gates as de-

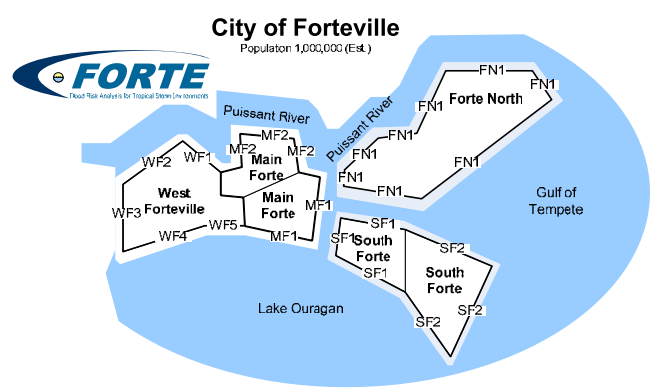

Fig. 3. Map of Forteville showing geographic bounds of study region considered in risk analysis

Table 1. Description of Forteville Hurricane Protection System (FHPS)

Basin	Number of subbasins	Interflow between subbasins	Reaches per subbasin	Number of transitions	Number of gates
South Forte	2	Yes	1	—	2
Main Forte	2	No	1	—	—
West Forteville	1	—	5	2	2
Forte North	1	—	1	—	—

scribed in Table 1. Subbasins are denoted with the initials of the basin followed by an integer identification number (e.g., WF3).

Stage-Storage Relationships

A stage-storage curve was constructed for each of the subbasins as shown in Fig. 4. Stage-storage curves describe the relationship between storage (i.e., volume) of water in a subbasin and the corresponding water stage (i.e., elevation). The starting elevation for each curve in Fig. 4 corresponds to the bottom elevation of the subbasin. A linear relationship between stage and storage is assumed for simplicity, though in practice the true relationship will be nonlinear due to uneven terrain, the presence of objects such as houses, and other obstacles.

Interflow

According to Table 1, the only basin where interflow, that is, the flow of water between adjacent subbasins, is a concern is South Forte. Table 2 provides the symmetric interflow matrix for the FHPS, where each cell provides an elevation of water (in feet) at which the subbasin in the column overflows into the subbasin of the corresponding row. In practice, the structure of this interflow matrix enables the user to define connectedness of an arbitrary pair of subbasins simply by specifying an overflow elevation. Absence of a value in any cell indicates that an interflow cannot occur between the corresponding pair of subbasins.

Table 2. Elevation in Feet for Interflow among Forteville Basins

Subbasin	SF1	SF2	MF1	MF2	WF1	FN1
SF1	—	−9.0	—	—	—	—
SF2	−9.0	—	—	—	—	—
MF1	—	—	—	—	—	—
MF2	—	—	—	—	—	—
WF1	—	—	—	—	—	—
FN1	—	—	—	—	—	—

Note: 1 ft=0.3048 m.

Reaches, Transitions, and Closures

The FHPS is comprised of reaches and transitions divided among the basins as described in Tables 3 and 4, respectively. Within the risk model, transitions and reaches are treated in a similar manner, the only difference being that the transition shares a hydrograph with its associated reach. For each reach and transition, the following characteristics are identified:

1. Length of the reach or transition;
2. Nominal top elevation of the reach or transition which is used to calculate the head of water entering a subbasin due to overtopping;
3. A design elevation needed for defining the respective fragility curve;
4. Whether the reach is a levee or a wall; and
5. The corresponding weir coefficient for calculating volume flow rates from the hydrograph data via the weir formula (Daugherty et al. 1985).

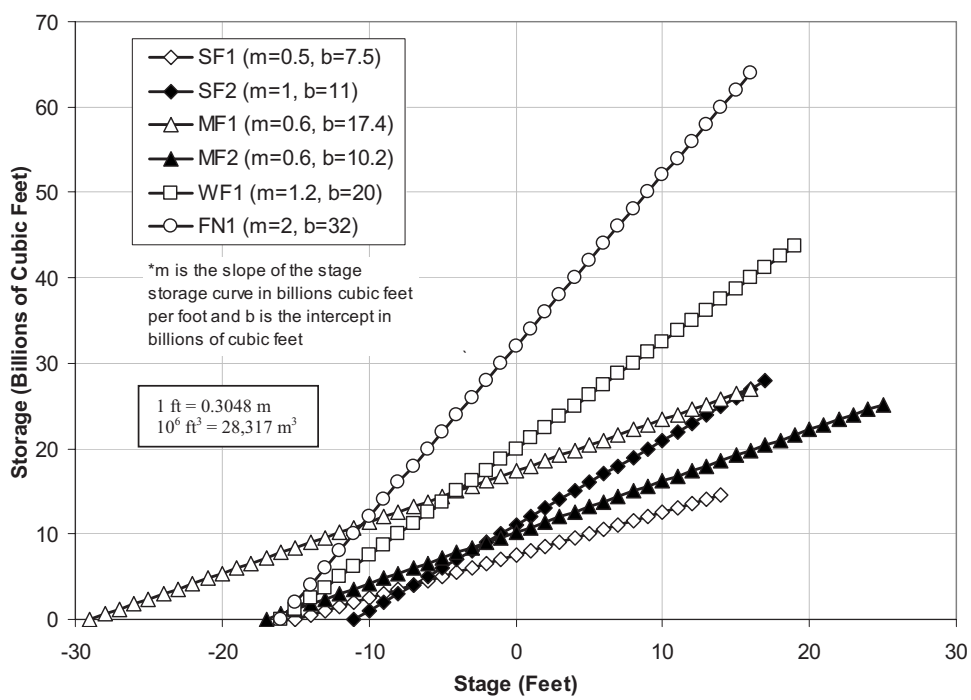

Fig. 4. Stage-storage relationships for Forteville subbasins

Table 3. Description of Forteville Reaches

Basin	Reach (ID)	Length (ft)	Top elevation (ft)	Design water elevation (ft)	Reach type	Reach weir coefficient
FN	FN1 (1)	230,000	17.6	14.0	Levee	2.6
SF	SF1 (2)	75,000	13.1	10.5	Levee	2.6
SF	SF2 (3)	135,000	16.9	13.5	Levee	2.6
MF	MF1 (4)	145,000	15.3	12.5	Wall	3.0
MF	MF2 (5)	55,000	24.3	20.5	Levee	2.6
WF	WF1 (6)	10,000	11.9	6.5	Wall	3.0
WF	WF2 (7)	35,000	14.4	12.0	Levee	2.6
WF	WF3 (8)	30,000	15.5	12.0	Levee	2.6
WF	WF4 (9)	55,000	25.4	22.5	Levee	2.6
WF	WF5 (10)	10,000	13.9	11.0	Wall	3.0

Note: 1 ft=0.3048 m.

Moreover, for transitions the associated reach is identified in order to map it to the hydrograph of the corresponding reach.

Several closures are associated with FHPS reaches as described in Table 5. Each closure is described by the following characteristics:

1. The associated reach containing the closure;
2. Length of the closure;
3. Bottom elevation of the closure which is used to calculate the head of water entering a subbasin; and
4. Probability that the closure will be left open during a storm event.

Table 4. Description of Forteville Transitions

Transition	Length (ft)	Top elevation (ft)	Design water elevation (ft)	Weir coefficient	Associated reach
T1	100	12	10	3.0	WF3
T2	50	15	15	3.0	WF4

Note: 1 ft=0.3048 m.

Table 5. Forteville Closure Data

Closure/gate	Associated reach	Length (ft)	Bottom elevation (ft)	Open probability
G1	SF1	35	1	0.2
G2	SF1	17	7	0.2
G3	SF2	20	10	0.2
G4	SF2	20	10	0.2
G5	WF2	6	6	0.2
G6	WF3	8	7	0.2

Note: 1 ft=0.3048 m.

Hurricanes

A set of ten hurricanes is considered with annual recurrence rates and precipitation volumes described in Table 6. For the purposes of this example, this set of hurricanes is assumed to be an exhaustive partition of the space of hurricane scenarios. Furthermore, it is assumed that Forteville will experience hurricanes at a rate of two per year. A set of hydrographs for the subbasins was constructed for each storm as illustrated in Fig. 5 and described in Table 7 with peak hydrograph elevations expressed as percentages of the reach height relative to sea level. For simplicity, each hydrograph was assumed to take on a trapezoidal shape spanning a period of 72 h (259,200 s), with rise to the maximum water elevation accounting for the first 50% of the hydrograph duration, sustainment of the peak water elevation described in Table 9 for the next 15% of the hydrograph duration, and a descend period accounting for the remainder of the hydrograph duration.

Fragility Relationships and Breach Model

A fragility curve was constructed for each reach and transition that provides the relationship between probability of breach fail-

Table 6. Storm and Precipitation Data

Storm	Rate (events per year)	Precipitation volume per subbasin (millions of cubic feet of water)[a]					
		SF1	SF2	MF1	MF2	WF1	FN1
1	9.437E−01	9.0	20.0	8.0	7.0	10.0	40.0
2	4.990E−01	74.7	85.7	73.7	72.7	75.7	105.7
3	2.638E−01	140.3	151.3	139.3	138.3	141.3	171.3
4	1.395E−01	206.0	217.0	205.0	204.0	207.0	237.0
5	7.376E−02	271.7	282.7	270.7	269.7	272.7	302.7
6	3.900E−02	337.3	348.3	336.3	335.3	338.3	368.3
7	2.062E−02	403.0	414.0	402.0	401.0	404.0	434.0
8	1.090E−02	468.7	479.7	467.7	466.7	469.7	499.7
9	5.765E−03	534.3	545.3	533.3	532.3	535.3	565.3
10	3.048E−03	600.0	1,000.0	700.0	700.0	1,000.0	2,000.0

Note: 1,000,000 ft^3=28,317 m^3.

[a]0.25 coefficient of variation assumed on all precipitation volumes.

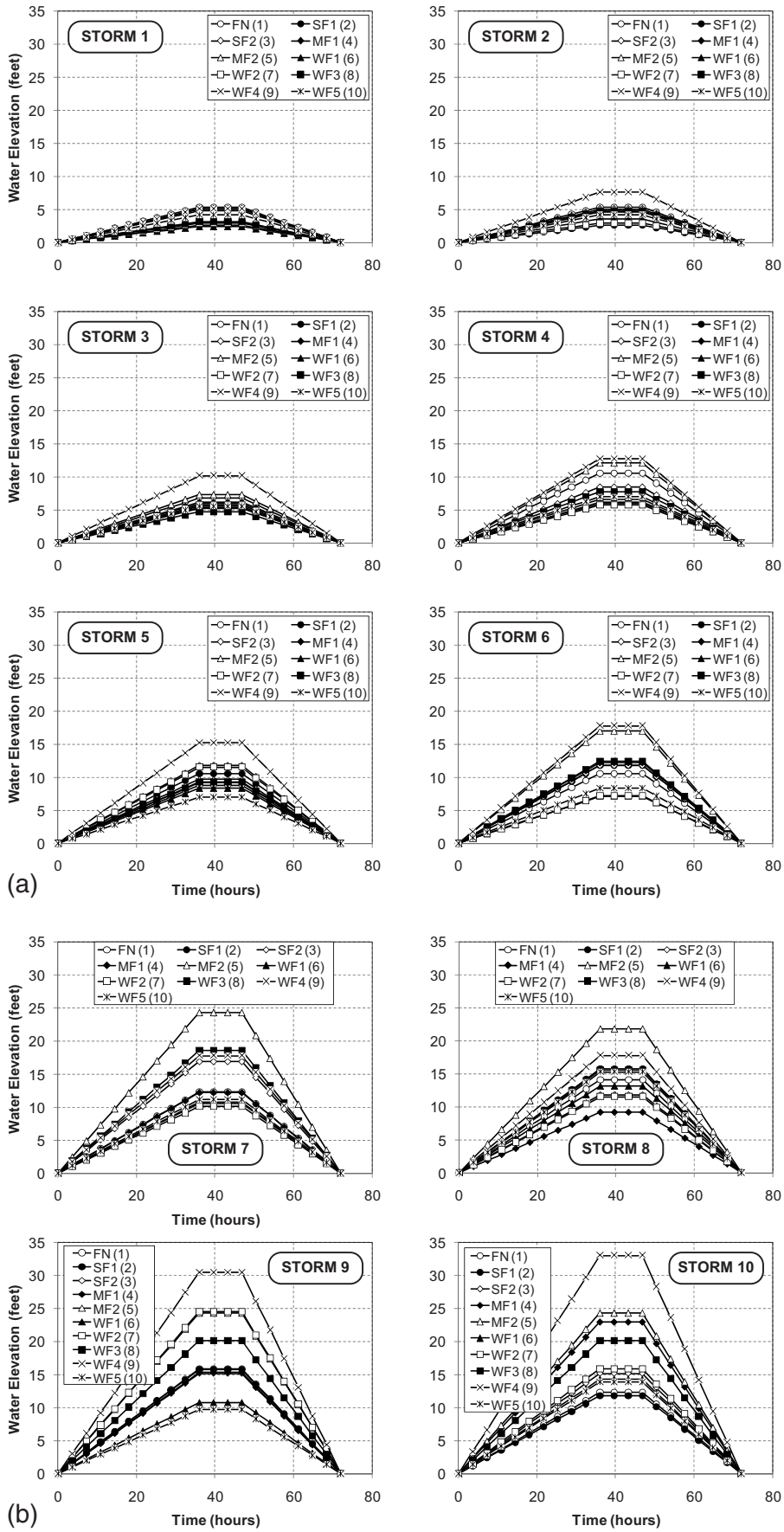

Fig. 5. Storm hydrographs (1 ft=0.3048 m)

Table 7. Maximum Hydrograph Elevation as Function of Storm

Reach ID	Top elevation (ft)	Maximum hydrograph elevation as percentage of reach height by storm									
		1	2	3	4	5	6	7	8	9	10
FN1	17.6	0.3	0.3	0.3	0.6	0.5	0.6	0.7	0.8	0.9	0.7
SF1	13.1	0.2	0.2	0.4	0.5	0.8	0.9	0.8	1.2	1.2	0.9
SF2	16.9	0.3	0.3	0.4	0.5	0.7	0.7	1	0.7	0.9	0.9
MF1	15.3	0.2	0.3	0.4	0.4	0.6	0.8	0.8	0.6	1	1.5
MF2	24.3	0.2	0.2	0.3	0.5	0.4	0.7	1	0.9	1	1
WF1	11.9	0.2	0.3	0.5	0.5	0.7	0.6	0.9	1.1	0.9	1.2
WF2	14.4	0.2	0.2	0.4	0.4	0.8	0.5	0.7	0.8	1.7	1.1
WF3	15.5	0.2	0.3	0.3	0.5	0.6	0.8	1.2	1	1.3	1.3
WF4	25.4	0.2	0.3	0.4	0.5	0.6	0.7	0.7	0.7	1.2	1.3
WF5	13.9	0.3	0.3	0.4	0.5	0.5	0.6	0.8	1.1	0.7	1

Note: 1 ft=0.3048 m.

ure and water level as shown in Fig. 6 and described in Tables 8 and 9 for reaches and transitions, respectively. Each fragility curve is nondecreasing with respect to increasing elevation, and is characterized by eight data points as follows:

1. *Low limit* with a default probability of breach failure of 10^{-12}. The low limit elevation corresponds to the sea level elevation (i.e., 0 ft). This is the minimum probability of breach, and applies to all elevations below the low limit;
2. The *design elevation* is the elevation of water that the reach or transition was designed to withstand. This elevation is less than the top elevation;
3. The *top elevation* is the elevation of the top of the reach or transition;
4. 0.5 ft *overtopping* (OT) corresponds to an elevation that is 0.5 ft above the top elevation;
5. 1.0 ft *overtopping* (OT) corresponds to an elevation that is 1.0 ft above the top elevation;
6. 2.0 ft *overtopping* (OT) corresponds to an elevation that is 2.0 ft above the top elevation;
7. 3.0 ft *overtopping* (OT) corresponds to an elevation that is 3.0 ft above the top elevation; and
8. 6.0 ft *overtopping* (OT) corresponds to an elevation that is 3.0 ft above the top elevation.

By default, the 6.0 ft OT elevation has a probability of breach failure of 1.0. The probability of breach failure at elevations between two data points is determined using semilog interpolation, where the elevation is linear and the probability of breach failure is logarithmic.

Given that a breach failure occurs, the depth and width of the breach is determined as a function of maximum hydrograph surge elevation as described in Table 10 for different reach and transition materials. This information permits calculation of the volume of water entering a subbasin due to breach using the weir formula, where the length of the opening is taken as breach width and the head of water is taken as the difference between the

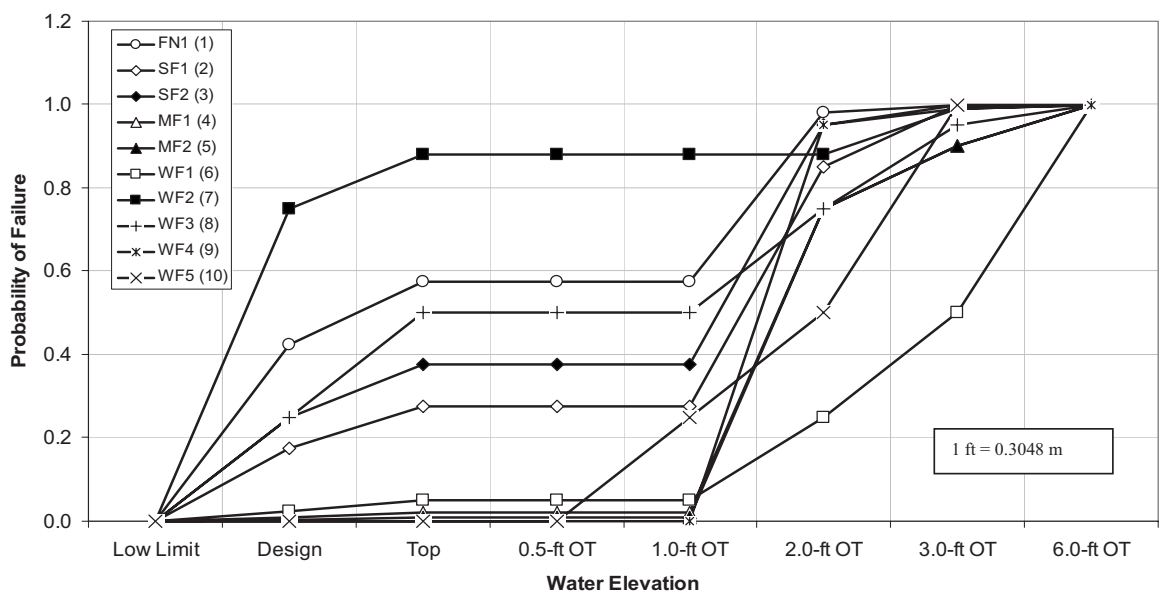

Fig. 6. Breach fragility curves for Forteville reaches

Table 8. Breach Fragilities for Forteville Reaches

Reach (ID)	Breach fragilities versus water elevation								Material
	Low limit	Design	Top	0.5 ft OT	1.0 ft OT	2.0 ft OT	3.0 ft OT	6.0 ft OT	
FN1 (1)	1.00E−12	4.25E−01	5.75E−01	5.75E−01	5.75E−01	9.80E−01	1.00E+00	1.00E+00	Hydraulic fill (HB)
SF1 (2)	1.00E−12	1.75E−01	2.75E−01	2.75E−01	2.75E−01	8.50E−01	1.00E+00	1.00E+00	Clay (CB)
SF2 (3)	1.00E−12	2.50E−01	3.75E−01	3.75E−01	3.75E−01	9.50E−01	1.00E+00	1.00E+00	Sand (SB)
MF1 (4)	1.00E−12	7.50E−03	2.00E−02	2.00E−02	2.00E−02	7.50E−01	9.00E−01	1.00E+00	Wall (WB)
MF2 (5)	1.00E−12	2.50E−03	7.50E−03	7.50E−03	7.50E−03	7.50E−01	9.00E−01	1.00E+00	Hydraulic fill (HB)
WF1 (6)	1.00E−12	2.50E−02	5.00E−02	5.00E−02	5.00E−02	2.50E−01	5.00E−01	1.00E+00	Wall (W7)
WF2 (7)	1.00E−12	7.50E−01	8.80E−01	8.80E−01	8.80E−01	8.80E−01	9.90E−01	1.00E+00	Hydraulic fill (H9)
WF3 (8)	1.00E−12	2.50E−01	5.00E−01	5.00E−01	5.00E−01	7.50E−01	9.50E−01	1.00E+00	Clay (C9)
WF4 (9)	1.00E−12	1.00E−12	1.00E−12	1.00E−12	1.00E−12	9.50E−01	9.90E−01	1.00E+00	Sand (SB)
WF5 (10)	1.00E−12	1.00E−12	1.00E−12	1.00E−12	1.00E−12	2.50E−01	5.00E−01	1.00E+00	Wall (W7)

Note: 1 ft=0.3048 m.

Table 9. Breach Fragilities for Forteville Transitions

Reach (ID)	Breach fragility curve								Type
	Low limit	Design	Top	0.5 ft OT	1.0 ft OT	2.0 ft OT	3.0 ft OT	6.0 ft OT	
T1	1.00E−12	1.00E−12	1.00E−12	4.47E−07	2.00E−01	9.00E−01	1.00E+00	1.00E+00	Drainage (*D*)
T2	1.00E−12	1.00E−12	1.00E−12	4.47E−07	2.00E−01	7.00E−01	1.00E+00	1.00E+00	Pumping (*P*)

Note: 1 ft=0.3048 m.

height of water from the hydrograph and the top elevation minus the breach depth. When this difference is negative, the head is set to zero.

Population and Property at Risk

In order to assess losses due to flooding resulting from a hurricane, a function that maps water elevation to loss is required. For the purposes of this example, a deterministic mapping from water elevation in a given subbasin to property damage measured in millions of dollars is used as described in Fig. 7. A linear relationship between property damage and water elevation is assumed with a unique slope m and intercept b for each subbasin as described in the legend of Fig. 7. The bottom elevation (i.e., first data point) of these curves corresponds to the bottom elevation of the associated subbasins, and the peak elevation (i.e., last data point) corresponds to the maximum possible property damage

Table 10. Breach Failure Data for Reaches and Transitions

Material	Symbol	No overtopping (depth independent)		Overtopping depth (ft)					
				0–2 ft		2–5 ft		>5 ft	
		Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)	Depth (ft)	Width (ft)
Hydraulic fill (30,000–39,999 ft)	H9	18	4,500	0	0	9	12,000	18	12,000
Hydraulic fill (>50,000 ft)	HB	18	7,500	0	0	9	20,000	18	20,000
Clay (30,000–39,999 ft)	C9	13	3,000	0	0	3	3,000	13	3,000
Clay (>50,000 ft)	CB	13	5,000	0	0	3	5,000	13	5,000
Sand (>50,000 ft)	SB	17	6,250	0	0	6	15,000	17	15,000
Wall (10,000–10,999 ft)	W7	17	750	0	0	0	0	17	1,000
Wall (>50,000 ft))	WB	17	3,750	0	0	0	0	17	5,000
Drainage structure (transition)	D	0	0	5.5	65	5.5	65	5.5	65
Pump station (transition)	P	0	0	5	100	5	100	5	100

Note: 1 ft=0.3048 m.

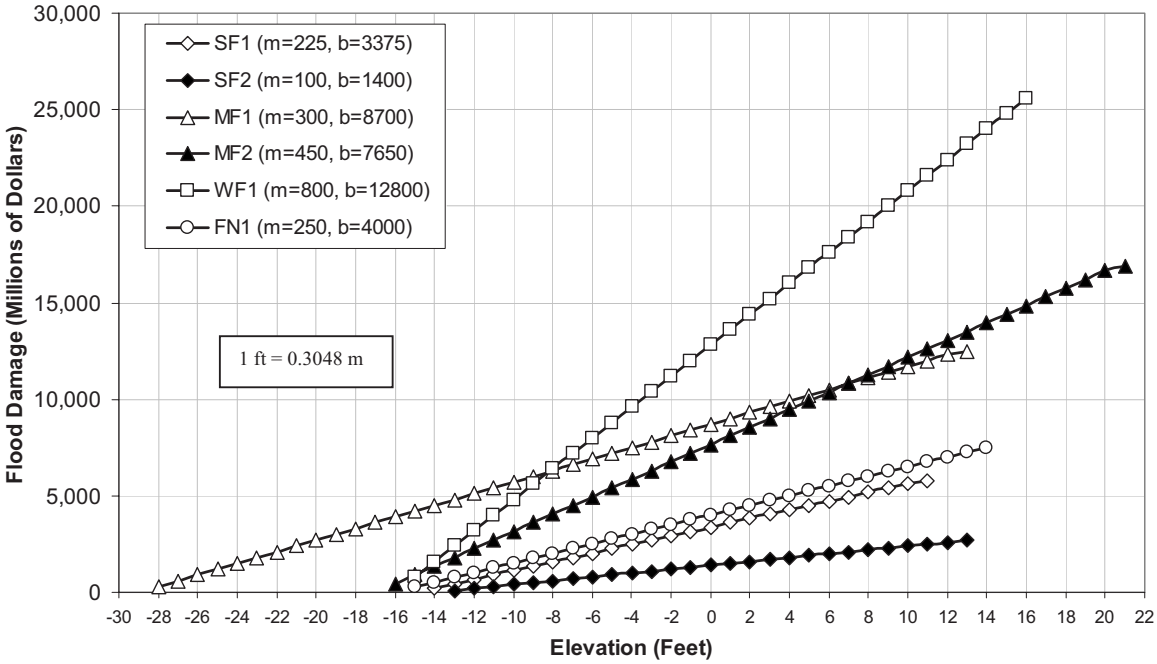

Fig. 7. Flood damage losses as function of water elevation

losses. In practice, a suite of curves that maps water elevation to both property losses and casualties, with associated percentile values, would be provided.

Further Assumptions

For the purpose of this example, the following additional assumptions are made:

1. The reduction of volume due to pumping is not considered. Thus, this example takes the conservative view that pumping is ineffective at reducing water volume. The omission of pumping reduces the number of event tree branches shown in Fig. 2 from 16 to eight;
2. All closures within a subbasin are assumed to be either all open or all closed during a storm event, with the probability being specified for the open condition (i.e., they are perfectly correlated);
3. Several distinct coefficients of variations (COVs) were assumed for the volume calculations as described in Table 11. These COVs account for the aleatory uncertainty (i.e., nonreducible uncertainty due to inherent randomness) in the weir coefficient and precipitation volumes. This example does not account for epistemic uncertainty (i.e., uncertainty due to lack of knowledge). Ayyub and Klir (2006) provides additional information on these uncertainty types and others;
4. Given the mean and standard deviation (i.e., the product of the coefficient of variation and the mean) for volume combined with the knowledge that volume is a non-negative quantity, the lognormal distribution was chosen to represent the uncertainty in water volume; and
5. To account for surge and wave effects on the hydrographs, the aggregate surge and wave hydrographs are adjusted by a bias factor with a median of 1 and log standard deviation of 0.15. Ten stratifications of this distribution were used in the analysis to account for uncertainty in the hydrographs.

Risk Results

Based on the description of the FHPS and the computation details provided by Ayyub et al. (2007) for calculating probability and water volume for each branch in Fig. 2, the elevation-exceedance curves for each subbasin was obtained as shown in Fig. 8. Though not illustrated as part of this example, the elevation-exceedence information can also be communicated via a flood inundation map that visualizes total wetted area as a function of return period. For illustration, selected intermediate calculations for Storm 7, Stratification 7 are given in Table 12 for reach calculations, Table 13 for basin probability calculations, Table 14 for interflow among SF subbasins for Branch 10, and Table 15 for subbasin branch probabilities and corresponding percentiles for water elevation. The property damage loss-exceedance curves for each subbasin were obtained as shown in Fig. 9. Note that these curves show the annual rate of exceeding a given value of loss as a function of loss. Using techniques for loss accumulation assuming that the loss potential is constant between hurricane events, the cunulative distribution function (CDF) for the accumulated property damage loss in a single year for each subbasin was obtained as shown in Fig. 10. Based on these results, the expected annual loss per year was calculated to be $2.28 (rounded) billion with a COV of 2.54. It should be noted that the large COV can be attributed to the multimodal nature of the loss distribution.

Table 11. Analysis Uncertainty Parameters

Variable	Coefficient of variation (COV)
Rainfall volume	0.25
Water volume resulting from breach	0.30
Water volume resulting from overtopping	0.20
Water volume resulting from open closures	0.20

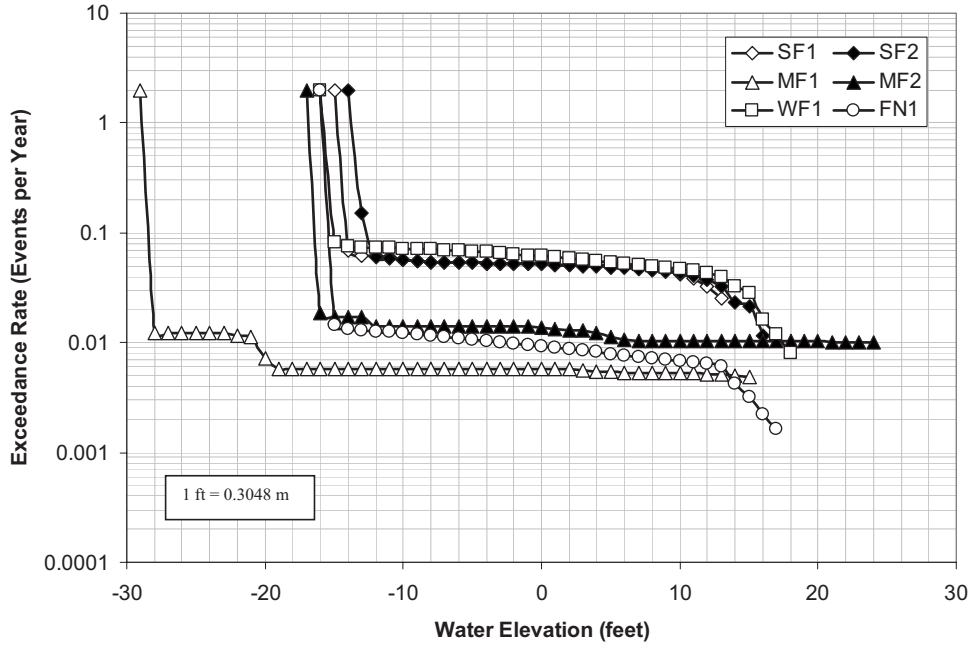

Fig. 8. Elevation-exceedance curves for FHPS subbasins

Table 12. Reach Calculations for Storm 7, Stratification 7

Reach/ transition number	Aggregated surge and waves (ft)	Probability of overtopping	Water volume from overtopping		Probability of breach	Water volume from breach		Probability of not closed	Water volume from open closures	
			Mean (ft^3)	StD (ft^3)		Mean (ft^3)	StD (ft^3)		Mean (ft^3)	StD (ft^3)
FN1 (R1)	13.06	0	—	—	0.07	6.56E+10	1.97E+10	—	—	—
SF1 (R2)	11.13	0	—	—	0.20	3.44E+10	1.03E+10	0.2	2.04E+08	4.09E+07
SF2 (R3)	17.88	1	1.83E+10	3.66E+09	0.38	2.79E+11	8.38E+10	0.2	1.31E+08	2.61E+07
MF1 (R4)	12.96	0	—	—	0.01	3.24E+10	9.72E+09	—	—	—
MF2 (R5)	25.72	1	1.29E+10	2.57E+09	0.05	3.44E+11	1.03E+11	—	—	—
WF1 (R6)	11.36	0	—	—	0.05	5.32E+09	1.60E+09	—	—	—
WF2 (R7)	10.68	0	—	—	0.04	2.91E+10	8.73E+09	0.2	9.51E+07	1.90E+06
WF3 (R8)	19.71	1	4.01E+10	8.01E+09	0.97	4.02E+10	1.21E+10	0.2	4.60E+08	9.20E+06
WF4 (R9)	18.84	0	—	—	—	—	—	—	—	—
WF5 (R10)	11.78	0	—	—	—	—	—	—	—	—
T1	—	—	—	—	1.00	2.51E+08	7.54E+07	—	—	—
T2	—	—	—	—	1.00	2.40E+08	7.20E+07	—	—	—

Note: 1 ft=0.3048 m, 1 ft^3=0.028317 m^3.

Benefit-Cost Analysis

Given baseline risk information for a hurricane-prone region protected by a HPS, benefit-cost analysis can be used to assess the cost effectiveness of alternative risk mitigation strategies. In the context of protecting a region against floods resulting from post-hurricane surges, risk mitigation options include strengthening levees, increasing the span and depth of the levees, relocating residential and commercial centers, and enhancing emergency response procedures. For the purposes of illustrating procedures to conduct benefit-cost analysis, consider a proposed action to increase the hardness of all WF reaches and transitions such that their heights are no less than 15 ft (while maintaining the same ratio of design-to-top elevation) and the probability of breach

Table 13. Basin Probability Calculations for Storm 7, Stratification 7

Basin ID	Probability one or more reaches overtopping $P(O)$	Probability of one or more breach failures $P(B)$	Probability of all pumps working $P(P)$	Probability of all closures are closed $P(C)$
SF (SF1+SF2)	1	0.497	0	0.64
MF1	0	0.009	0	0
MF2	1	0.052	0	0
WF1	1	1.000	0	0.64
FN1	0	0.071	0	0

Table 14. Interflow Calculations for SF Basin, Storm 7, Stratification 7, Branch 10

	Preinterflow volume (ft^3)		Postinterflow volume (ft^3)	
Subbasin	Mean	Std. dev.	Mean	Std. dev.
SF1	4.03E+08	1.01E+08	7.77E+09	1.93E+09
SF2	1.87E+10	3.66E+09	1.14E+10	2.23E+09

Note: 1 ft=0.3048 m, 1 ft^3=0.028317 m^3.

failure is set equal to 10^{-12} for all elevations up to 1.0 ft overtopping (leaving the probability of failure for surge elevations above 1.0 ft overtopping unchanged). Note that the same hydrographs were used for both the original FHPS and the system after implementation of the risk mitigation measures; in practice, the shape of the hydrographs is dependent on the system configuration, and any change to an HPS would require a new set of hydrographs to be generated. Following the procedures described in the companion paper for calculating accumulated property damage losses in a single year (Ayyub et al. 2009), the expected annual loss is $1.46 billion (rounded) after implementing the risk mitigation. Thus the proposed risk mitigation would yield an expected benefit of approximately $822 million per year with a COV of 4.23. The CDF for benefit was obtained as shown in Fig. 11.

Assuming an average cost of $10 million to raise 1,000 ft of reach by 1 ft (with a coefficient of variation of 0.10 on the total cost) and an expected fixed cost of $250 million (COV of 0.05) to improve the fragilities of the levees, the total cost to implement this strategy is $883 million (with a COV of 0.073). All costs are assumed to be normally distributed random variables. Given an upfront (present value) cost, P, the total annual equivalent cost, A, over a lifetime of n years with a fixed annual interest rate i, can be determined as follows (Ayyub and McCuen 2003):

$$A = P\left(\frac{i(1+i)^n}{(1+i)^n - 1}\right) \tag{4}$$

Assuming a project lifetime of 25 years (n=25) and a fixed 10% annual interest rate (i=0.1), the total equivalent annual cost of the proposed risk mitigation is $97,278,408 per year with a COV of 0.073 obtained through standard techniques for uncertainty propagation. Differential maintenance costs between the before and after states of the FHPS are assumed to be negligible.

The probability of exceeding a specified benefit-to-cost ratio for this proposal was determined, as shown in Fig. 12, using the equations described in the companion paper (Ayyub et al. 2009). Though the expected benefit is much greater than the expected cost in this case, the probability of realizing a favorable benefit-to-cost ratio (~0.2) does not assure a positive return on investment. Combined with other factors, such as affordability of the proposed action, its ability to meet risk reduction objectives, and stakeholder buy in, the probability of realizing a favorable benefit-cost ratio provides valuable information that supports the decision making process.

Further analysis can be conducted to determine the sensitivity of various model parameters on the final results (Modarres et al. 1999). The change in risk with respect to changes in model parameters yields insights into which aspects of a hurricane protection system offer the most potential for cost-effective risk reduction. Such parameters that would be considered include the height and hardness of the levees, probability of gates not being closed, etc. For example, if a sensitivity analysis showed that a small change in levee height leads to a significant reduction in risk, whereas a large change in closure probability offered only a small change in risk, the initial focus of risk reduction would be on increasing levee height. Moreover, sensitivity analysis would also reveal how the uncertainty in each parameter (e.g., storm surge heights, storm rates, fragilities, weir coefficient, etc.) contributes to the uncertainty in the final risk results; knowledge

Table 15. Subbasin Results for Storm 7, Stratification 7

			Percentile values for water elevation (ft)		
Subbasin	Branch	Probability	25%	50%	75%
SF1	10	0.3219	−0.71	−0.05	0.66
	12	0.1810	−0.35	0.28	0.75
	14	0.3181	11.25	11.50	11.75
	16	0.1790	11.25	11.50	11.75
SF2	10	0.3219	−1.08	−0.24	0.70
	12	0.1810	−0.95	−0.12	0.81
	14	0.3181	16.23	16.49	16.74
	16	0.1790	16.23	16.49	16.74
MF1	2	0.9912	−28.75	−28.50	−28.25
	6	0.0088	12.23	12.49	12.74
MF2	2	0.9485	3.72	4.54	5.40
	6	0.0515	24.25	24.50	24.75
WF1	14	0.6400	15.23	15.49	15.75
	16	0.3600	15.23	15.49	15.75
FN1	2	0.9292	−15.75	−15.50	−15.25
	6	0.0708	−4.31	5.22	13.31

Note: 1 ft=0.3048 m.

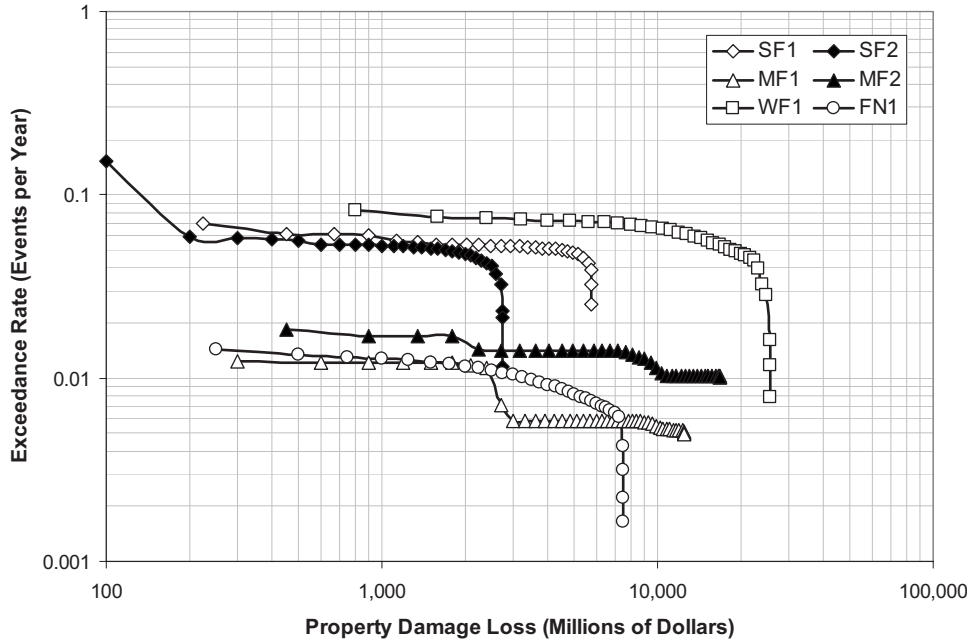

Fig. 9. Property damage loss-exceedance curves for FHPS subbasins

Fig. 10. Cumulative distribution on accumulated loss in single year for FHPS subbasins

of uncertainty importance would lend insight into where to focus resources so as to better understand and control sources of uncertainty.

Conclusions and Recommendations

This paper presents a case study of a simple, notional city to demonstrate the risk analysis methodology for protected hurricane-prone regions as described by Ayyub et al. (2009). This paper demonstrates that the results from the risk model can be used to inform resource allocation decisions for cost-effective risk mitigation and disaster recovery. The implementation of the risk model is packaged as the Flood Risk Analysis for Tropical Storm Environments (FoRTE) tool currently in use by the U.S. Army Corps of Engineering Interagency Performance Evaluation Team (IPET) charged with assessing the risks to New Orleans due to hurricanes (USACE 2006).

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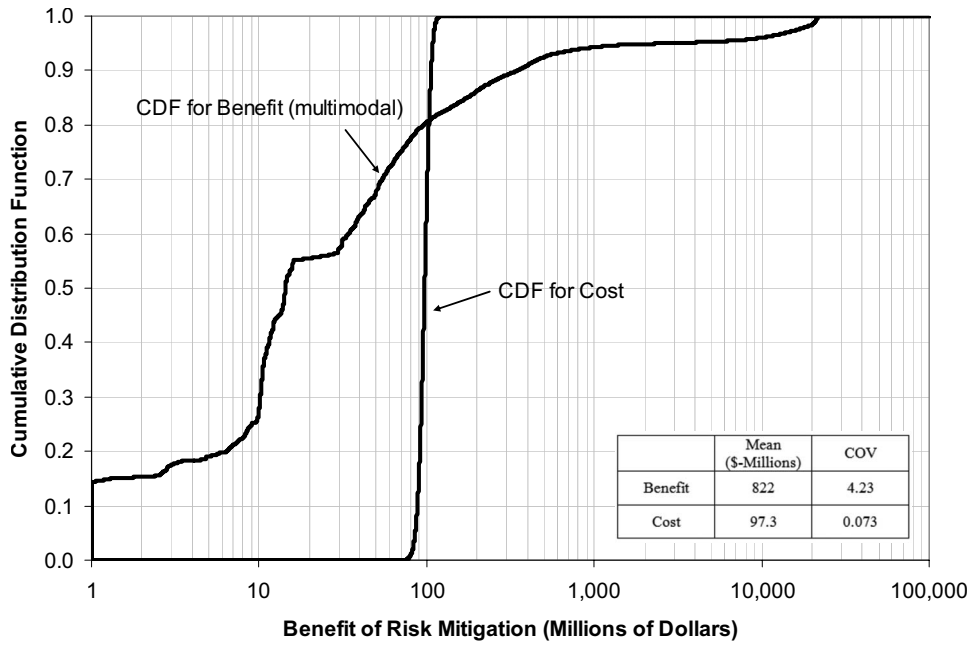

Fig. 11. Cumulative distribution function for benefit computed as risk reduction

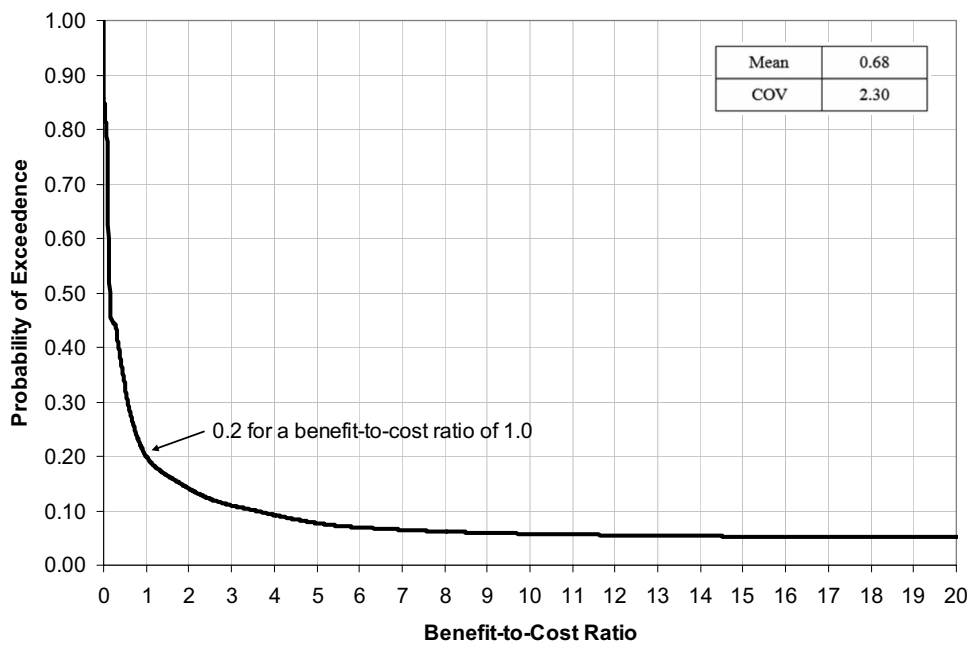

Fig. 12. Exceedence probability for various benefit-to-cost ratios

David Bowles, Jennifer Chowning, Robert Dean, David Divoky, Bruce Ellingwood, Richalie Griffith, Mark Kaminskiy, Burton Kemp, Fred Krimgold, Therese McAllister, Martin W. McCann, Robert Patev, David Schaaf, Terry Sullivan, Pat Taylor, Nancy Towne, Daniele Veneziano, Gregory Walker, Mathew Watts, and Allyson Windham, and the contract administration and support provided by the U.S. Army Corps of Engineers and the help of Mr. Andy Harkness. Information provided in the paper is personal opinions of the writers, and does not represent the opinions or positions of other entities including the U.S. Army Corps of Engineers.

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