APPENDIX C
RISK ASSESSMENT

<u>PART I</u>
RISK ASSESSMENT
TRINITY RIVER CORRIDOR

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TRINITY RIVER CORRIDOR

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TRINITY RIVER CORRIDOR

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Risk Assessment

Trinity River Corridor Dallas Floodway near Dallas, TX

US Army Corps of Engineers®

7 September 2012

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Executive Summary

From September 2011 through January 2012, the Corps of Engineers and the City of Dallas evaluated the risks posed by the levee system that protects Dallas from flooding on the Trinity River. The team was composed of individuals from the Fort Worth District, St. Louis District, St. Paul District, Tulsa District, Risk Management Center, HQUSACE, the City of Dallas, and HNTB.

The team thoroughly examined the large amount of information available for the site and used this information to evaluate the risks posed by the system. The team concluded that the highest risks posed by the levee system are from overtopping and breach of both the East and West levee systems. Overtopping followed by breaching would cause flooding that is significantly more than overtopping where the levee does not breach. There is also a possibility that risks from internal erosion are also high. Although the perception prior to the risk assessment was that the existing system had high risk due to internal erosion, heave, and stability, this is not the case. These failure modes were determined to have fairly low risks primarily due to two factors. First, large floods on the Trinity River in the Dallas Floodway that would raise the river elevation near the crest of the levee system are of relatively short duration. Second, although the recurrence of floods that would raise the river elevation to ½-height of the system is frequent (on the order of 1 in 300 years) the recurrence of these large floods is very infrequent (on the order of 1 in 4,000 years).

The levees themselves are composed of compacted low-to-high plasticity clays. There are sections of the system that have a basal sand foundation layer of varying thicknesses that sits on top of bedrock that potentially runs from the river side to the protected side. The team estimated risks from potential failure due to internal erosion, heave followed by internal erosion, and global slope stability. The combination of low frequency of loading, short duration of loading, and the ability of the system to resist those loads led the team to conclude that risks were tolerable for these failure modes.

The estimated durations of large floods where river elevations would exceed ½-height of the levees are no more than two weeks. Even considering the desiccation cracking that occurs in the system, the duration of loading is likely not sufficient to saturate the levee system enough to cause effective strengths to reduce far enough to lead to global slope failure. The gradients induced in the basal sand layer are likely not sufficient to cause internal erosion to progress beneath the levees. Although there may be sand boils during large floods, these boils are not likely to progress beneath the levee leading to an internal erosion failure.

The levee system protects a large population and a significant amount of infrastructure. Although the consequences of failure are high, the City of Dallas has a robust Emergency Action Plan (EAP) that minimizes the potential life safety consequences.

The team developed four recommendations:

1. Life safety risks for overtopping of the East and West Levee systems exceed Tolerable Risk Guidelines. Alternatives to reduce these risks should be explored.
2. The team believes that the way USACE and the City of Dallas have approached managing the system is the most prudent way to proceed in the future, as other steps that would need to be considered to eliminate performance uncertainty would be so expensive that they would outweigh the benefits currently provided by the system.
3. The risk assessment used seepage and stability models that depended on our ability to model the situation adequately. The team believes instrumentation options should be explored to be able to confirm those assumptions in critical areas during flood events.
4. The sewage outfall tunnel situation warrants close attention and the investigations related to that collapse should be incorporated as an addendum to this risk assessment if the findings are significant.

Introduction

Risk Assessment

This risk assessment is a beta test of a proposed procedure for evaluating levee risk in more detail than the levee screening currently done by the Corps. The risk assessment is intended to verify the risk factors identified by the Levee Screening Tool (LST), refine the priority of the project assessments, and inform the upcoming feasibility study.

This risk assessment for the Dallas Floodway evaluates the risks as they exist at the time of the risk assessment meeting. Base conditions include only measures taken during floods that are in accordance with normal operation.

Project Authorization

Location and Owner

The Dallas Floodway Project is a federally authorized and non-federally operated and maintained, urban flood protection project. As shown in Figure 1, the Dallas Floodway Project is located on the right (East) and left (West) banks of the Trinity River in Metropolitan Dallas.

Background

Project Description

The Dallas Floodway project consists of a complex system that includes levee embankments, a concrete floodwall, sumps and pumping stations, bridge crossings, conduits, and other penetrations. Only a brief description is included here. More details are described in subsequent sections dealing with potential failure modes and risks. The Dallas Floodway project is located on the Elm Fork, West Fork and Trinity River in Dallas, Texas. The project includes 22.6 miles of levee embankments: 11.7 miles on the northeast levee (usually referred to as the East levee) and 10.9 miles along the southwest levee (generally referred to as the West levee). The East levee protects the Stemmons Corridor (a major transportation route through the City), and parts of Downtown Dallas and the Central Business District from flooding on the Trinity River, while the West levee protects a large portion of West Dallas (largely residential areas). These embankments were originally constructed by the City of Dallas and the Dallas County Levee Improvement District in the 1930's in response to extreme flooding along the Trinity River in 1908. Originally constructed with 2.5H:1V side slopes, a maximum height of 35 feet and a crest width of 6 feet, the levee system was "strengthened" by USACE in the late 1950s by flattening the side slopes and increasing the crest width to 16 feet in a river-side shift. There are several pump stations on both the East and West Levees. These pump stations have low areas near them where water collects on the land side to be pumped out. These low areas are referred to as "sumps". The levee embankments are generally comprised of low plasticity clays and high plasticity clays founded on recent alluvial soils. The concrete floodwall is located on the downstream end of the east levee, with a crest elevation generally a few feet lower than the embankment. The alluvial floodplain soils generally consist of alluvial clay deposits, underlain by sandier deposits (becoming coarser with depth), followed by basal sands and gravels. The basal sands and gravels mark the bedrock contact with the Eagle Ford Shale or the Austin Chalk.

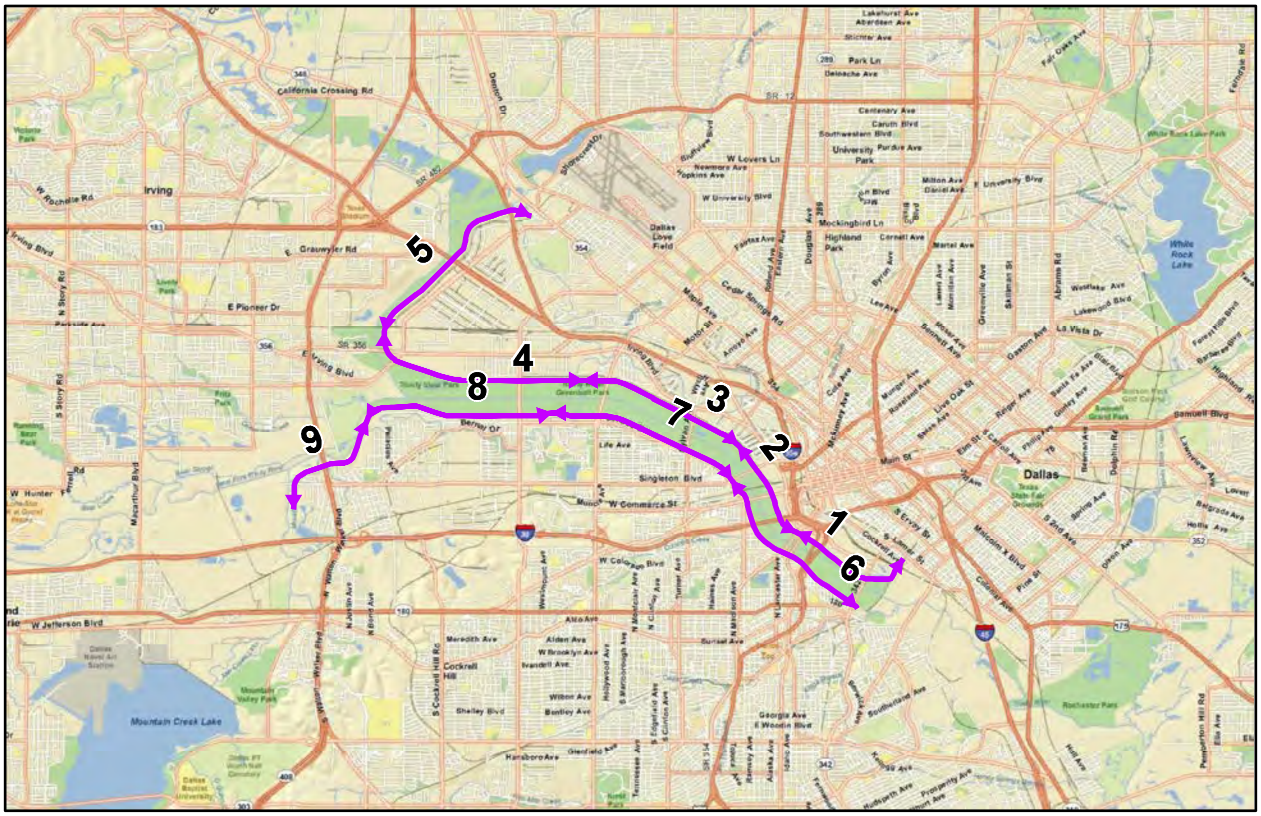


Figure 1 - East and West Levees that protect parts of Dallas shown in magenta.

Geomorphology

The geology and geomorphology of the Trinity River Basin has a significant influence in the selection and likelihood of the potential failure modes discussed at the Dallas Floodway Risk Assessment. Relevant issues relating to the geology and geomorphology include; what are the soil materials in the valley floor, what was their origin, how were they deposited and how do they impact the failure mode analysis.

The materials moving through the Trinity River Basin are sediments derived from the deposits of retreating glacial ice sheets and the erosion of bedrock formations like the Eagle Ford Shale. The interpolated terrain surface showed evidence for various paleochannels incised in the bedrock. The top of Eagle Ford Shale has an average depth of 47 feet below ground surface throughout the project area. The figure titled "Top of Eagle Ford Shale" shows the top of rock contours for the Eagle Ford Shale and the overlying Austin Chalk which occurs in the downstream reach of the floodway. The Austin Chalk appears to be more resistant to the river erosion since the floodplain is narrower in the downstream reach. Most of the sediment in the Dallas Floodway river valley is presumed to be the glacially derived material that has been repeatedly carried and deposited in clay, silt, sand and gravel depositional sequences throughout this reach of the Trinity River.

The Trinity River fluvial system had an actively migrating, or meandering, main channel prior to the construction of the Dallas Floodway. Each time the river channel changed its course the material available from the previous river deposition was transported to a new location. The fluvial deposits created by a migrating channel range from coarse to fine grained depending on the velocity of a particular river segment. High velocity flows sort and deposit coarse grained materials and lower velocity flows deposit correspondingly finer grained materials.

The constant shifting of the channel location and subsequent variation in velocity zones re-deposited the different grain sizes in a lateral disbursement as well as, various changing vertical sequences. As the channel moved back and forth over a set location in the river bed, repetitive sequences of clay, silt, sand and gravel was deposited stratigraphically. The dynamics that influence the migration of the river channel can be a relatively slow response to constant tractive forces as gravity pulls the river and the accompanying sediment load through the basin, or a quick, catastrophic response to large flash floods. The slower, constant dynamic river system usually results in a gradual vertical change in grain size for a particular location, which is sometimes referred as a "fining upward" or "fining downward" depositional environment. The quick, catastrophic occurrence can abruptly truncate existing deposits by cutting through, thereby scouring the river bed, and rapidly re-depositing large volumes of differently graded materials.

Five major fluvial environments of deposition were found in the Dallas Floodway: (1) point bar; (2) backswamp; (3) abandoned channels; (4) abandoned courses; and (5) natural levee and crevasse splay.

Point bar deposits are commonly found in the floodway area. They consist of sediments laid down on the insides of river bends as the channel meanders back and forth across the valley floor. There are two basic types of deposits in point bar features: silty and sandy, elongate bar deposits or "ridges" which are laid down during high river stages, and silty and clayey deposits in arcuate depressions or "swales" which are laid down during falling river stages.

Backswamp deposits consist of fine-grained sediments laid down in broad shallow basins during river flood stages. The sediment laden floodwater may be ponded in low lying areas or between natural levee ridges, where the flow velocity is lower and the fines drop out.

Abandoned channels are partially or wholly filled segments of stream channels that were left in place when the river meandered and changed course. Initially they may be characterized as swales or contain water as an oxbow lake, subsequently becoming backfilled with river sediment.

Abandoned courses are lengthy channel segments of a river, abandoned when the stream forms a new course across the floodplain. They can vary in length from a few miles to tens of miles, often occupied by a smaller or "underfit" stream. The smaller stream can deposit new sediment and rework existing material as it meanders within the abandoned channel.

Natural levees are low ridges which flank both sides of a river. When a river overflows its banks coarser material is deposited adjacent to the channel and fine material further away. Small scale erosion rills can cut into the natural levees at right angles, flowing away from the main channel, and rising water can spill out of the main channel and create a crevasse splay deposit of river sediments.

A migrating river system like the Trinity River, with rapidly changing depositional environments, usually creates relatively smaller, irregularly shaped fluvial deposits in contrast to larger, widespread deposits in higher flow, entrenched river systems. The Dallas Floodway geomorphology seems to reflect a more chaotic depositional environment, given the wide variation in material samples found in the boring information. The samples indicated a suite of deposits ranging from fine-grained, low permeable clays and silts to permeable, clean sand and gravel. Most of the deposits were limited in lateral extent but often repeated sequentially and a degree of hydraulic conductivity was assumed for specific reaches.

Most of the information contained in this section was obtained from reports prepared by others (see below). The distribution of surficial geologic units defined in the study was determined from: aerial photos (vintage 1929, 1930s, and 1954); historic topographic maps from 1889, 1918, and 1954; and boring and CPT data provided by USACE, USGS, and Trinity River Project contractors (HNTB and Fugro).

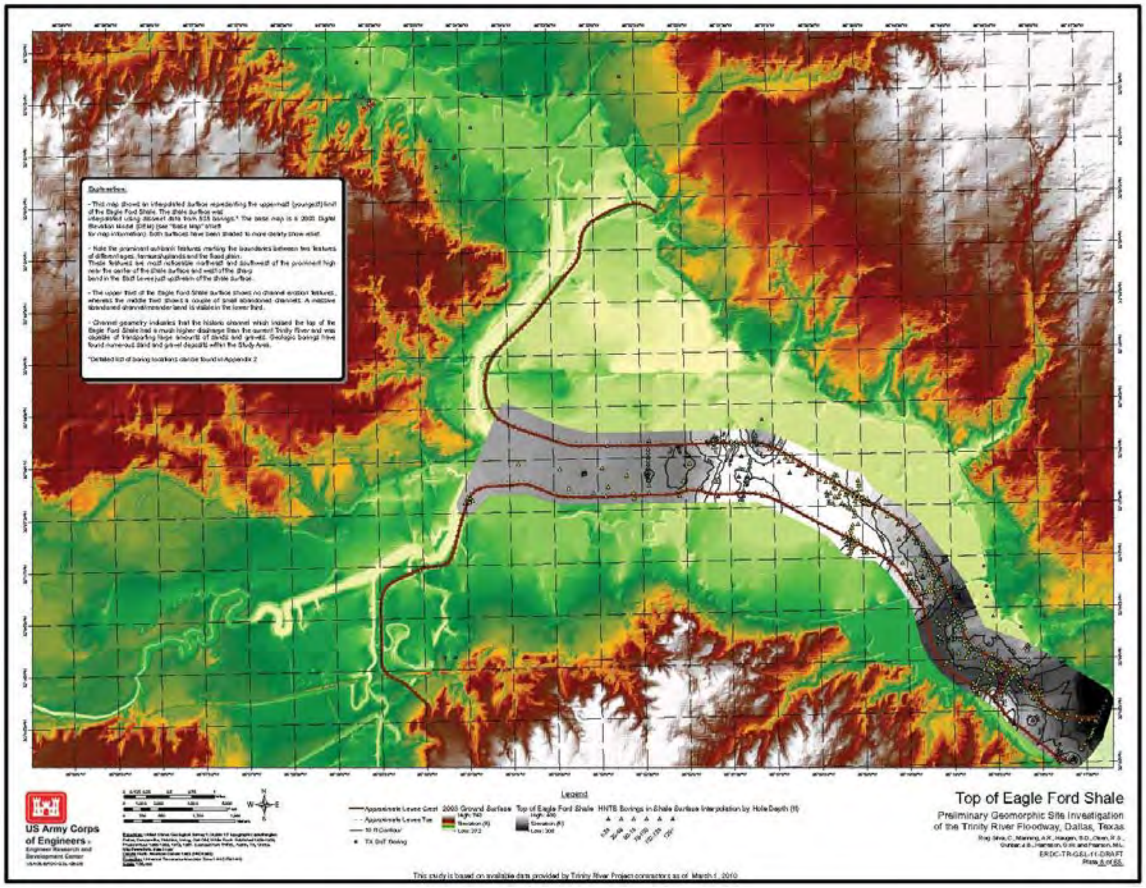


Figure 2 - Top of Bedrock Contours

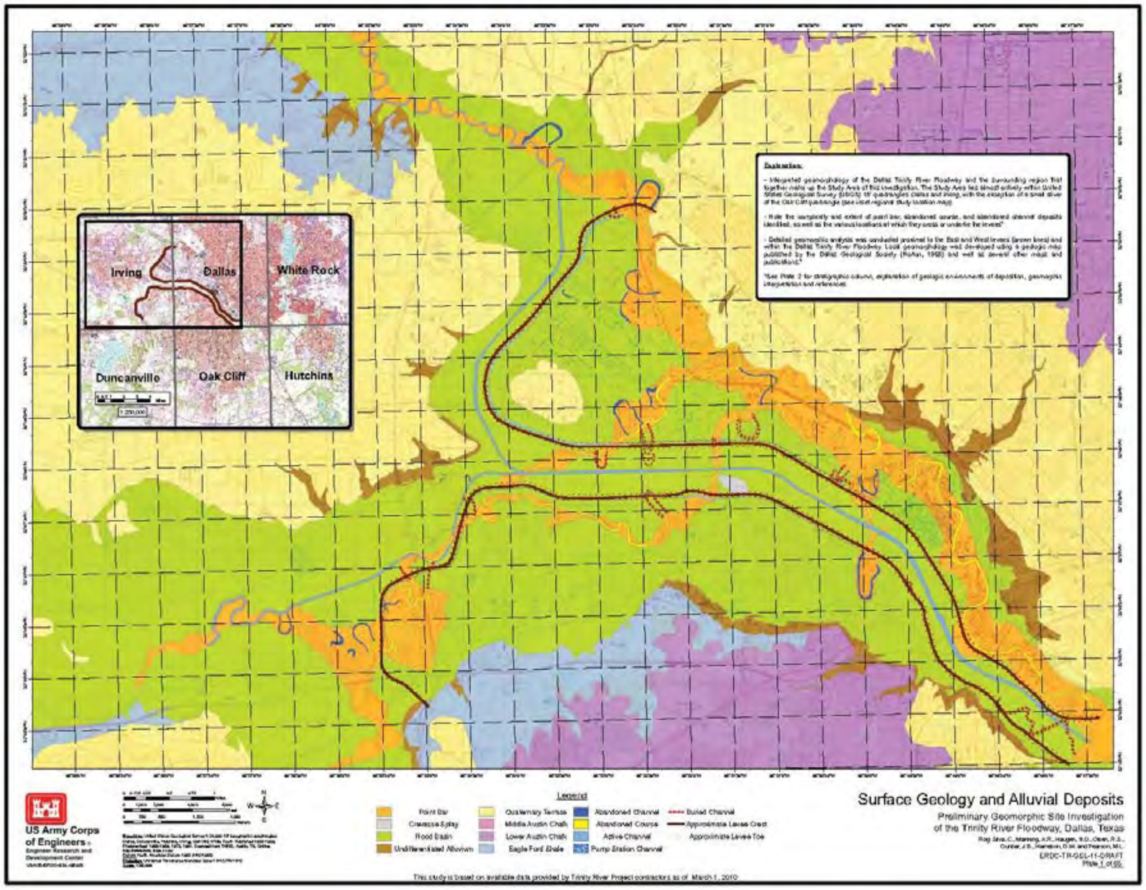


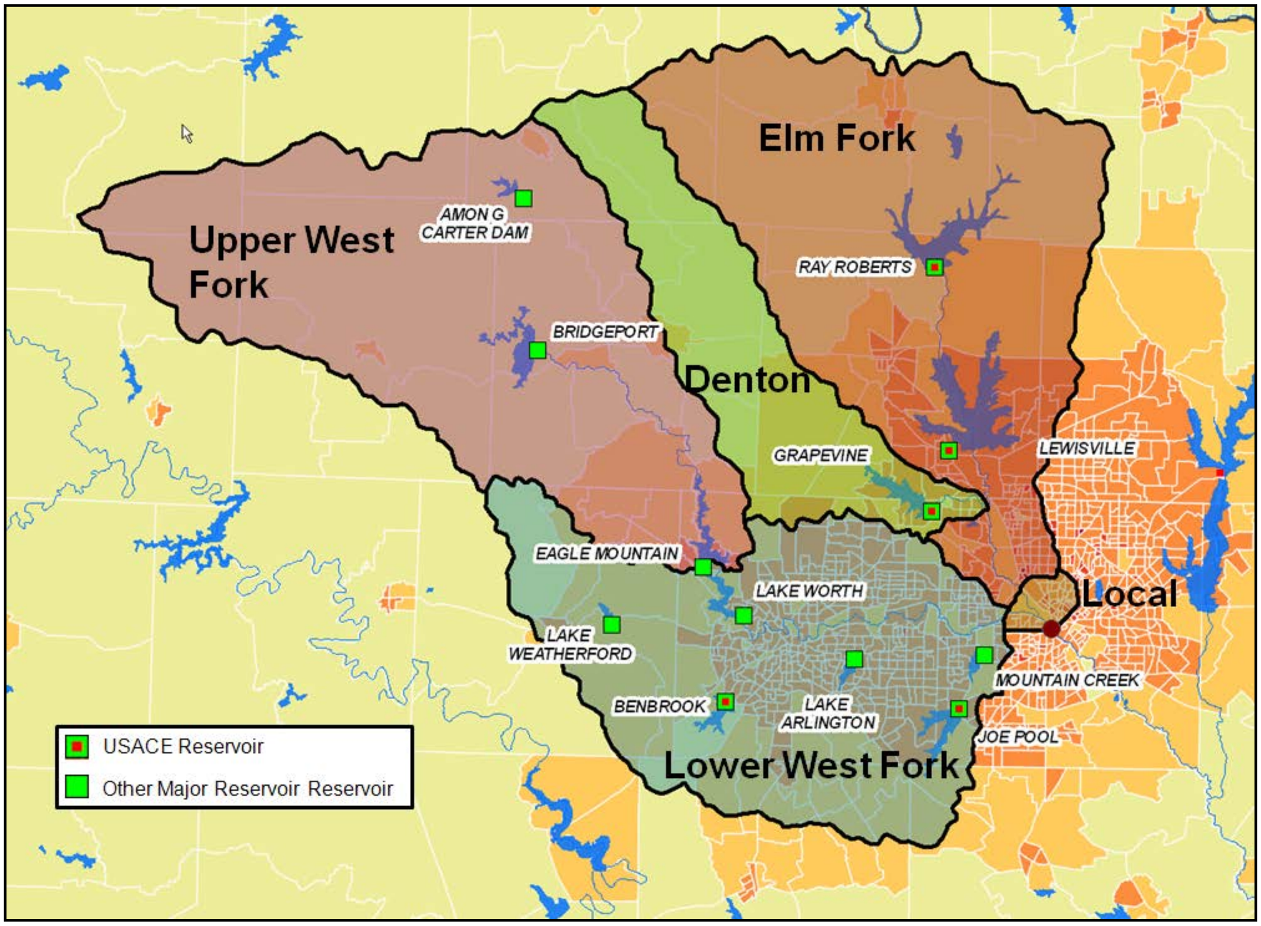
Figure 3 - Surface Geology

Hydrology

The drainage area of the Trinity River, from its headwaters to the confluence of Five Mile Creek, near the Interstate Highway 20 bridge in south Dallas, was evaluated during this study. This area, which is commonly referred to as the “Upper Trinity” watershed, covers about 6,275 square miles. It includes the majority of the Dallas-Fort Worth (DFW) Metropolex. Terrain in this watershed varies in elevation from about 1,200 feet National Geodetic Vertical Datum (NGVD) at the headwaters of the West Fork of the Trinity River just northeast of Olney, Texas, to about 380 feet NGVD at the confluence of Five Mile Creek.

Of the five US Army Corps of Engineers (USACE) flood control reservoirs in the study area, three (Lakes Benbrook, Lewisville, and Grapevine) were impounded in the early 1950's. Impoundments in the other two USACE reservoirs (Lakes Joe Pool and Ray Roberts) were initiated in January 1986 and June 1987, respectively. Additional major USACE flood control projects in the study area include the Fort Worth Floodway and Dallas Floodway levee/channel improvement systems.

The two largest non-Federal lakes in the study area, both of which are situated on the West Fork of the Trinity River, are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located just west of Bridgeport in Wise County. Eagle Mountain Lake is located in northwestern Tarrant County, just upstream from the much smaller Lake Worth, which is owned by the City of Fort Worth. Eagle Mountain Lake has two sets of outlet gates and an emergency spillway, but since it has no dedicated flood control storage, large releases are required during flooding periods. Smaller lakes within the Upper Trinity watershed include: Lake Amon Carter, located on Big Sandy Creek south of Bowie in southwestern Montague County; Lake Weatherford, located on the Clear Fork of the Trinity River northeast of Weatherford in Parker County; Lake Arlington, located on Village Creek in western Arlington in Tarrant County; and Mountain Creek Lake, located on its namesake in Grand Prairie in western Dallas County.



Reservoir	Year Completed	Normal Storage (Acre-ft)	NLD ID
Lake Worth	1914	38130	TX00785
Bridgeport	1931	386539	TX01496
Eagle mountain	1932	190460	TX00779
Mountain Creek	1937	40000	TX00827
Benbrook	1951	88250	TX00003
Grapevine	1952	188550	TX00005
Lake Arlington	1955	38785	TX00776

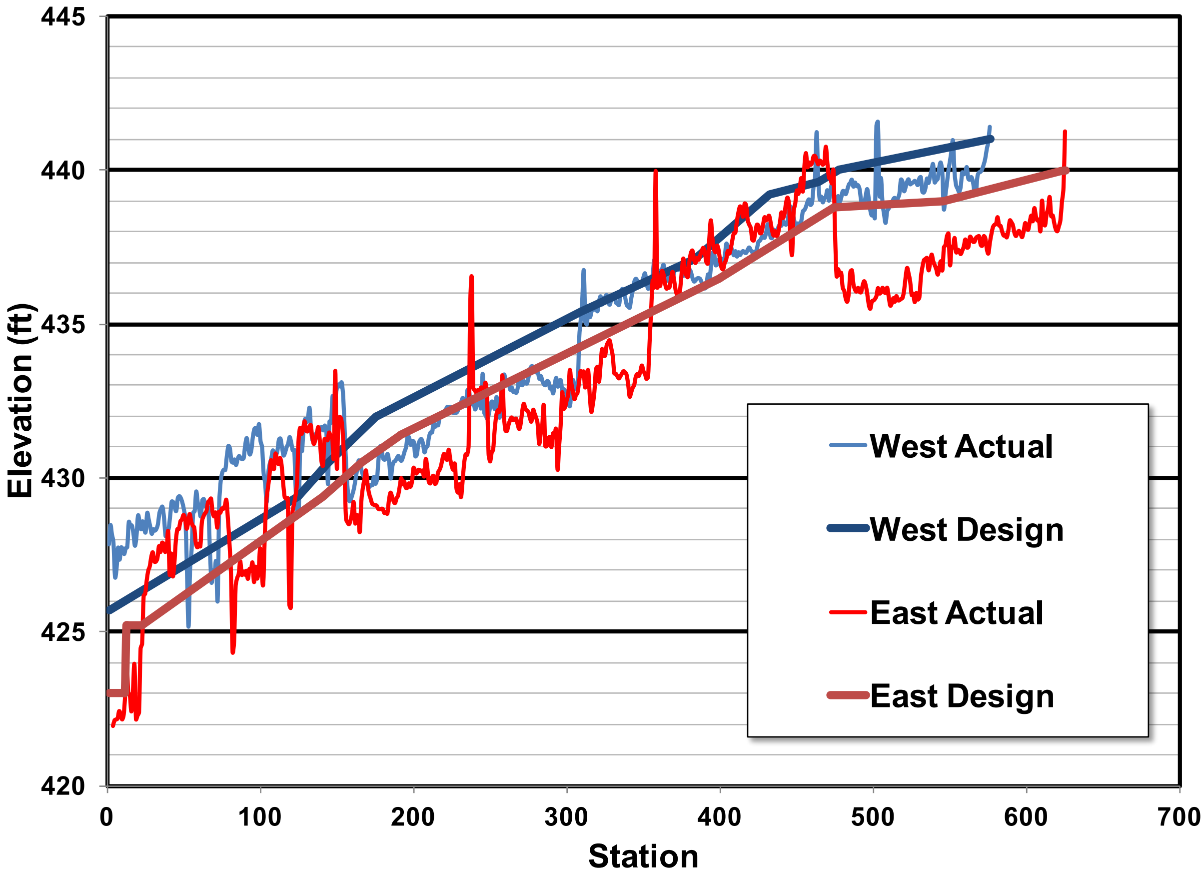
Reservoir	Year Completed	Normal Storage (Acre-ft)	NLD ID
Lewisville	1955	618400	TX00008
Lake Amon Carter	1956	20050	TX00699
Lake Weatherford	1957	19866	TX01222
Joe Pool	1986	176900	TX08009
Ray Roberts	1987	799600	TX08010

The climate in the Upper Trinity watershed is humid subtropical with hot summers and mild winters. Snowfall and subfreezing temperatures are experienced occasionally during the winter season. Generally, the winter temperatures are mild with occasional cold periods of short duration resulting from the rapid movement of cold pressure air masses from the northwestern polar regions and the continental western highlands.

Recorded temperatures at the DFW International Airport have ranged from a high of 113°F in June 1980 to a low of -1°F in December 1989. The average annual temperature over the watershed varies from 64°F at Bridgeport in the northwestern extremity of the watershed to 66°F at DFW International Airport. The mean annual relative humidity for the DFW Metropolis is about 65 percent. The average annual precipitation over the watershed varies from about 30 inches at Jacksboro, in the northwestern extremity of the watershed, to about 32 inches in the DFW Metropolex. The extreme annual precipitation amounts since 1887 include a maximum of 53.54 inches in 1991 at the DFW International Airport and a minimum of 17.91 inches in 1921 at Fort Worth. The maximum recorded precipitation in a 24 hour period was 9.57 inches, at Fort Worth on the 4th and 5th of September 1932. A large part of the annual precipitation results from thunderstorm activity, with occasional very heavy rainfall over brief periods of time. Thunderstorms occur throughout the year, but are more frequent in the late spring and early summer. The average length of the warm season (freeze-free period) in the DFW Metropolex is about 249 days, extending from mid-March to mid-November.

The largest historic flood event on Trinity River at Dallas was in 1908, prior to the construction of major flood storage reservoirs in the basin, when gage reached a peak stage of 52.6’ and had an estimated flow of 184,000 cfs. Since the construction of the flood storage reservoirs, the maximum observed stage was in 1990 and had a peak stage of 47.1’ and an estimated peak flow of 82,300 cfs.

The Dallas Floodway System was designed and built in the 1950s and used the Standard Project Flood (SPF) of 226,000 cfs as the basis for design. The levees in throughout the system are typically about 30 feet high from natural ground elevation to levee crest. Profiles of the levee crest elevations on the east and west levee segments are shown in the figure below.



Description of Study
HQUSACE and the Southwest Division requested this study. The purpose of the study is to quantify and evaluate risks posed by the East and West Levee systems from flooding associated with the Trinity River.

Methodology
Risks were estimated for this assessment using the Best Practices in Dam Safety Risk Analysis[1] which is a joint methodology developed by the Corps of Engineers and the Bureau of Reclamation. Estimates of levee failure risk require quantifying the likelihood of loads, the structural responses given the load and the adverse consequences given a failure occurs as well as the uncertainties associated with each. The estimation process relies on engineering techniques whose applications differ little in principle from traditional deterministic safety assessments. The difference between risk analysis and traditional engineering is quantifying the uncertainties in all of their various forms. Probabilistic methods inherently address these uncertainties.

[1] Best Practices in Dam Safety Risk Analysis, Version 2.2, April 2011. U.S. Department of the Interior, Bureau of Reclamation and the U.S. Army Corps of Engineers.

The risk was estimated in terms of Annualized Loss of Life (ALOL) and Annualized Probability of Failure (APF), which include uncertainty in their estimates for each node of the decomposed structural response indicated by a range on their estimates. Most likely probabilities were elicited from the team, and the range of estimates was used to create a distribution that represented the uncertainty.

The program @RISK was used to perform the computation for the probability of failure and to compute the ALOL. The computer program @RISK uses a simulation called a Monte Carlo analyses – in this case using Latin Hypercube sampling – generated by the software to simulate the range of distributions and results from each branch of the event tree. These numeric values represent the expected range of risk estimated for the probability of failure and ALOL.

The failure modes were decomposed to develop detailed event trees and probability estimates. The probabilities are based on the estimates given by each team member on a scale that ranges from virtually impossible (0.001) to virtually certain (0.999), as given in the table of Verbal Descriptors in the Best Practices in Dam Safety Risk Analysis. The verbal descriptors are shown in Table 1 below.

Table 1 - Table of Verbal Descriptors

Descriptor	Probability
Virtually Certain	0.999
Very Likely	0.99
Likely	0.9
Equally Likely	0.5
Unlikely	0.1
Very Unlikely	0.01
Virtually Impossible	0.001

Team members used the probabilities from Table 1 as anchors and were allowed to estimate between these probability ranges. Individuals were asked to write down their estimates, and then these results were tallied. If the range was small, the team continued to the next node. If there were significant variances, the team discussed the reasons for those variances and attempted to characterize those differences either numerically or qualitatively.

APF and ALOL plots were developed using the estimated probabilities in event trees developed for each failure mode.

Participants

This risk assessment for the Dallas Floodway was performed in three phases. The first phase was the Potential Failure Mode Analysis (PFMA). The potential failure mode analysis was performed October 31 through November 3, 2011 in Dallas, Texas. Appendix G – Participants contains a participant list from each meeting. It should be noted that operations staff from the City of Dallas were present. Their participation was critical to understanding the potential vulnerabilities of the system. Technical staff from the city's consultant, HNTB and from ERDC and the Fort Worth District of the Corps of Engineers also provided valuable information from studies that have been performed for the project. The PFMA was facilitated by Nathan Snorteland and Gregg Scott of the Risk Management Center. For this phase of the evaluation, it was not necessary to limit participation to the risk assessment team, and everyone in attendance was invited to have input. In the end there seemed to be general agreement regarding the results of the evaluation by all present.

The second phase of the evaluation, performed the week of December 12, 2011 and the week of January 9, 2012 (again in Dallas) involved development of event trees and risk estimates based on additional studies performed since the PFMA.

Potential Failure Mode Analysis

Arguably, the most important part of a risk assessment is identifying and describing the most likely potential failure modes based on the perceived vulnerabilities of the project. If this is not done well, then the results of a risk assessment will be of limited value, or even potentially misleading.

Procedures

Prior to the PFMA meeting, collections of reports and drawings were distributed to the participants for review. A half-day field review of the project was conducted the first morning of the PFMA meeting. Although it was not possible to examine the entire levee system in detail, several key areas were observed and an overall impression of the system was obtained. Due to the sheer volume of material that needed to be covered, several technical presentations were made to the group by those most familiar with the project prior to beginning the PFMA exercise.

After the familiarization process, potential failure modes were "brainstormed" based on the group's understanding of the vulnerabilities of the levee system. Anyone was allowed to propose a potential failure mode. These were captured in rough form without significant discussion or development. Then the list was reviewed and several potential failure modes were ruled out as being obviously extremely unlikely. These are summarized later along with the reasoning behind ruling them out. The remaining potential failure modes were then evaluated in more detail.

The first step in evaluating a potential failure mode was to describe it fully from initiation, through progression, to breach and flooding of the protected side. This was necessary to assure everyone in the room had a common understanding of what was being discussed, and that those picking up this report in the future would have an understanding of what the team was thinking.

After a potential failure mode was thoroughly described, factors were identified and captured which made the mode "more likely" to develop (adverse factors) and "less likely" to develop (favorable factors). Anyone was allowed to propose factors for consideration in evaluating the potential failure mode. However, the most significant factors were identified for classifying the potential failure modes.

Risk is composed of two components: (1) the likelihood of failure and (2) the consequences should failure occur. Therefore, after all adverse and favorable factors had been captured each potential failure mode was classified according to risk ranking descriptors described below. Both a likelihood descriptor and a consequence descriptor were assigned to each developed potential failure mode. This allowed the results to be portrayed on the risk matrix shown in Figure 4, where risk increases going diagonally from the lower left corner to the upper right corner. In addition, a confidence rating was assigned to each category. When low confidence was assigned, additional information that could help bolster the confidence was identified. The descriptors and matrix are strictly relative ranking tools that allowed the team to identify those potential failure modes that would be the largest contributors to the risk posed by the project.

Failure Likelihood Descriptors

- **Very Low –** Failure is unlikely up to and including the Standard Project Flood (1/1500)
- **Low –** Failure is unlikely for the 1/100 flood and uncertain for the Standard Project Flood
- **Moderate –** Failure is unlikely at the 1/100 flood but likely for the Standard Project Flood
- **High –** Failure is uncertain at the 1/100 flood but likely for the Standard Project Flood
- **Very High –** Failure is likely at the 1/100 or more frequent flood

Consequence Descriptors

- **Level 0 (No Hazard) –** No significant impacts to the protected population other than temporary minor flooding of roads or unoccupied lands
- **Level 1 (Low Hazard) –** Discharge results in minor property damage but no direct life loss is expected
- **Level 2 (Significant Hazard) –** Discharge results in moderate property damage; direct life loss less than 10 is expected
- **Level 3 (High Hazard) –** Discharge results in extensive property damage; direct loss of life loss up to 100 is expected
- **Level 4 (Very High Hazard) –** Discharge results in severe property damage; direct life loss exceeding 100 is expected

Confidence Categories

- **High** – Confidence in the rating is high; it is unlikely that additional information would change the rating.
- **Low –** Confidence in the rating is low; additional information could very well result in a change to the rating.
- **Moderate** – In between High and Low.

Table 2 - Screening Matrix for Failure Modes

FAILURE LIKELIHOOD	CONSEQUENCES OF FAILURE				
	LEVEL 0	LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4
VERY HIGH					
HIGH					
MODERATE					
LOW					
VERY LOW					

After the discussion of favorable and adverse factors for each potential failure mode, individuals were asked to select and write down their estimate of the likelihood category, the consequence category, and confidence in each. Due to the large size of the group, it was queried by a show of hands. Typically, the majority of the participants fell into one category, with a few on either side. Those higher and lower than the majority were asked for their reasoning. The group was asked whether anyone wanted to change their estimate. If so, then the group was queried again to see if the majority had changed, although typically this did not happen. The majority estimate was captured along with the confidence and rationale. The group was queried for any strong objections although typically none were voiced. In several cases it was noted that some individuals had estimated higher likelihood categories but lower consequence categories than others, and vice versa, such that the estimates of “risk” tended to be more consistent than the individual category estimates.

Potential Failure Modes

The potential failure modes identified and analyzed during the potential failure mode analysis are summarized below. Following each heading is a complete description of the potential failure mode, the adverse and favorable factors identified during the session, the likelihood and consequences categories selected, and the rationale and confidence in each.

1. *Scour around a bridge pier leading to slope instability*

A flood higher than the largest historical event occurs. Scour begins in the embankment at the bridge pier. Enough material is removed causing the slope of the embankment to degrade. The degraded slope begins to progressively slough and wash away on the water side. Enough material is removed until the crest is breached leading to overtopping and erosion to the base of the levee. In areas where the embankment is clayey, breach progression would be slow. In areas where the embankment is sandy, the breach could progress somewhat more rapidly. Any breach would be localized to an area adjacent to the bridge.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Scour Around Bridge Pier Leading to Progressive Slope Instability
Location:	Bridge Pier
Event and Initiator:	Flood Greater Than Historical Maximum

Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Localized turbulence could create erosion	CH2M Hill scour study to SPF indicated low velocity and no signification problems, (but study not provided to group)
Clay has desiccation cracks – could be “nick point” for erosion if near bridge piers	River side water load has stabilizing effect
Not much grass slope cover under bridges to provide erosion protection	In general, high clay content near bridges which is more erosion resistant
	Desiccation cracks not as open certain times a year
	Good access to area for flood fighting since bridges are typically connected to roadway
	Sponsor (City of Dallas) is proactive in dealing with erosion issues – repairs are possible during flooding
	Many of these areas have some form of scour protection near bridge piers (e.g. rip rap or concrete paving)
	Bridge may shade/protect embankment from severe drying which reduces the tendency for desiccation cracks

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Scour Around Bridge Pier Leading to Progressive Slope Instability
Location:	Bridge Pier
Event and Initiator:	Flood Greater Than Historical Maximum
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
	Water may recede quickly from upper bridge piers

Likelihood Category: Low **Confidence**: Moderate

Rationale: The group put a lot of stock in the reported CH2M Hill study results (even though very few had seen the actual report) which indicated little tendency for this type of erosion even at high stage levels. The main area of uncertainty related to likelihood of this potential failure mode involves unknowns about the presence and erodibility of sandy materials near the bridge piers, though it is unlikely that highly erodible sands exist in the levees at these locations.

Consequence Category: Level 2 **Confidence**: Moderate

Rationale: The breach is likely to be localized and slow to develop with good access for evacuation. The primary uncertainty stems from the fact that no breach studies have been performed for this type of potential failure mode.

2. *Overtopping and breach of a levee*

A flood high enough to overtop the levee embankment occurs. Overtopping occurs for a duration sufficient to begin erosion of the embankment. Erosion continues and progressively enlarges to a large breach of the levee crest and the embankment erodes down to the foundation level.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Overtopping Erosion of the Levee
Location:	Low Areas based on Survey Results
Event and Initiator:	Very Large Flood with Possible Debris Blockage at Bridges
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Expect there to be more debris at large flood flows than has been seen in the past	Needs close to SPF to trigger (overtop) without debris blockage
Trestle bridge has closely spaced supports which are more likely to catch debris	Except for trestle bridge, bridge piers are typically widely spaced
Bridge decks may catch debris at high flow since they are typically close to the levee crest	Backwater at bridges due to debris would be of limited extent upstream

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Overtopping Erosion of the Levee
Location:	Low Areas based on Survey Results
Event and Initiator:	Very Large Flood with Possible Debris Blockage at Bridges

Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Some areas of the levee would overtop at SPF without debris blockage by up to 1 to 2 feet	Small area near DART line most susceptible (lowest crest), could be sand bagged (1,000 to 2,000 feet)
Largest peak storm is a flashy local thunderstorm occurring between the upstream reservoirs and the levee – may not have much time to react	Could attempt to deal with debris at bridges using backhoes or other equipment
Local inundation of the exit roadways may hinder evacuation	Fairly confident in hydraulic model and predicted water surface profile, so should have relatively good idea when overtopping will occur (with no debris)
Vulnerable population (hospitals, nursing homes, etc.) may need assistance to evacuate	Short distance to safety – the inundated areas will be relatively close to the river, evacuation to upper floors of buildings possible
	EAP would likely be initiated for event like this which would lead to early evacuations
	Short duration of overtopping may not breach levee – hydrographs indicate peak flows may not be long duration
	The CH soils have low erodibility

Likelihood Category: Low to Moderate **Confidence**: Moderate
Rationale: Although it is likely the levee embankments would overtop during a flood equal to the Standard Project Flood (SPF) or greater, the compacted clay soils of the embankments will likely survive some level of overtopping without breach. The main uncertainty had to do with the possible duration of overtopping at large floods similar to the SPF that would overtop the dam.

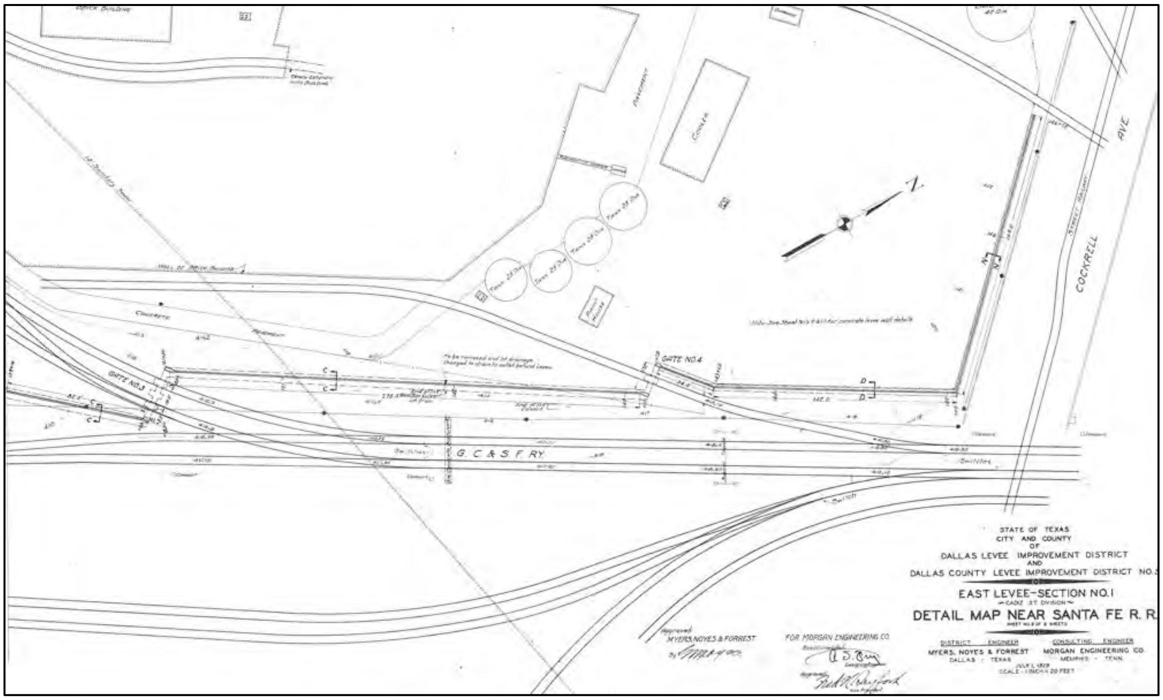
Consequences Category: Level 3 **Confidence**: Moderate
Rationale: If the levee fails during overtopping in a major storm it is expected that the flooding on the land side would be severe, and might occur in more than one location. The main uncertainty stems from how effective evacuations would be, and how quickly and deeply areas would flood since breach inundation studies have not yet been performed

3. Failure of a flood wall

A very large flood occurs and loads the concrete flood wall to a high level. The high water level either fails the wall by moment or shear, or overtopping erodes and undermines the wall. The wall collapses or is undermined and breaches leading to an uncontrolled inundation. The wall collapse spreads laterally inundating the sumps and adjacent areas.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Failure of Floodwall by Moment/Shear or Overtopping
Location:	Concrete Floodwall
Event and Initiator:	Very Large Flood

Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Nappe at 1’ depth of overtopping impacts beyond the concrete footing leading to possibility of eroding foundation soils	Stop log house built against wall on landside will buttress the wall in this location
Overtopping flows will plunge onto foundation imparting erosive forces to soil	The wall is keyed into the footing to help resist shear.
Rebar could be deteriorating (corroding) within the wall (but no evidence of this)	A large stretch of the floodwall is buttressed by a parking lot on the land side
	Reinforcing steel was placed on the upstream face and tied into the footing to resist moment
	Joint seals have been repaired which helps keep water from squirting through the joints and eroding the foundation of the wall
	Could drop rip rap on landside in locations of overtopping erosion, if materials and equipment were available
	If pool develops on landside of wall it could dissipate energy and prevent undermining



Likelihood Category: Moderate **Confidence**: Moderate

Rationale: Foundation erosion and undermining was thought to be the most likely mode of failure given the short wall height, and it is expected that erosion would occur if the wall is overtopped. However, the main uncertainty had to do with how fast the erosion might occur and how far it might progress under the wall footing.

Consequence Category: Level 2 **Confidence**: Moderate

Rationale: The wall is only about 7 feet high and the sump area will be inundated first then spreading out into an industrial area. The uncertainty stems from the fact that no breach inundation studies have been performed for this area.

4. Failure of the closure structures

A large flood occurs raising the river to unprecedented levels. The closure structure, consisting of stacked soil filled “Hesco Baskets” overtops and fails or collapses under the increased loading. Some down-cutting and lateral erosion into the foundation soils may occur. The baskets are already in place for the abandoned spur line, and would need to be installed in the main railroad closure. Materials are available and plans are in place to construct the closure, so the team thought the chances of not getting the closure installed are minimal.

Event Information	
Loading Condition:	Hydrologic

Failure Mode:	Failure of Closure Structure
Location:	Railroad Closure Sections
Event and Initiator:	Flood to Level of Closure
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Some failures noted of this type of structure at other projects when on soft surface	Closures are proven technology that has been used successfully at other projects – design of the closures has been reviewed and approved by USACE.
	Short closure, approximately seven feet high, two basket across base, width similar to height – these factors should provide stable structure
	Top elevation of the stacked baskets is higher than adjacent wall – baskets should not overtop first

Likelihood Category: Very Low to Low **Confidence**: Moderate

Rationale: The closures are well planned and use proven technology. The only uncertainty is possible performance if the foundation conditions prove to be very soft.

Consequence Category: Level 1 to Level 2 **Confidence**: Low to Moderate

Rationale: There is limited area for flow to occur through these openings in the concrete wall. However, there is uncertainty as to how much foundation and lateral erosion might occur.

5. *Scour through desiccation cracking in the crest*

A large flood occurs causing the water elevation on the river side to intercept existing transverse desiccation cracks in the crest of the embankment. Water begins to flow through these cracks with enough velocity to begin to scour the embankment materials adjacent to the cracks. Erosion progresses and expands the opening leading to downcutting and breach of the embankment.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Scour Through Desiccation Cracks in the Crest
Location:	High Liquid Limit CH Material Near Embankment Crest
Event and Initiator:	Flood Near Crest Elevation
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Desiccation cracking observed in the	Traverse cracks continuous upstream to

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Scour Through Desiccation Cracks in the Crest
Location:	High Liquid Limit CH Material Near Embankment Crest
Event and Initiator:	Flood Near Crest Elevation
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
embankment.	downstream have not been observed on the levees
Gravel surface may mask traverse cracks at crest	Gravel surface may mask or reduce cracking by providing some protection of the underlying soils from drying
Water has not been high enough against the slopes of the levees to enter some of the observed desiccation cracks – it is uncertain if they connect in through-going fashion	Most areas of cracking highly plastic, erosion resistant
Bad incidents have occurred in Australia and Arizona due to desiccation cracking in the crest of embankment dams	Flood fighting is aided because distresses can be observed
	Cracks may swell shut once they are exposed to water from the river. Desiccation cracking observed in dry season, during rainy season they tend to close up
	Cracks traverse to crest are likely not deep (less than five feet) – observed cracking is longitudinal or occurs down on the embankment slope
	Water would need to be near crest of levee to provide enough water and head to drive failure mode
	Possibly short duration of loading of water sufficiently high to enter and erode the cracks, particularly for local thunderstorm loading

Longitudinal cracking in the crest of the embankment.

Depth of longitudinal cracking in the crest of the embankment.

Likelihood Category: Very Low to Low **Confidence**: Moderate
Rationale: The clays are erosion resistant and no continuous open transverse cracks have been observed. The main uncertainty relates to the potential presence of unobserved cracks.

Consequence Category: Level 2 to Level 3 **Confidence**: Low
Rationale: The clay material should be erosion resistant and there should be time to react if the condition is noted, access is passable, and resources are available. However, if it is not noticed in time, the breach flows may surprise the protected population since breach would occur prior to overtopping.

6. Internal erosion through a levee

A large flood raises the water on the river side of the levee to higher than historical levels and causes high gradients between the land and water side of the levee. There are pervious interconnected sand lenses in the embankment. The hydraulic forces overcome the capacity of the material in these lenses and water begins to exit the land side face of the levee. Sand particles begin to move as internal erosion begins on the land side of the embankment and progresses towards the river beneath a roof formed by overlying clay material. Once the piping channel reaches the river rapid flow of water thorough the hole enlarges the pipe, and eventually the embankment sloughs into the void and breaches, eroding to the foundation level and releasing uncontrolled flows. It is thought that the East Levee between the Hampton Street Pumping Station and the Hampton Bridge is the most likely reach for this to develop as this is where the embankment soils are the sandiest.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Internal Erosion Through Levee Embankment
Location:	Locations where Sand Layer Persists through Embankment
Event and Initiator:	Flood Greater Than Historical
More Likely (Adverse)	Less Likely (Favorable)
Some SPT N-values as low as 2 were recorded in sandy embankment zones	Sand layer would have to line up across the 1930’s and 1950’s levee construction for continuous layer to be present; the 1950’s add-on largely on riverside – the newer construction would form the river barrier
Some gaps in samples were reported from the borings which could be explained by lost sand	Most sandy material logged as SC, which indicates they are not clean sands but possess some cohesion and erosion resistance. Logs indicate fines contents greater than 30%
Cracks or shallow slide scarps in upstream slope of embankment could feed water into sand layer	Lost zones would have been tested with CPT
Gradients may be sufficient to move loose clean sand	Relatively low gradients would exist in areas through the levees due to relatively wide crest

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Internal Erosion Through Levee Embankment
Location:	Locations where Sand Layer Persists through Embankment
Event and Initiator:	Flood Greater Than Historical
More Likely (Adverse)	Less Likely (Favorable)
	and flat slopes
	1950's construction attempted to place coarser material on the protected side which would be more resistant to erosion
	Unlikely to have cracking or scarps capable of feeding water into a sand layer in sandy embankment zones

Likelihood Category: Low **Confidence**: Low to Moderate
Rationale: There is no clear evidence of continuous clean sand layers within the levee embankments. The two separate construction eras make it unlikely that a continuous sand layer would line up through the embankments. The primary uncertainty related to whether there might be sandy layers in areas of the embankment that have not been thoroughly explored.

Consequence Category: Level 2 to Level 3 **Confidence**: Low
Rationale: The group was pretty well split as to how severe the consequences would be. The primary uncertainties related to how effective the industrial area evacuations would be, and how quickly the embankment would erode to breach.

7. *Internal erosion through the foundation*

A large flood raises the river to unprecedented levels which imparts high water pressures in an exposed basal sand/gravel layer through an outcrop in the river channel or bridge pier penetrating to the sand layer. The sand layer is continuous beneath the alluvial clays in the foundation of the levee and outcrops on the land side in a low sump or ditch. A path for unrestricted water flow through the foundation develops. Internal erosion begins by movement of soil into the sump or ditch on the land side of the embankment and progresses towards the river by backward erosion beneath a foundation clay layer capable of forming a roof. Erosion progresses to the river and water entering the piping channel erodes and expands the pipe until eventually the embankment sloughs into the void and breaches causing uncontrolled flooding. This is most likely to occur at a pumping station where there are exposed channels on both the river side and land side.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Internal Erosion in Sand Layer Beneath Levee
Location:	Continuous Exposed Sand Layer in Foundation
Event and Initiator:	Flood Greater Than Historical

Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Geomorphology report shows three point bars extending under the levee from the protected side to river side	Areas with no sump have no daylighting sand layers
Basal sand and gravel layer exists under levee protected side to river side	No seepage has been observed through the sand layer at areas where sand daylights on the leveed site
Numerous bridge piers extend down to rock through sand on the river and protected side; basal sand exists at this location	Sumps are kept low after rain but during a large flood event it may be possible to keep the sump water level high to reduce the differential head across the sand layer
Basal sand is observed to daylight in some locations	Seepage analysis suggest small vertical seepage velocity near bridge piers (although this analysis was not available to team)
During large floods the sump may be flooded and it may not be possible to observe initiation of erosion	Material would need to move upward at bridge piers to exit on protected side
Critical gradient may be low for fine sand, if it exists in the basal sand unit	Average gradient is low due to long distance between source and exit
The levee and basal sand layer are untested for floods greater than about 1in 40	Lower sand layers in flood plain foundation soils are unlikely to daylight on protected side
Sand layers are more continuous in areas of terrace deposits	Could have vertical and horizontal discontinuity in sand layers extending under levee due to pinching and irregular disposition
Gravel mines that have been developed in the area may indicate deposits of sand and gravel are large	
Numerous sand pockets are possible in the levee foundation soils due to meanders of the river and deposition environment; continuity is unknown	

Likelihood Category: Low **Confidence**: Low
Rationale: The system has experienced flood levels to within a couple of feet of the 1/100 flood level with no observed seepage. The primary uncertainties relate to continuity of sand layers.

Consequence Category: Level 3 **Confidence**: Moderate
Rationale: These areas are the highest embankment sections and therefore would have the highest breach flows. The sump areas may be full of water which would make it difficult to observe initiation of this potential failure mode. The primary uncertainty relates to the ability to monitor for its development and implement emergency actions.

8. Heave leading to internal erosion through the foundation

A large flood causes high water pressures in the basal sand/gravel layer beneath the alluvial clays in the foundation of the levee. The water pressures exceed the weight of the confining clay soil and water above the sand layer at the land side toe, and the ground heaves opening a path for unrestricted water flow through the foundation. Internal erosion begins on the land side of the embankment and begins to progress towards the river under the clay layer which is capable of supporting a roof. Where these sand/gravel layers are continuous and pervious, erosion progresses, expands, and connects to the river. Uncontrolled flow through the "pipe" causes additional erosion and eventually the embankment crest collapses into the void and breaches causing uncontrolled flooding.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Heave
Location:	Continuous Sand Layer Confined by Clay Cap at Landside Toe
Event and Initiator:	Flood Greater Than Historical
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Thinner clay blankets exist at sump locations (e.g. Hampton) where this is most likely to occur	The most responsive piezometers (change rapidly with change in river stage) occur in deeper Paleo sand channels where clay blanket is thickest and provides more confinement
A relatively thin clay blanket exists at the land side toe along some levee sections	Dry side piezometers in sand layer not responsive to recent floods which indicates there may not be a direct connection to river in some locations
Steady state seepage models show a FOS of approximately 2 for heave with a critical gradient of 0.5 for 1 in 100 level event	Heave does not imply failure, still need to have backward erosion of the sand layer develop back to the river
A thin clay blanket exists on the high terrace downstream of the Hampton Pump Station	No seepage has been observed into sumps through clay, which indicates either the clay cap is very tight or a seepage path has not developed
A steeper (higher) gradient exist where there is a shorter seepage path from the river to the landside toe	

Likelihood Category: Low to Moderate **Confidence**: Low
Rationale: There are no clear indications of a problem area with respect to this potential failure mode. However, there are many locations where it could manifest and it is not possible to completely understand all of the anomalies that might exist.

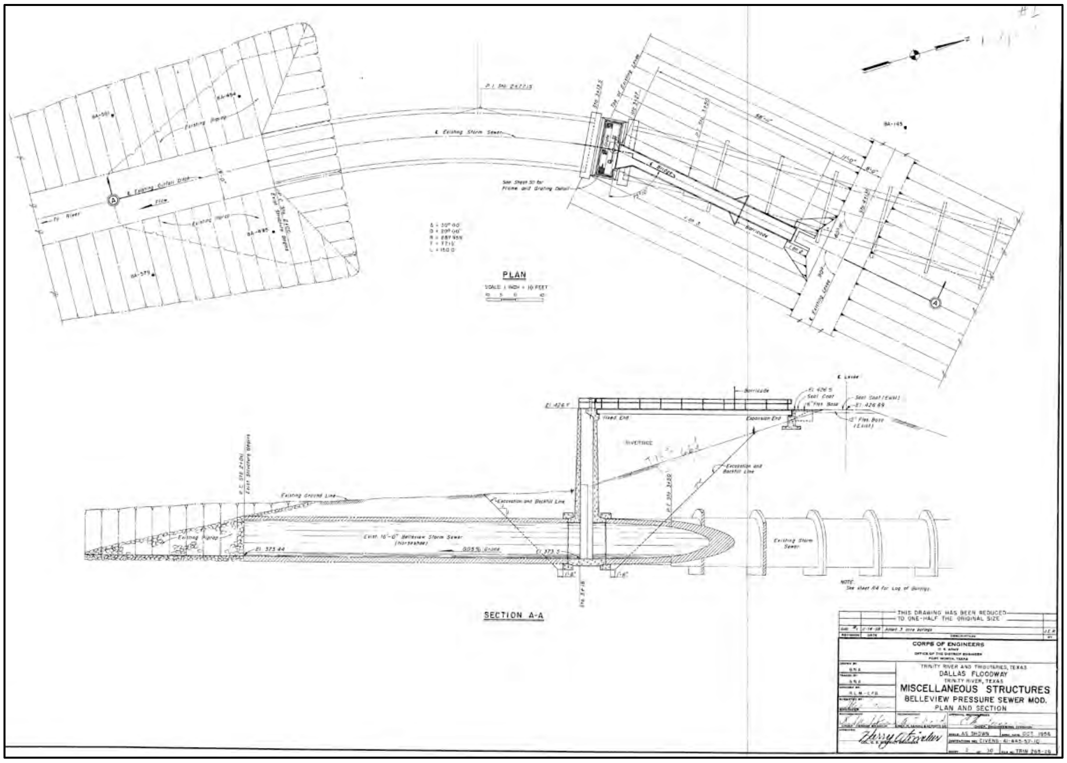
Consequence Category: Level 3 **Confidence**: Moderate

Rationale: Similar to PFM #7, these areas are the highest embankment sections and therefore would have the highest breach flows. The sump areas may be full of water which would make it difficult to observe initiation of this potential failure mode. The primary uncertainty relates to the ability to monitor for its development and implement emergency actions.

9. Internal erosion following rupture of a pressurized conduit

Pressurization of a deteriorated sewer conduit or conduit joint causes breach of the conduit and water pressure is exerted in the fill material adjacent to the conduit. A large flood occurs. High differential heads develop between the conduit breach and land side of the levee system causing seepage to begin to flow adjacent to the conduit. The water flow begins to erode the surrounding soil near an exit point on the protected side adjacent to the conduit. Erosion progresses toward the river until there is an open pathway between the landside exit and the conduit breach, resulting in high gradients between the riverside levee and the conduit breach. Backward erosion then proceeds along this portion of the conduit until the levee is breached. The embankment breach expands due to large water flows, eventually causing sloughing and breach of the embankment, and uncontrolled flooding.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Rupture or Leak of Pressurized Conduit
Location:	Any of the Pressurized Conduits where Passing through Levee Embankment
Event and Initiator:	Flood Greater Than Historical
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Belleview sewer is within approximately eight feet of levee base and its outfall is near the river side embankment toe, which leads to a shorter seepage path that could affect the embankment	Video inspection of indicates concrete is in good shape
The pressure head within the conduit could approach 50 feet based on the elevation of inflows to the pressure conduits	Material near conduit (Belleview sewer) is CH, which is more plastic and erosion resistant
Conduit joint treatment is unknown, it is not known whether waterstops were installed which would mitigate concerns at the joints	Some conduits have been run pressurized for six to eight weeks with no observed problems
Belleview conduits are old, constructed in the 1920's, and have seepage collars which have been shown to reduce the density of fill adjacent conduits due to difficulties with compacting adjacent fill	
Not sure what type of soil material was used for backfill around the conduits; silty material would be more erodible	



Likelihood Category: Very Low to Low **Confidence**: Moderate
Rationale: It is likely that high plasticity erosion resistant soils were placed adjacent to the conduit and indications are that the conduits are in good shape.

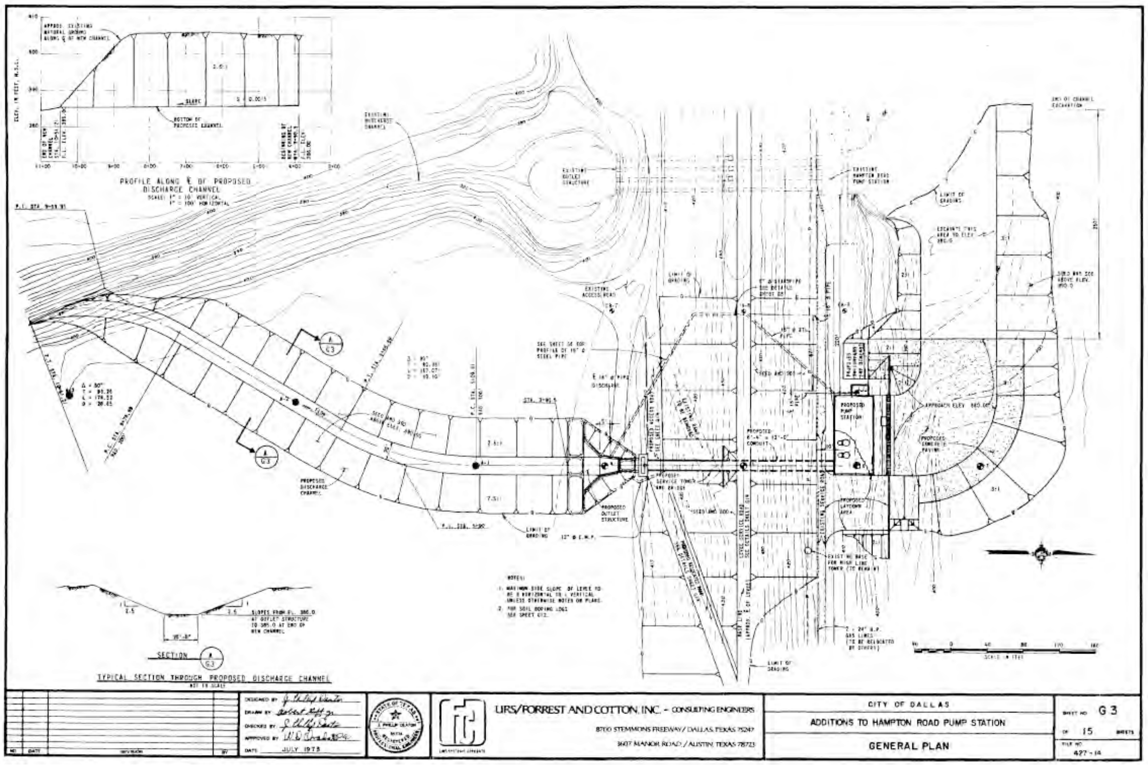
Consequence Category: Level 1/Level 2 **Confidence**: Moderate
Rationale: The areas near the pressurized conduits are business areas that are not highly populated and the embankment would likely erode slowly such that there would be time to evacuate the area.

10. Internal erosion along a penetration through the embankment or foundation

A large flood occurs causing high differential heads between the water side and land side of the levee system adjacent to an unpressurized conduit or penetration. Because of either construction practices (i.e. difficulties in compacting fill adjacent to the conduit) or settlement, a low stress or low density embankment zone exists for a significant length along the penetration. High gradients overcome the resisting forces and water begins to flow along the conduit. The water flow begins to erode the surrounding soil near an exit point on the protected side. Erosion progresses toward the river and expands eventually resulting in an open pathway between the river side and protected side. Rapid flow through the pathway results in sloughing and breach of the embankment causing uncontrolled flooding.

Event Information

Loading Condition:	Hydrologic
Failure Mode:	Scour Along Embankment Penetration
Location:	Any Conduit or Penetration through the Levee Embankments
Event and Initiator:	Flood Greater Than Historical
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Pump station conduits are shortest seepage path and are a location this could develop	No performance issues have been observed that would suggest flaws exist along any of the conduit
Seepage collars were constructed in the top part of the conduit (through soils) at some locations which make it difficult to compact soil in this area	Landside conduit connected to pump houses would limit any exit areas for soil movement
A narrow slot was left adjacent to the upper part of a conduit where it would have been difficult to compact soil	Lower portion of conduits were cast against rock cut in some locations would not be an erodible seepage path
Gravity drains were constructed in 1930's and have seepage collars	Filter (sand/gravel) was placed surrounding the conduit at the protected side in some locations which should mitigate potential erosion along the conduit
Average gradient is approximately 0.25 which is large enough to move poorly compacted erodible soils	A headwall was constructed at the river side of some conduits would help lengthen and seal the seepage path along the conduit
	Some pump stations conduits were constructed in high plasticity erosion resistant clays
	Conduits are covered by a roadway on the protected side in most areas which would lengthen the seepage path and reduce the gradient


PROFILE ALONG ℄ OF PROPOSED DISCHARGE CHANNEL
TYPICAL SECTION THROUGH PROPOSED DISCHARGE CHANNEL
URS/FORREST AND COTTON, INC. - CONSULTING ENGINEERS
8700 STEMMONS FREEWAY / DALLAS, TEXAS 75247
3607 MANOR ROAD / AUSTIN, TEXAS 78723
CITY OF DALLAS
ADDITIONS TO HAMPTON ROAD PUMP STATION
GENERAL PLAN
JULY 1973
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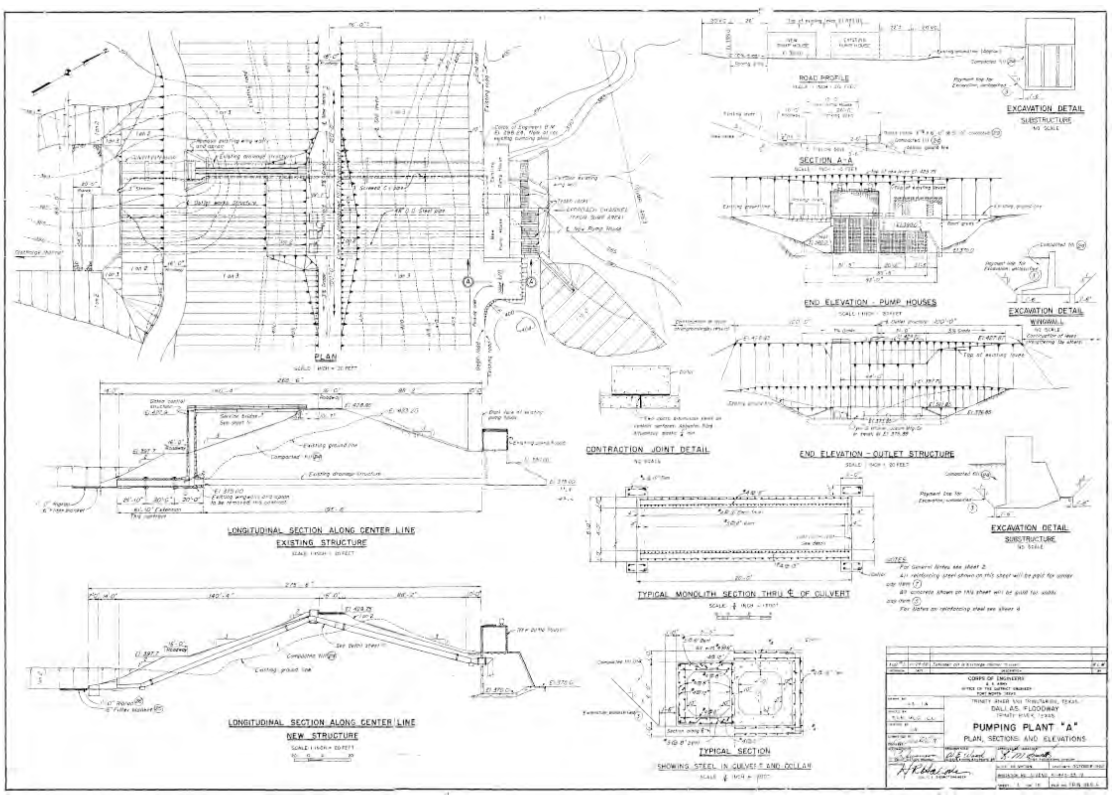
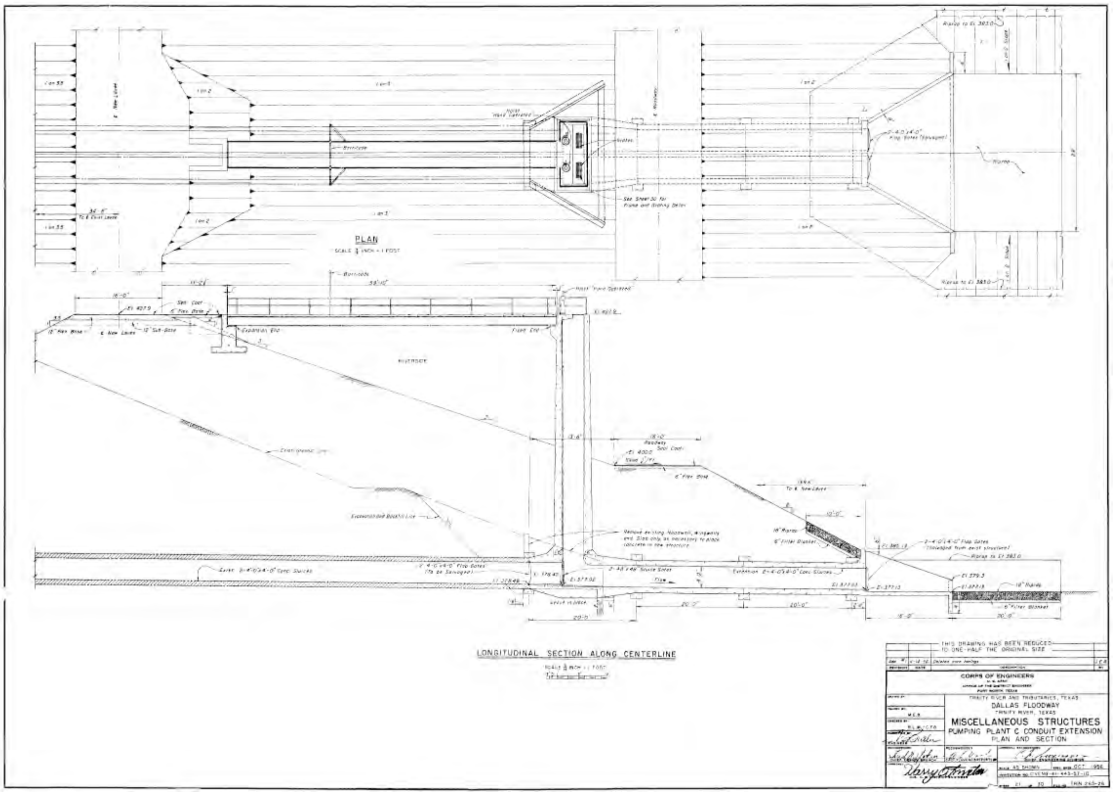

ROAD PROFILE
EXCAVATION DETAIL
SUBSTRUCTURE
SECTION A-A
END ELEVATION - PUMP HOUSES
EXCAVATION DETAIL
WINGWALL
PLAN
CONTRACTION JOINT DETAIL
END ELEVATION - OUTLET STRUCTURE
LONGITUDINAL SECTION ALONG CENTER LINE
EXISTING STRUCTURE
TYPICAL MONOLITH SECTION THRU ℄ OF CULVERT
EXCAVATION DETAIL
SUBSTRUCTURE
LONGITUDINAL SECTION ALONG CENTER LINE
NEW STRUCTURE
TYPICAL SECTION
SHOWING STEEL IN CULVERT AND COLLAR
CORPS OF ENGINEERS
DALLAS FLOODWAY
PUMPING PLANT "A"
PLAN, SECTIONS AND ELEVATIONS



Likelihood Category: Low **Confidence**: Moderate
Rationale: It is likely that high plasticity erosion resistant soils were placed adjacent to the conduit and indications are that the conduits are in good shape. The conduits are in areas that are easily observable and good access exists to intervene if necessary.

Consequence Category: Level 3 **Confidence**: Moderate
Rationale: If a breach were to develop it would be near the base of the highest portions of the levees, resulting in complete breach and flows would likely be deep and fast.

11. Global instability following leaks from a pressurized conduit

A large flood occurs. During the flood, a pressurized conduit ruptures more than 5' high in the embankment. This rupture saturates the embankment. The saturated soil reduces the effective strength of the levee and the embankment begins to slough. The ruptured line continues to saturate the embankment leading to more sloughing, possibly on both landslide and river side slopes. Erosion and slumping progresses and expands eventually leading to loss of crest below the river level, overtopping and breach of the embankment causing uncontrolled flooding.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Failure of Water Pipe Leading to Saturation of Embankment Causing Slope Instability

Location:	Water Lines Passing over or through the Levee Embankments
Event and Initiator:	Flood Greater Than Historical

Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
One known incident where a water line valve broke and induced a slope slide	All valves of similar design from the one that failed were replaced
Five water lines (24" to 48") traverse the levee: 2-24", 1-36", 2-48"	Valves are located near edge of crest making it unlikely that both slopes would be affected
One jet fuel line traverses the levee which could produce a similar effect	The one known incident of valve failure was more of a local failure than a global failure
It could take a while to detect a ruptured line if the leak is small (except for jet fuel)	Coincident high stage and valve/pipe failure is unlikely
Soils are prone to sliding when saturated as evidenced by numerous slope failures historically	High river stage could stabilize river side slope by putting loading against the slope
Coincident flood stage and valve leak could lead to deeper saturation and therefore deeper slide	Water line inspection tools include a device to listen for leaks; inspection occurs once a year and all water lines are inspected

Likelihood Category: Low **Confidence**: Moderate
Rationale: A string of unlikely events, water line break that is not repaired in concurrence with a large flood that brings the river level near the levee crest, are required for this potential failure mode to develop.

Consequence Category: Level 3 **Confidence**: Moderate
Rationale: The leaking pipe would need to go undetected and unrepaired for an extended period of time, which means the failure would likely happen without much warning. This could result in significant consequences.

12. *Instability at the interface between 1930's and 1950's levees*

There is a weak zone at the interface between the 1930's and 1950's embankment section and a tension crack develops above this interface. Antecedent rainfall causes saturation of interface and a slide occurs along this plane of weakness. A large flood occurs prior to repairing the previous slide, causing additional saturation of the embankment. This causes a reduction in effective strength and additional sliding, or possibly internal erosion through the upper portion of the embankment. The slide progressively worsens eventually leading to a breach of the levee and overtopping from the river flow.

Event Information	
Loading Condition:	Hydrologic
Failure Mode:	Antecedent Rain Saturates Embankment and Weakens Old-New Levee Interface Causing Retrogressive Slides
Location:	Location where Levee Slopes are Steepest Adjacent to the Old Levee Core

Event and Initiator: Flood Greater Than Historical	
Influence Factors	
More Likely (Adverse)	Less Likely (Favorable)
Treatment (e.g. depth of old levee excavation, benching, scarifying) at the tie between old and new levees is unknown, it could have been minimal leaving a plane of weakness in this location	No indication that tension cracks line up with interface
If slide occurs, it reduces levee section near the crest and increase average gradient across embankment	Mid slope borings (9 total) show no appreciable difference between old and new levee (no apparent interface could be indentified)
The majority of historical slides have been on the river side slope and would not be able to repair slide on river side with arriving flood if it followed quickly after antecedent rain	If slide occurs on river side, buttressing of land side can take place as an intervention measure
	River loads stabilize the river side slope
	It would take a very large flood after initial slide occurred to impinge on initial scarp
	It would take some time to retrogress to breach formation which would allow for time to intervene or evacuate the population at risk

Likelihood Category: Low **Confidence**: Moderate
Rationale: There is no indication that the interface between the old and new levees is a plane of weakness. A string of unlikely events (antecedent rain, slide at interface, large flood, continued sliding with no intervention) would be required for breach.

Consequence Category: Level 2 **Confidence**: Moderate

Rationale: The retrogression would take some time, so there would be time to react and evacuate the population at risk.

Note: Although this potential failure mode was assigned to a low risk category, a similar potential failure mode was later added and evaluated quantitatively, that was not necessarily related to the interface between the old and new levees.

13. Global slope instability

This potential failure mode was not developed in detail, but the decision was made that it should be carried forward for detailed risk assessment. It was tentatively assigned a Failure Likelihood of “Moderate” at Consequence Level 3 pending additional evaluation. Embankment slope slides have been prevalent throughout the history of the levee system. Although none of these have breached a portion of the crest of the levee, the possibility exists that this could occur at higher stage levels than have been experienced to date.

The majority of the discussion related to this potential failure mode concerned input parameters for seepage and stability analyses. These are discussed in more detail in the risk assessment section and appendices of this report.

14. *Failure modes not developed*

The chances of the following potential failure modes developing were judged to be obviously remote by the team following the brainstorming exercise, and they were not carried forward for failure mode analysis or quantitative evaluation. The reasons for these judgments are also provided below.

- **Channel Erosion on the Levee Slopes Breaching the Levee Crest.** Although long-term neglect related to maintenance of the slopes might lead to erosion channels contributing to the likelihood of other failure modes, it was difficult for the team to envision a scenario that would indicate this is a significant issue that could lead to levee breach.
- **Debris Blockage at Bridges Leads to Premature Levee Overtopping**. Debris blockage is a contributing factor to overtopping and was included in the overtopping failure mode evaluation.
- **Failure to Install Houston Street Viaduct Closure Leads to Flow Through the Area and Widening Erosion.** The viaduct is a hard surface and would only require a single Hesko basket row to close. Plans are in place and materials are available for this closure. Some sandbagging on either side might be required on the levee around the viaduct. It seems likely the plan would be carried out.
- **Failure to Install Railroad Closure Leads to Flow Through the Area and Widening Erosion**. Materials are stockpiled and in place. A plan exists to install the Hesko baskets. The closure is put into place during the 100-year event, which is far below the water elevation required to load the closures. A similar closure has been successfully installed at the abandoned spur line. Therefore, it seems likely that an effective closure would be installed.

Results

The results of the potential failure mode analysis are shown in the Figure 4 matrix. The decision was made to evaluate those potential failure modes that plotted in the Moderate or Low to Moderate likelihood categories at Level 2 or Level 3 Consequences quantitatively in more detail, since these pose the highest risk. In addition, PFM #7, backward erosion piping along a continuous foundation sand layer, could be evaluated as part of PFM #8, foundation heave, since it is a necessary part of that potential failure mode. That being the case, one might question why the team categorized PFM #7 as lower risk than PFM #8. The reasoning as to why this was not the case is as follows. If there was an open unconfined sand conduit, seepage or problems should have been observed in previous flood events. On the other hand, a blanket may have been sufficient to obscure seepage and keep the materials intact for lower flood elevations experienced to date, and only masked the potential heave problems that could be manifested at a higher flood stages. However, it was noted that when quantitative estimates are made, the relative ranking of these two potential failure modes could reverse. Finally, a progressive slope instability potential failure mode (PFM 13b), not necessarily tied to the interface between the old and new levee embankments, was also evaluated quantitatively after additional discussions during the risk assessment.

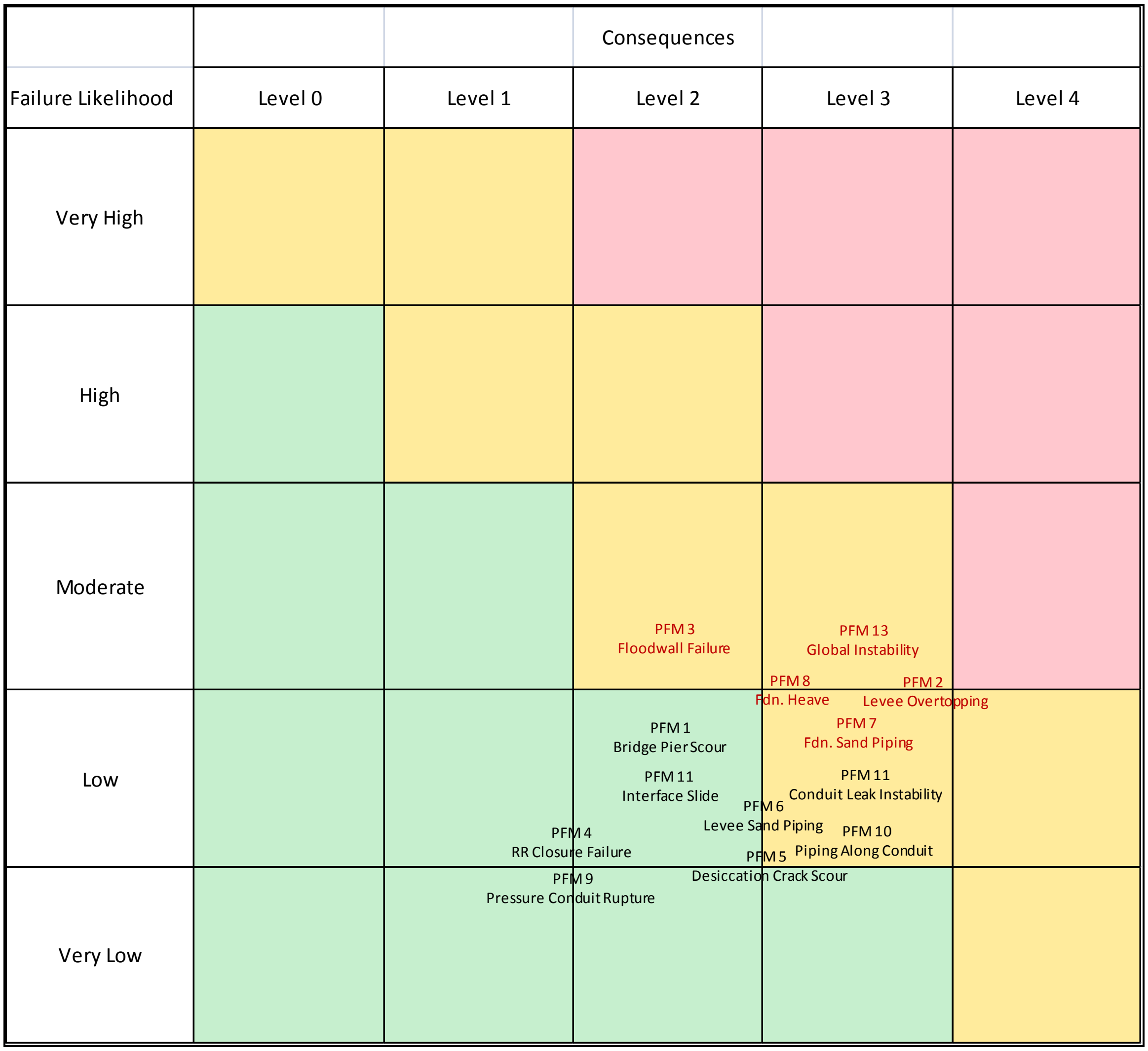
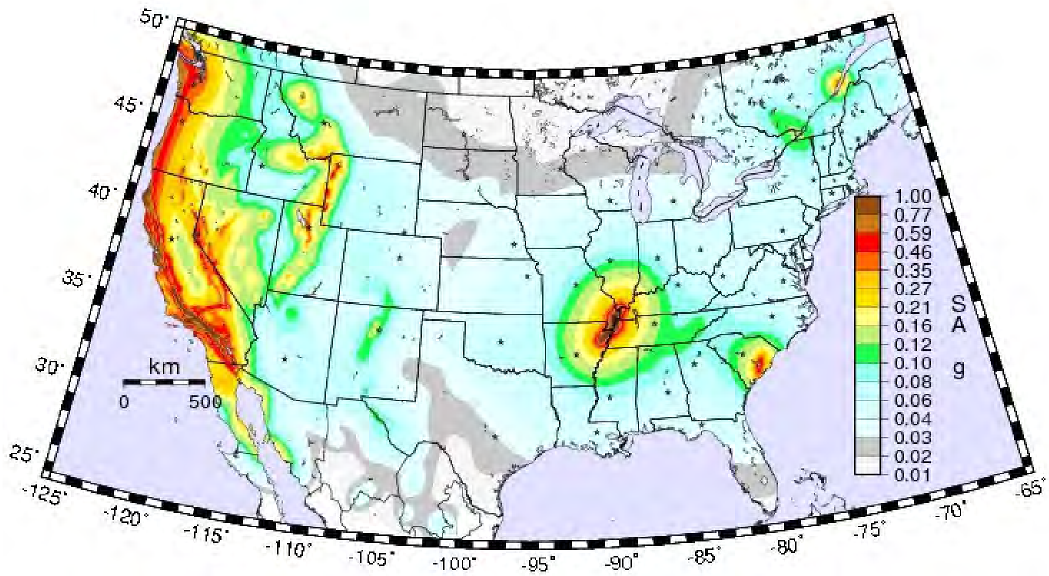


Figure 4 - Resulting Failure Mode Matrix

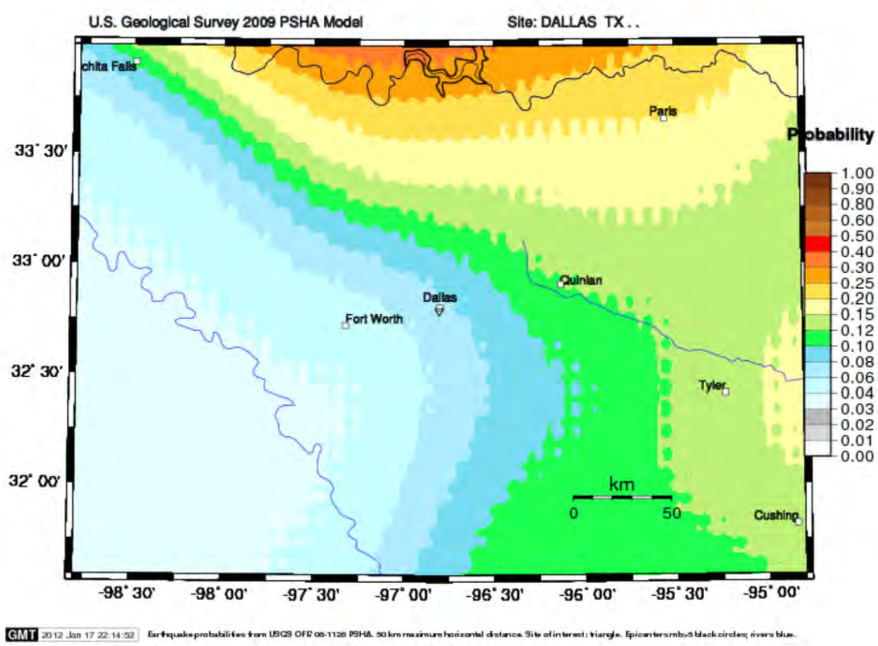
Seismic Analysis

A detailed seismic analysis was not completed for the system given the relative seismicity in the region. Given the infrequent seismic loads and the infrequent hydraulic loads, further analysis is not required.

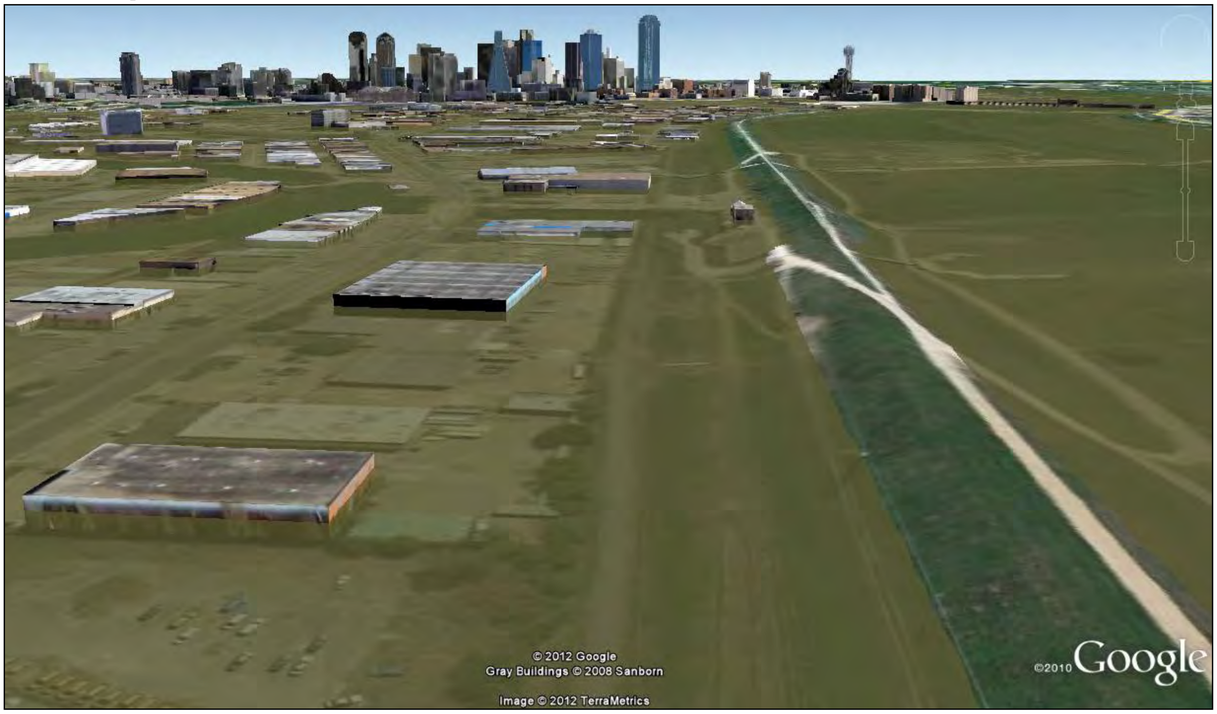
1.0-s SA with 2% in 50 year PE. BC rock. 2008 USGS



Probability of earthquake with M > 5.0 within 1500 years & 50 km



Consequences



A failure of Dallas levee system would likely result in high consequences for the population at risk (PAR), likely leading to loss of life and millions of dollars of economic damage; this section of the report is intended to provide summary results from attempts to quantify such consequences. A more detailed account of both the data and methods used as well as the sensitivity of results to various assumptions are provided in Appendix E.

Estimates of PAR and potential loss of life were made using the USACE Hydrologic Engineering Center's Flood Impact Analysis (HEC-FIA[2]) model. The life loss methodology in HEC-FIA is based on the LifeSim[3] methodology developed by Utah State University's Institute for Dam Safety Risk Management. HEC-FIA is a stand-alone, GIS enabled model that is also used to analyze flood impacts to structures, contents, vehicles and agriculture. Warning issuance ranges for this assessment were determined through Expert Opinion Elicitation involving the risk cadre and local officials. The most likely condition is several hours of advanced warning for overtopping and global instability failures but warning after breach for internal erosion failures. Depending on the time of day, hydrologic event, failure location and failure mode, the total PAR typically ranged from 20,000 to 100,000.

FIA's consequence methodology involves several steps, some of which occur simultaneously. First, an inventory of the structures within the potential hazard area is compiled, involving such features as occupancy type and number of stories. Second, population is calculated for each

[2] HEC-FIA (Flood Impact Analysis) is a software product developed by USACE that calculates economics damages and potential life loss.
[3] http://uwrl.usu.edu/people/faculty/DSB/ASDSO%20LIFESim%20Paper-FINAL.pdf

census block and then distributed between each structure within that census block. A warning issuance assumption must be made indicating when, relative to breach, the first warning to the public would occur. A warning diffusion curve is created, which reflects how long after the first warning issuance it takes to warn 100% of the PAR. A Mobilization curve is used to reflect the rate at which the warned PAR begins to evacuate. An evacuations velocity is assumed, indicating the speed at which evacuating PAR will travel on their way out of the hazard area. Grids created in the H&H modeling process are used to determine how quickly the flood wave will arrive at each structure and what the depth ultimately will be at each structure. The fatality rate used for each individual structure is a factor of the water surface elevation at that structure, the foundation height of the structure, the number of stories of the structure and whether or not the population within that structure is assumed to be either elderly or disabled. This fatality rate is then multiplied by the number of unmobilized PAR within the structure to obtain the loss of life within the building. Loss of Life across all structures is then summed, along with the estimated loss of life among PAR caught while in the process of evacuating, to obtain the total Loss of Life for the scenario.

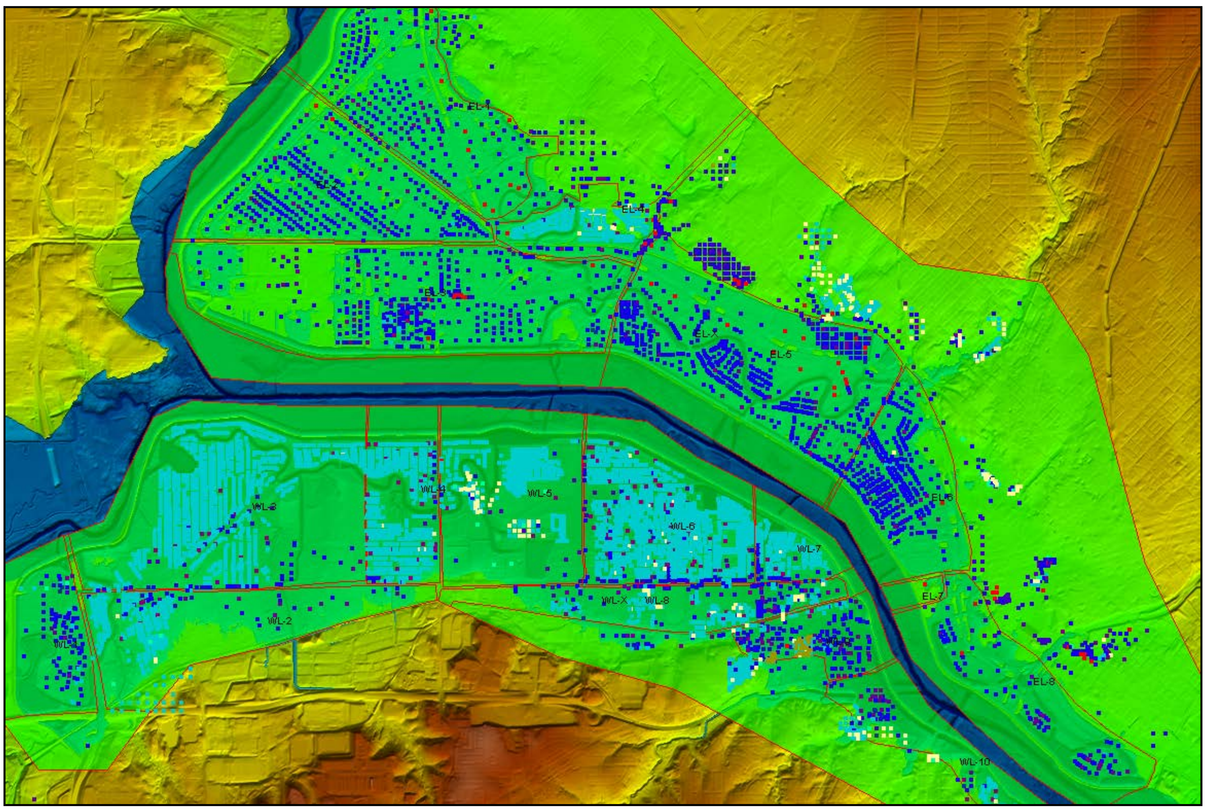


Figure 5 - FIA Screenshot Displaying Impact Areas and Structure Inventory

Practically speaking, the PAR has two relatively homogenous groups. Behind the East Levee is downtown Dallas, it is primarily a commercial zone, filled with warehouses, offices, and retail buildings. Likewise, the population behind the east levee is largely, but not entirely, commercial workers who work within the hazard zone but reside elsewhere. Because of this, there is a significant difference between the PAR behind the East Levee during the day and the PAR at night (potentially 91,400 Day PAR and 35,500 Night PAR) and we would also expect higher evacuations rates during the day when the PAR is disproportionately composed of workers. Much of the night PAR are visitors staying in hotels in the downtown area or institutionalized populations; it is important to note that such PAR are generally in hi-rises, because this PAR is able to "vertically evacuate", they are less directly threatened by floodwaters.

Though smaller in number (19,600 Day PAR and 23,500 Night PAR), the PAR behind the West Levee is largely, but not entirely, residential. Most of the PAR lives in one-story single-family structures, with a smaller percentage living in multi-family units. According to Census data used in this analysis, households behind the West Levee are often low-income, without a fluent English speaker, and may not have access to a vehicle. Such demographic factors reduce the likelihood that the PAR will personalize warnings, perceive significant risk and have the resources available to successfully evacuate.

In scenarios where there is significant advanced warning, such as the overtopping scenarios modeled in the most likely condition, the majority of the population can be expected to evacuate. Given sufficient depths to result in fatalities, the driver becomes the size of the minority who are physically incapable or otherwise unwilling to evacuate before arrival of floodwaters.

The maximum percent of the PAR from a zone that will attempt evacuation provided sufficient time is known as the "max mobilization rate". While there is considerable uncertainty as to what the mobilization rate would be for a given hazard, the hazards literature suggests 95% as a useful average of community wide evacuation rates for a preventive evacuation due to a forecasted levee failure[4]. To account for site-specific variance from this rate, several different impact areas were constructed.

For the primarily residential population behind the West Levee, an index was created to weigh various factors against each other. While there is uncertainty around relative significance, variables used in this index have generally been shown in the literature to be correlated with evacuation status[5]. Example demographic variables include percentage of elderly households, percentage of households below 150% of the poverty line, and percentage of households without vehicles. Non-demographic variables were also used to weigh site specific factors that may have an impact on risk perceptions; examples of such variables include the average distance from the levee, presence of environmental cues (extreme weather), and quality of warning message. Ultimately, while an evacuation rate of 94.5% for the most extreme hydrologic conditions and 94% for less extreme (no threat of overtopping) were used in these residential zones in the west levee.

The commercial zones behind the east levee did not lend themselves to a similar method. Instead, a likely aggregate maximum evacuation rate was estimated by assuming the vast majority of commercial workers would be willing and able to evacuate, but only the standard 95% rate of other categories of PAR would be willing and able to evacuate. The resulting

[4] Jonkman, Sebastian Nicolaas. *Loss of Life Estimation in Flood Risk Assessment: Theory and Application*. 2007.
[5] Mileti, Dennis and Sorenson, John. *Communication of Emergency Public Warnings: A Social Science Perspective and State-of-the-Art Assessment*. 1990.

aggregated max mobilization rate for predominately-commercial areas is 99.5% during the day and 96% at night. A mainly residential zone behind the east levee used a 95% mobilization rate for both day and night.

Appendix E – Consequences provides a more detailed explanation of the utilized methodology, parameters used, their justification and the sensitivity of the results to various assumptions. The following tables are provided for summary purposes.

In the most likely scenarios, life loss consequences were generally higher for the West Levee than the East Levee. This is because, with significant advanced warning, most of the commercial workers are able to evacuate and many of those who do not evacuate face relatively low fatality rates in hi-rise structures. Meanwhile, the more vulnerable PAR behind the East Levee is less likely to successfully evacuate, and those who do not evacuate find little refuge in one-story homes; this is particularly true of the elderly and disabled.

Internal Erosion scenarios have higher loss of life than Global Instability cases due to less warning opportunity time. It is also important to note that for Internal Erosion scenarios, there is a decrease in Loss of Life as loading moves from "3/4 Height" to "Threshold", or full loading. This is because when freeboard becomes an issue, overtopping concerns control the warning issuance assumption. Meaning, more advanced warning would be available for these extreme events. This increase in warning more than offsets the increased danger due to higher eventual depths.

The results show that many scenarios have a wide range of uncertainty (with the "expected" column in Table 4 reflecting a weighted average of Day and Night Loss of Life). While the most likely condition is that there will be significant advanced warning (for Overtopping and Global Instability), if there is not, it is less likely that the PAR will successfully evacuate. The three calculated scenarios (best case, worst case and most likely) were used to create a PERT distribution (min, max and most likely); the resulting mean from the distribution was used as the best estimate.

Warning has a significant effect on the consequences (life loss) that would be experienced upon breach of the levee. The more time that people have to react, the better they are able to get out of harm's way. In addition, if it is known that failure is imminent, additional warning time will allow evacuation of people not able to evacuate on their own. The time it takes for breach formation will also affect the consequences. A slow breach will result in slowly rising breach inundation flows and will not only allow more time for evacuation, but will also alert people to increasing threat as the water rises. The team decided to address these two issues in a group setting. The following table summarizes the team estimates.

Case	Low Estimate	Best Estimate	High Estimate
Internal Erosion – Time of Forceful Warning (hours before breach) – Applies with several feet of freeboard	-3	0	0

Internal Erosion – Breach Formation Time (hours)	12	26	40
Slope Instability – Time of Forceful Warning (hours before breach) – Applies when river is very high	0	8	12
Slope Instability – Breach Formation Time (hours)	3	6	10
Overtopping – Time of Forceful Warning (hours before breach) – Applies when river approaches top of levee	0	8	12
Overtopping – Breach Formation Time (hours)	6	13	20

These numbers were used to run sensitivity cases using the HEC-FIA software. A complete discussion of the input and results can be found in Appendix E – Consequences.

Risk Assessment

The risk estimates were completed during team meetings in December 2011 and January 2012. During the Potential Failure Modes Analysis, the team developed a set of material parameters to be used for all the supporting analyses.

Two cases were run in the event trees - one with the median estimate for each branch (the most likely case) and one with a normal distribution defined by the mean and standard deviation to study uncertainty. Since the mean and median were relatively close, the results were similar.

Levee Cross Sections

Figure 6 shows the locations of cross sections used to evaluate seepage, internal erosion, heave, and stability. The sections were chosen to be worst case sections representative of the entire system. Detailed descriptions of the cross sections chosen and their parameters can be found in Appendix A – Cross Section Selection. Cross sections are included in each portion of the risk assessment.

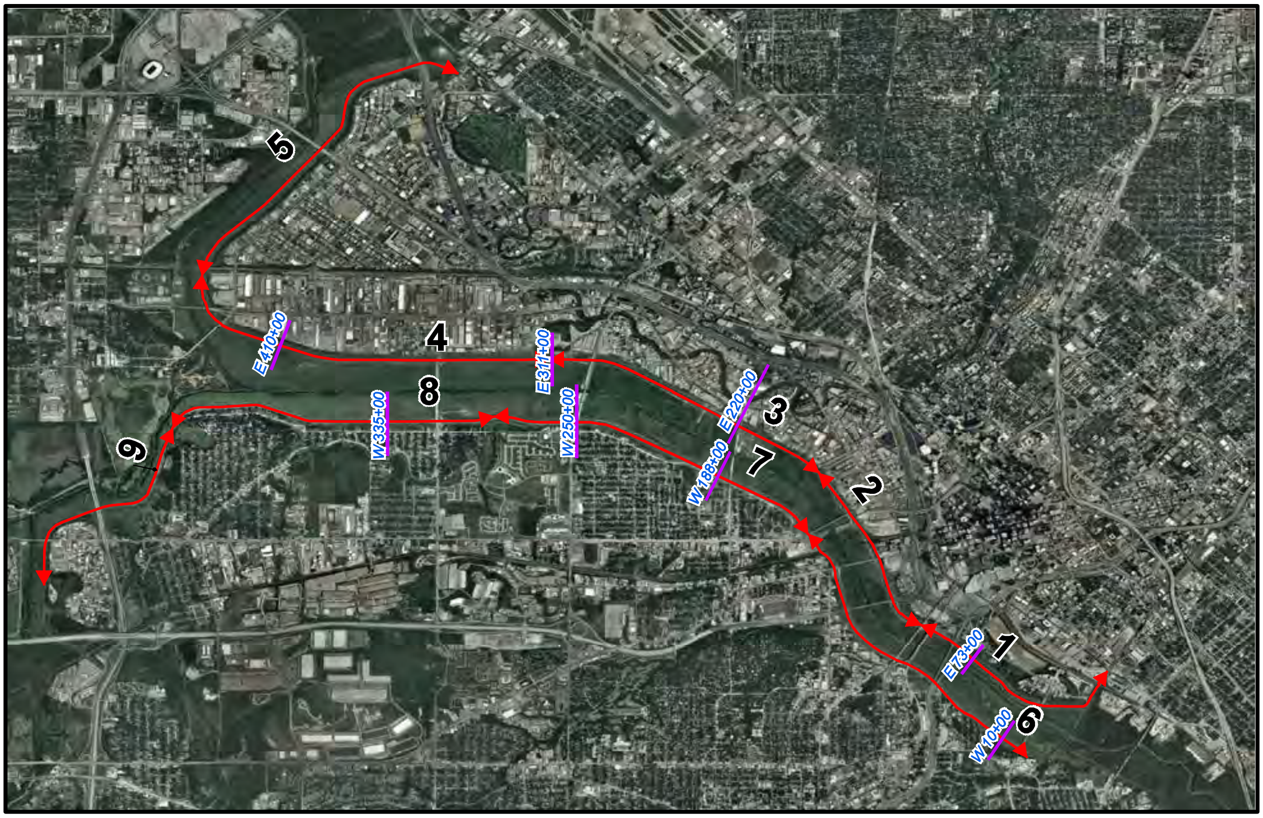


Figure 6 – Reaches are labeled in black, cross section locations labeled in blue.

Geotechnical Parameters

Permeability

During the PFMA session, the team examined the large amount of data available for the site and used that information to develop a range of permeabilities[6] for the various layers that exist in the foundation and embankment of the floodway. The data is shown in Table 5 and Table 6 below:

Table 3 - Material Permeability in feet/second

	CH or CH-Fill	CL or CL Fill	GP	GW	GP-GC	GW-GC	GC	SC
Number of Tests	**245**	**144**	**4**	**9**	**14**	**8**	**1**	**41**
min	2.08E-09	1.53E-08	1.84E-02	4.02E-04	3.35E-03	1.05E-02	7.44E-04	8.20E-09
mean	7.77E-05	4.88E-04	6.52E-01	7.92E-01	5.15E-02	4.08E-02	7.44E-04	4.24E-03
median	5.82E-07	1.10E-05	3.29E-01	3.11E-01	4.91E-02	3.93E-02	7.44E-04	5.12E-04

[6] Permeability information was compiled in the City of Dallas' 408 submittal package.

Number of Tests	CH or CH-Fill	CL or CL Fill	GP	GW	GP-GC	GW-GC	GC	SC
	245	**144**	**4**	**9**	**14**	**8**	**1**	**41**
max	3.20E-03	1.32E-02	1.94E+00	2.87E+00	1.35E-01	8.75E-02	7.44E-04	4.11E-02
10th Percentile	2.30E-08	7.38E-08	2.03E-02	6.61E-02	9.24E-03	1.80E-02		4.15E-07
33rd Percentile	1.41E-07	1.05E-06	2.46E-02	2.33E-01	2.78E-02	2.86E-02		1.07E-04
50th Percentile	5.82E-07	1.10E-05	3.29E-01	3.11E-01	4.91E-02	3.93E-02		5.12E-04
67th Percentile	1.93E-06	1.38E-04	6.49E-01	5.12E-01	6.40E-02	4.60E-02		3.54E-03
90th Percentile	1.26E-04	1.62E-03	1.55E+00	2.42E+00	9.88E-02	6.55E-02		9.88E-03

Table 4 - Material Permeability in Centimeters/Second

Number of Tests	SM	SP	SW	SP-SC	SP-SM	SW-SC	SW-SM	Shale
	10	**85**	**4**	**184**	**52**	**37**	**7**	**4**
min	5.15E-08	9.17E-05	9.60E-04	8.20E-04	3.69E-04	4.30E-04	1.15E-04	1.56E-07
mean	4.66E-03	4.11E-02	6.22E-02	2.89E-02	2.18E-02	3.08E-02	3.08E-02	3.84E-07
median	2.66E-03	4.11E-02	6.95E-02	2.07E-02	1.78E-02	1.52E-02	1.26E-02	2.20E-07
max	1.37E-02	1.26E-01	1.08E-01	1.15E-01	6.40E-02	1.60E-01	8.60E-02	9.36E-07
10th Percentile	1.74E-04	1.97E-03	2.03E-02	5.15E-03	7.99E-03	2.86E-03	6.64E-03	1.59E-07
33rd Percentile	1.38E-03	2.31E-02	6.46E-02	1.44E-02	1.36E-02	8.93E-03	1.22E-02	1.65E-07
50th Percentile	2.66E-03	4.11E-02	6.95E-02	2.07E-02	1.78E-02	1.52E-02	1.26E-02	2.20E-07
67th Percentile	4.82E-03	5.00E-02	7.41E-02	3.04E-02	2.92E-02	1.88E-02	3.93E-02	2.82E-07
90th Percentile	1.34E-02	7.44E-02	9.78E-02	6.55E-02	3.84E-02	1.00E-01	6.77E-02	7.38E-07

From that data set and incorporating the judgment of the team, values from Table 7 below were used for all analyses done during the risk assessment.

Table 5 - Permeabilities Used in the Risk Assessment

Material	Low k (cm/sec)	Best Estimate k (cm/sec)	High k (cm/sec)	Basis for Estimate
Basal Sands	6.5E-05	1.3E-03	2.4E-03	HNTB SP
Point Bar Sands	6.5E-05	1.3E-03	2.4E-03	HNTB SP
High Plasticity Clay	1.0E-08	1.0E-07	1.0E-05	EM, HNTB Data
Desiccated Clay	1.0E-06	1.0E-05	1.0E-04	EM, HNTB Falling Head Data

Material	Low k (cm/sec)	Best Estimate k (cm/sec)	High k (cm/sec)	Basis for Estimate
Clean Basal Gravel	1.0E-03	1.0E-02	6.0E-02	HNTB GW, GP
Dirty Basal Gravel	4.0E-04	1.0E-03	3.0E-03	HNTB GW-GC, GP-GC
Lean Clay	2.4E-09	5.0E-07	5.3E-05	HNTB CL
Clayey Sand	1.0E-08	1.0E-06	3.0E-04	HNTB SC

Strength

During the PFMA session, the team examined the data available for the site and used that information to develop a range of strengths available for the various layers that exist in the foundation and embankment of the floodway. A more detailed description of strength development and stability model parameters can be found in Appendix C – Stability Analysis. The data is shown in Table 8 below:

Table 6 - Material Strength Parameters Used in the Risk Assessment

Material	Parameter	Min	Best	Max
CH Fill	Phi	15.5	18.4	30
	c (psf)	100	300	500
CH	Phi	16.7	19.3	26.6
	c (psf)	200	250	300
CL Fill	Phi	21.3	23.5	31
	c (psf)	100	300	500
CL	Phi	18.4	24	26.5
	c (psf)	150	300	500
CH FSS	Phi	14	18	27
	c (psf)	100	180	250
Basal Sands	Phi	29	32	34
	c (psf)	-	-	-
Clean Basal Gravel	Phi	32	35	38
	c (psf)	-	-	-
Clayey Sand	Phi	27	30	32
	c (psf)	-	-	-
Shale	Phi	15	24	36
	c (psf)	200	1950	3000

Hydraulic Conditions

Levee Profiles

In some areas, there is a difference between the design grade of the East and West Levee systems. The 2003 and 2010 survey data were nearly identical. This profile was compared to the water surface profile when evaluating overtopping risks for the system.

Frequency of Loading

Figure 8 depicts the relationship between discharge and frequency for the Dallas Floodway used in the risk assessment. The team developed 5th percentile, median, and 95th percentile relationships to use in the monte carlo simulation. For each failure mode, the hydraulic loading was first calculated using the median curve. For the monte carlo simulation on each failure mode, the 5th, median, and 95th percentile values were put into a lognormal distribution. The lognormal distribution is not a perfect match for the loading curve uncertainty bounds, so several distribution shapes were examined. The results were not sensitive to the type of distribution, so the lognormal was selected to represent hydrologic uncertainty.

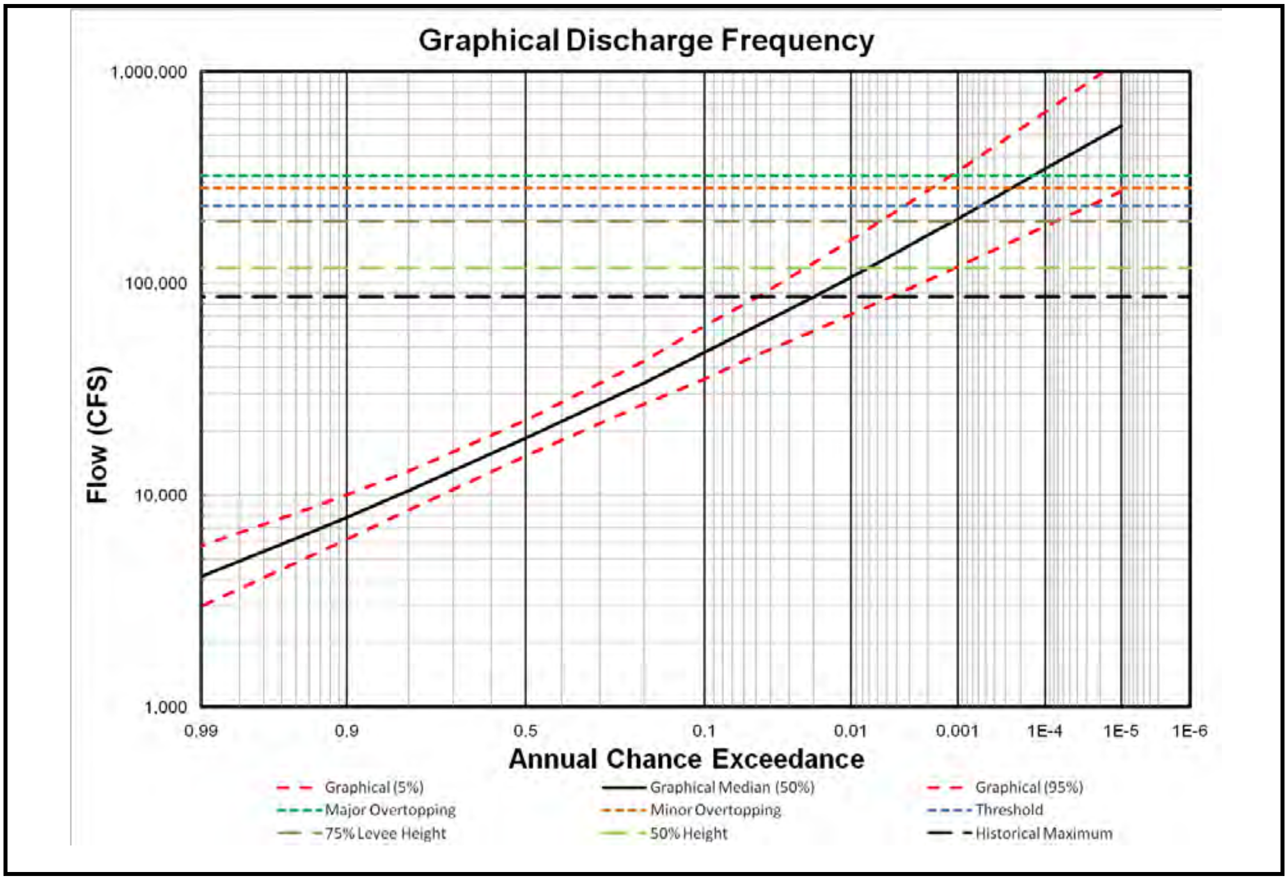


Figure 7 - Discharge Frequency Curve with Uncertainty Bands

Hydrographs

Three hydrograph shapes were selected from the historical and regional datasets as the most critical from both a hydrologic and geotechnical perspective. All three hydrographs were developed for the Dallas Floodway. Two historical hydrographs were selected, the May 1990 flood and the June 2007 flood. The Standard Project Flood (SPF) hydrograph was also selected. Each of those hydrographs were scaled to match discharges from Figure 8. A detailed description of the flood hydrographs are shown in Appendix D – Hydrology and Hydraulics. The June 2007 flood hydrograph is the most severe for geotechnical analyses because it stays above ½-height of the levee for longer than the other two. However, no hydrograph could be reasonably envisioned that rose above ½-height for more than 8 days. Hydrographs were examined that had total volumes that could cause much longer loading, but the recurrence of those types of events were significantly less frequent than what's shown in Figure 8. A typical group of hydrographs is shown in Figure 9 below.

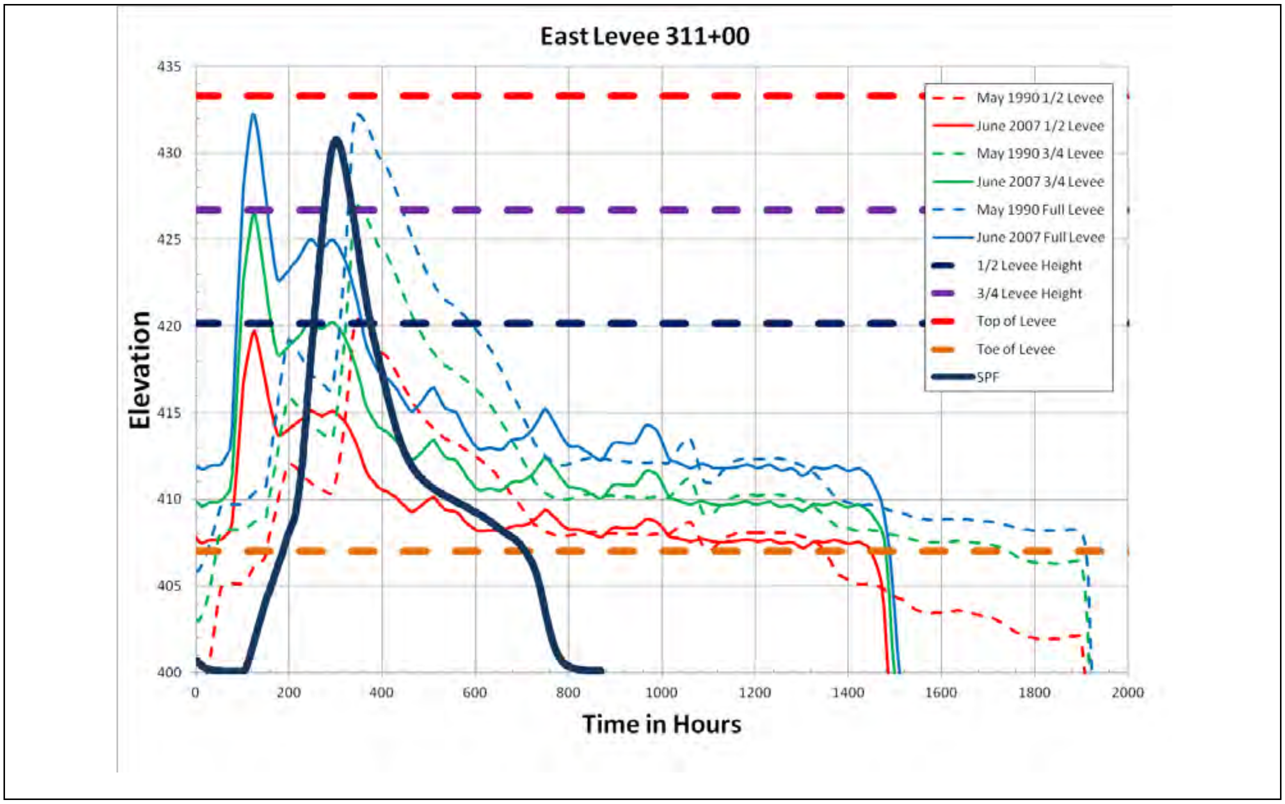


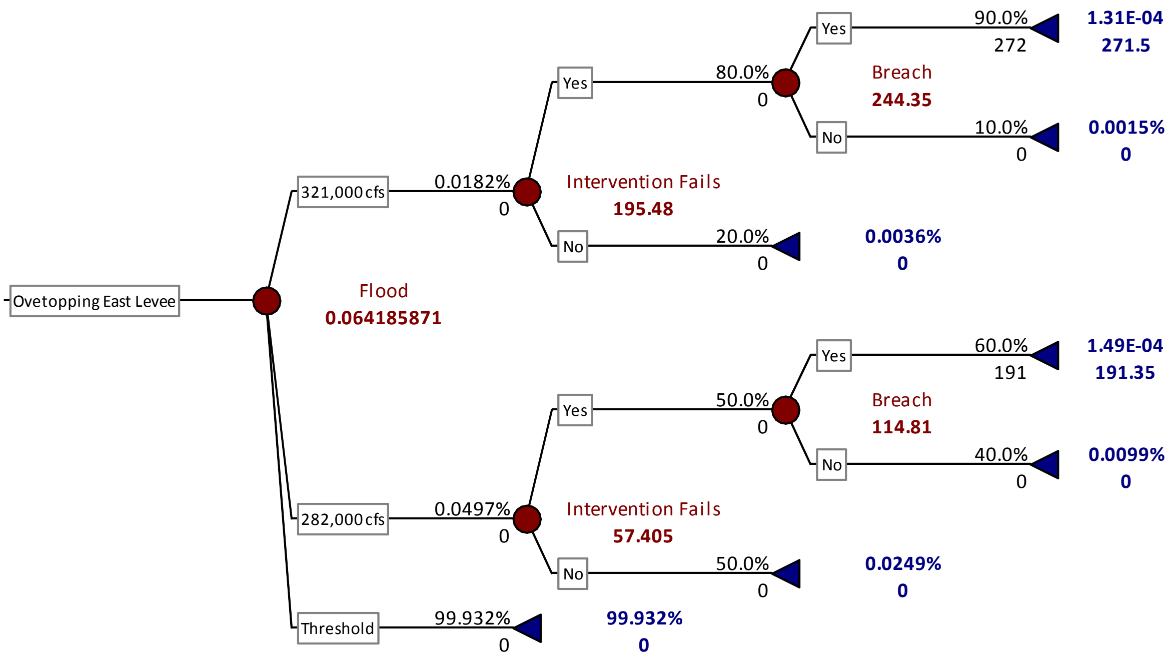
Figure 8 - Example Stage-Hydrograph used in the risk analysis

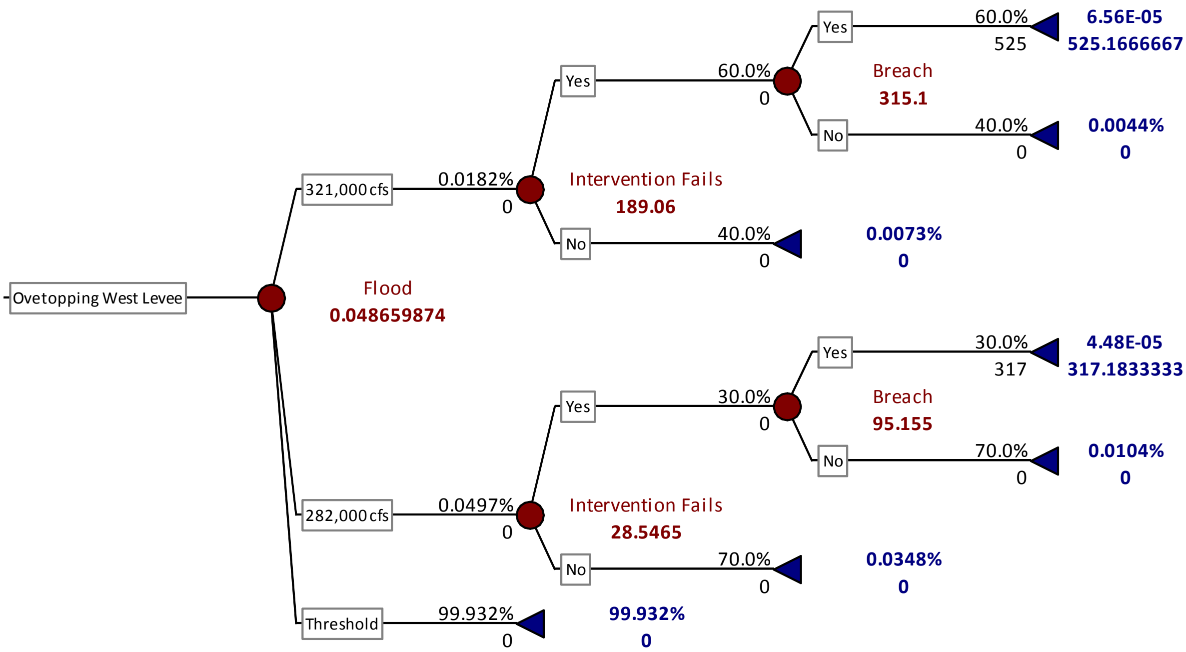
PFM #2 – Overtopping of the East and West Levee Embankments

The potential for failure of the levee embankments due to a large flood where the river stage exceeds the crest height of the levee embankment, resulting in overtopping and erosion breach of the embankment, was evaluated by the team.

Initiating Event
The initiating event would be a flood that overtops the East or West Levee embankment. Two overtopping ranges were evaluated in the event tree, as described below.

Event Tree
The event tree used to evaluate overtopping of both the East and West Levee embankments is shown in the following figure. The two flood load ranges represent two levels of overtopping; minor overtopping and major overtopping. The threshold of overtopping corresponds to a river flow of approximately 232,000 ft3/s. The first overtopping flood load range represents loading from the threshold of overtopping up to about 1.3 (West) to 2.2 (East) feet of overtopping. Based on the hydrographs previously discussed, overtopping would occur for about 15 (West) to 24 (East) hours under this scenario. The second flood load range represents overtopping depths greater than about 1.3 to 2.2 feet. Overtopping could exceed 30 to 40 hours for this scenario. The event tree evaluates two conditions for each load range; the likelihood that intervention would be unsuccessful and given that it was unsuccessful, the likelihood that an erosion breach of the embankment would occur.



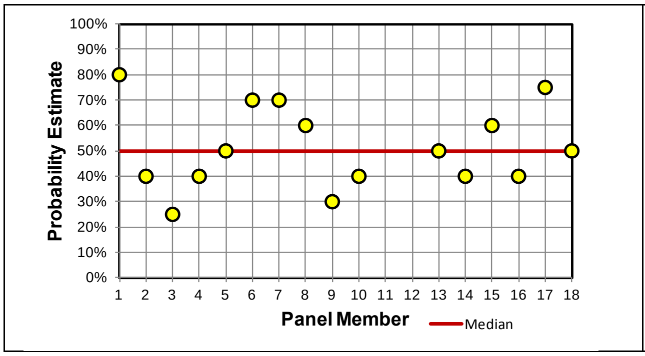


The estimates for each node and load range are discussed in the following paragraphs.

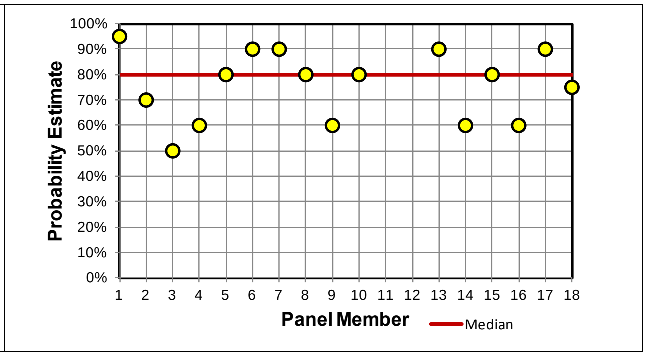
Unsuccessful Intervention

The possibility of using a road grader or motor patrol to “blade up” the crest of the embankment to prevent overtopping was raised. The discussion of this node focused on the relative ease of intervening between the East and West side as well as between the lower and upper flood load ranges. Since the crest is higher on the West side, it would take less effort to mitigate this side. For the higher flood load range a major effort would be required, as the length of levee that would need to be raised is about twice that of the lower flood load range, and the embankment raise would need to be about twice that of the lower flood load range. The team estimates, summarized in the following table, generally reflected this discussion. However, in general there is significant uncertainty, with intervention unlikely (unsuccessful intervention likely) at the upper load range for the East side, ranging to somewhat likely (unsuccessful intervention unlikely) at the lower flood load range for the West side.

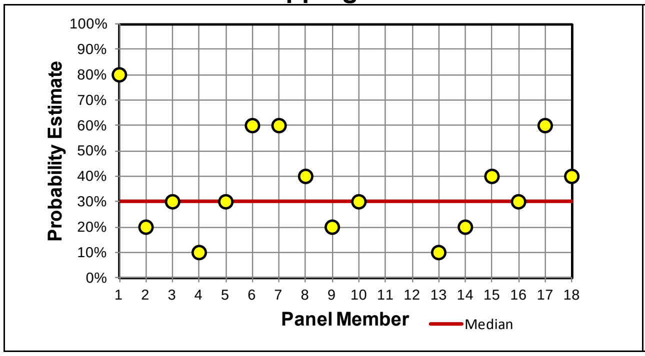
Case	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East Side Minor O.T.	0.25	0.4	0.5	0.51	0.16	0.80
East Side Major O.T.	0.50	0.60	0.80	0.76	0.14	0.95
West Side Minor O.T.	0.10	0.30	0.30	0.36	0.19	0.80
West Side Major O.T.	0.40	0.50	0.60	0.62	0.14	0.95



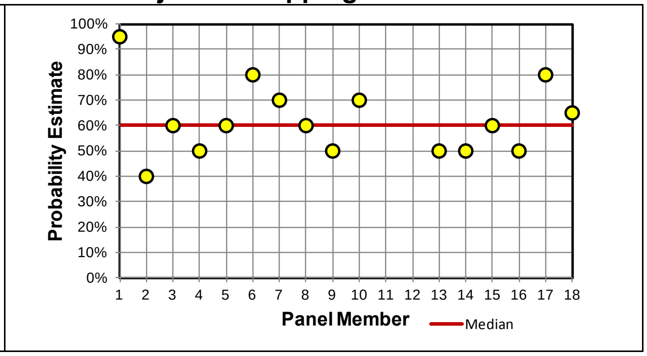
Minor Overtopping – East Levee



Major Overtopping – East Levee



Minor Overtopping – West Levee

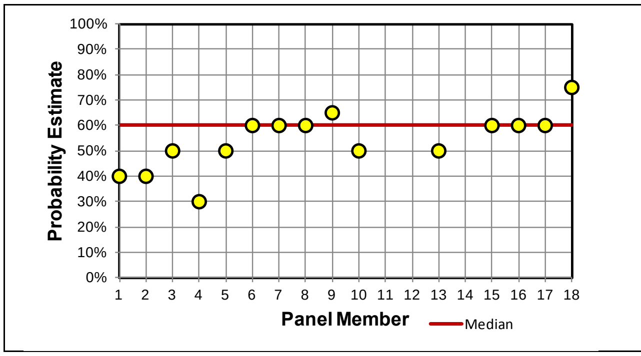


Major Overtopping – West Levee

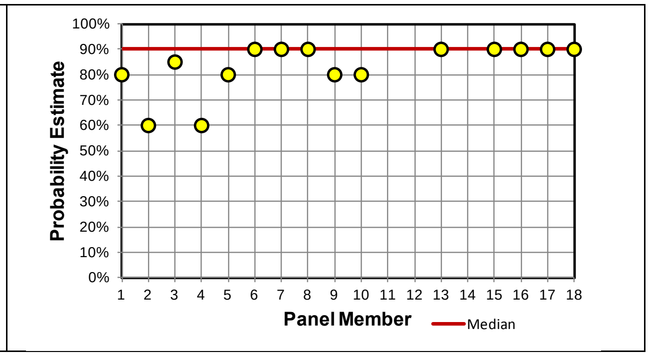
Progression to Failure

The team discussed adverse and favorable factors related to this node. The clay embankment material is generally erosion resistant, but Johnson grass clumps and desiccation cracks could create nick points for headcutting erosion. In the end, the team focused on the depth and duration of overtopping, both being fairly small for the West Levee and lower flood load range, and both being fairly large for the East Levee and higher flood load range. The estimates generally ranged from generally unlikely for the former to generally likely for the latter, with conditions in between being more uncertain.

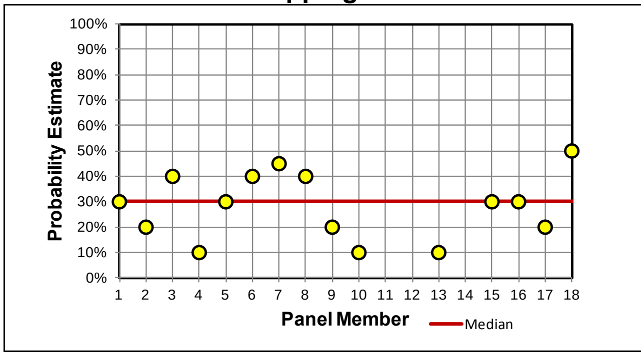
Case	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East Side Minor O.T.	0.30	0.60	0.60	0.54	0.11	0.75
East Side Major O.T.	0.60	0.90	0.90	0.83	0.10	0.90
West Side Minor O.T.	0.10	0.30	0.30	0.28	0.13	0.50
West Side Major O.T.	0.30	0.60	0.60	0.54	0.12	0.75



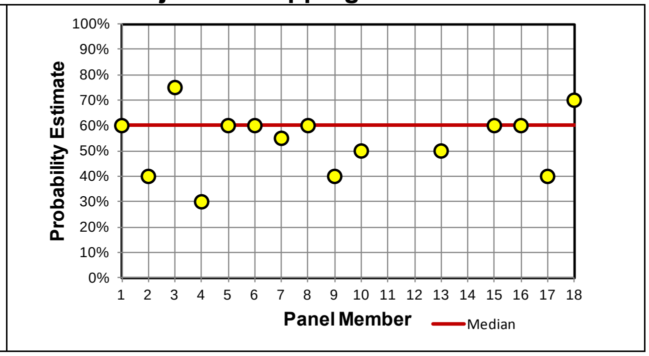
Minor Overtopping – East Levee



Major Overtopping – East Levee



Minor Overtopping – West Levee



Major Overtopping – West Levee

Consequences

Consequences were estimated for breach of both the East and West Levee embankments separately. The methods have been previously described. The results are summarized in the following table.

Location and Loading	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
East Minor O.T.	39	16	129	76	1382	186	192
East Major O.T.	61	23	175	100	2470	295	311
West Minor O.T.	31	50	21	331	579	886	320
West Major O.T.	68	99	427	615	928	1363	562

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below. The risks exceed tolerable risk guidelines for dams.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee	2.8E-04	6.4E-02
West Levee	1.1E-04	4.9E-02

The failure probabilities and risks are similar for the two load ranges for both the East and West Levees. The somewhat higher failure probability at the lower overtopping level (driven by the threshold flood frequency) is offset to a certain extent by the higher consequences at the higher overtopping level. The results are most sensitive to the flood frequency and the consequences. The estimated consequences are quite high for relatively short embankments. They are controlled to a large extent by the percentage of individuals who are assumed not to evacuate even though they have enough warning time to do so. See the discussion on consequences for a discussion of these assumptions.

PFM #3 – Overtopping of the Concrete Floodwall

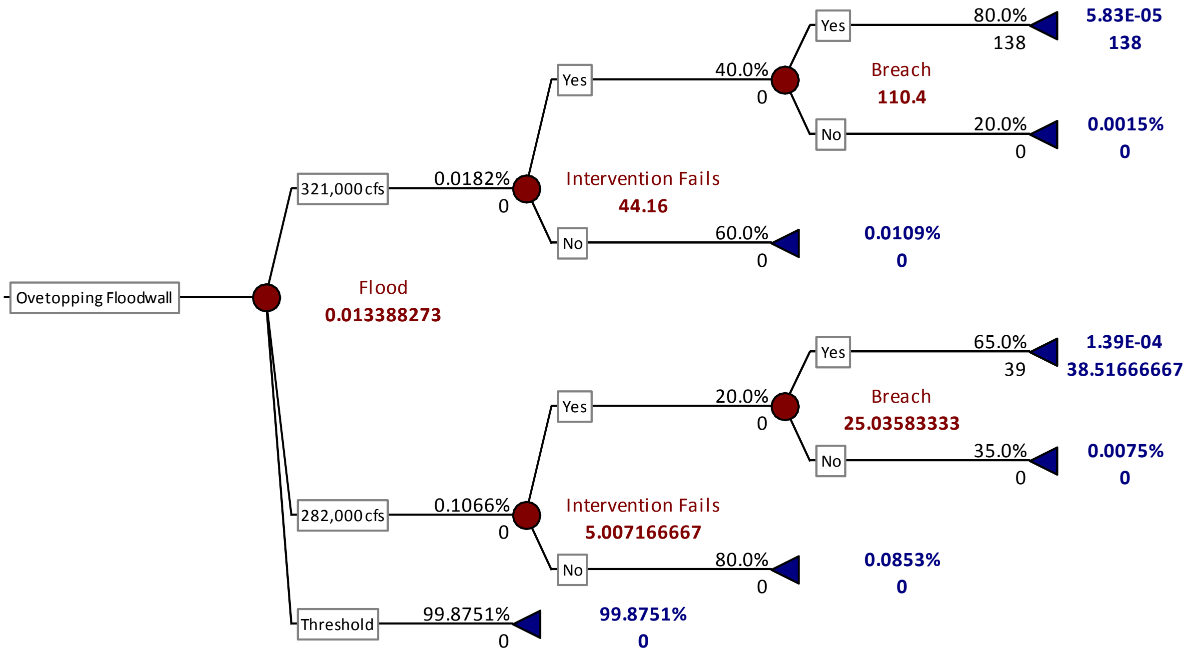
The crest of the concrete floodwall is lower than the adjacent embankment by over a foot. The plan is to replace the wall with a levee embankment extension. However, since the wall is still currently in place, the team evaluated the risks in this area due to overtopping of the wall.

Initiating Event

The initiating event would be a flood that overtops the concrete floodwall. The same two overtopping ranges that were used for the levee embankments were evaluated in the event tree, described below.

Event Tree

The event tree used to evaluate overtopping breach of the floodwall is similar to that used for the levee embankments, and is shown in the following figure. The threshold of overtopping in this case corresponds to a river flow somewhat less than 232,000 ft3/s. Thus, the overtopping depths and durations would be somewhat greater than for the East Levee embankment. The two conditions evaluated for each load range include the likelihood that intervention would be unsuccessful and given that it was unsuccessful, the likelihood that an undermining erosion breach of the wall would occur. A review of the wall design indicates the wall is most likely to fail by undermining erosion before it would fail from structural instability.

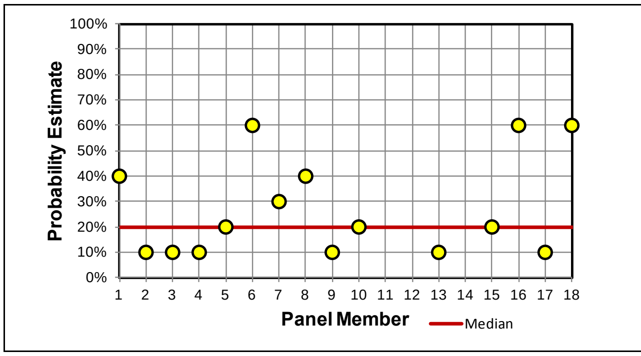


The estimates for each node and load range are discussed in the following paragraphs.

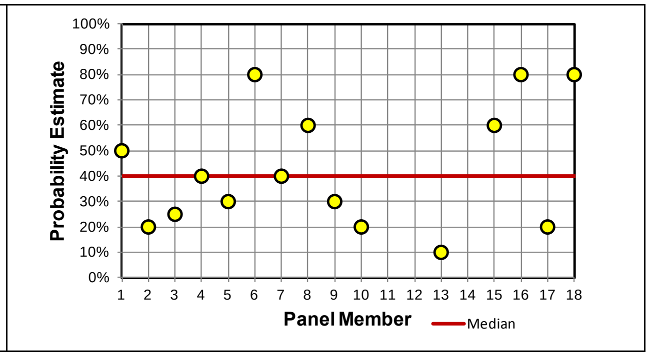
Unsuccessful Intervention

The short wall (~ 6 feet high), the good access to the protected side of the wall, and the relatively short distance that would need to be protected (~ 1,000 feet) were the primary reasons the team felt intervention would have a better chance of succeeding here than for the embankments. Dumping of large rip-rap stone on the protected side toe of the wall would likely be an effective mitigation strategy to prevent failure by undermining erosion. Nevertheless, the possibility that a large local thunderstorm could move in quickly and overtop the wall before intervention could be taken led to a fair amount of uncertainty in the estimates, especially at the higher flood loading, as summarized in the following table.

Case	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
Minor Overtopping	0.10	0.10	0.20	0.17	0.19	0.60
Major Overtopping	0.10	0.20	0.40	0.43	0.23	0.80



Minor Overtopping

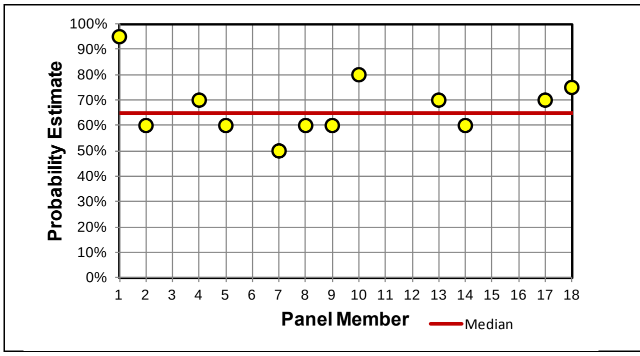


Major Overtopping

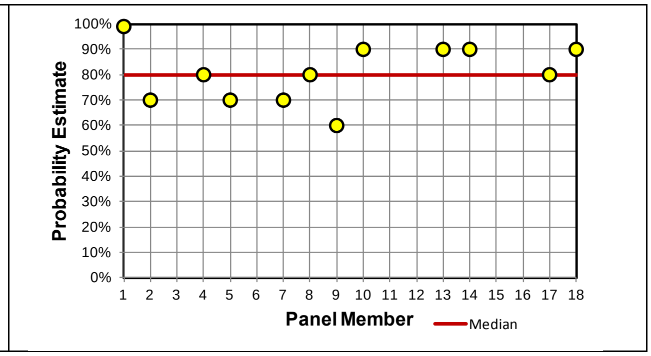
Progression to Failure

Although a layer of somewhat erosion resistant clay overlies the toe of the wall on the protected side, the team was not confident the wall would survive overtopping of the depths and durations considered, especially at the higher overtopping flows. The main factors contributing to this assessments were related to the wall footing and key which are skewed to the river side, and thus not in a good location to resist erosion. In addition, it was doubtful any sort of tailwater would build up as it would be more likely to flow into the sump away from the wall.

Case	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
Minor Overtopping	0.50	0.60	0.65	0.68	0.11	0.95
Major Overtopping	0.60	0.98	0.80	0.81	0.11	0.99



Minor Overtopping



Major Overtopping

Consequences

Consequences were estimated for breach. The methods have been previously described. The results are summarized in the following table.

Loading	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
Minor Overtopping	6	3	22	16	292	41	39
Major Overtopping	35	15	124	82	1148	190	176

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below. The risks exceed tolerable risk guidelines for dams.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee	2.0E-04	8.1E-03

Although the failure probability is higher for the lower flood range due to the frequency of the threshold flood, the risks are higher for the upper flood load range due to the rather large increase in consequences at that level. As with the other overtopping scenarios, the results are most sensitive to the flood frequency and the consequences. The estimated consequences are quite high for the relatively short wall. They are controlled to a large extent by the percentage of individuals who are assumed not to evacuate even though they have enough warning time to do so. See the discussion on consequences for a discussion of these assumptions.

PFM #7 – Internal Erosion

For a levee section to fail, the following events must occur in sequence. This process is modeled with the event tree progression listed below and detailed in the Internal Erosion chapter in the Best Practices in Dam Safety Risk Analysis[7]. This sequence was used to evaluate the internal erosion failure modes. The team added a node at the beginning, as the existence of a continuous sand layer was required to make this particular failure mode plausible.

[7] http://www.usbr.gov/ssle/damsafety/Risk/methodology.html

- A continuous Clean sand layer exists
- The river elevation rises
- There is a sufficient gradient to erode sand
- Intervention is unsuccessful
- The levee embankment forms a roof allowing erosion to progress
- Heroic intervention fails
- The levee breaches

For each event in the sequence, each team member was asked to estimate the probability of that event occurring. The range of estimates were used to create a distribution to describe the likelihood of the event happening.

The team determined that the most likely location for internal erosion to occur is where the basal sand layer exits on a free face on the land side, as interpreted by borings. This will occur in the area having the shortest seepage path and shallowest sand layer elevation, likely in the leveed side sump. Other that have variations in the elevation of the basal sand layer and distance to the river areas would likely have significantly lower risks.

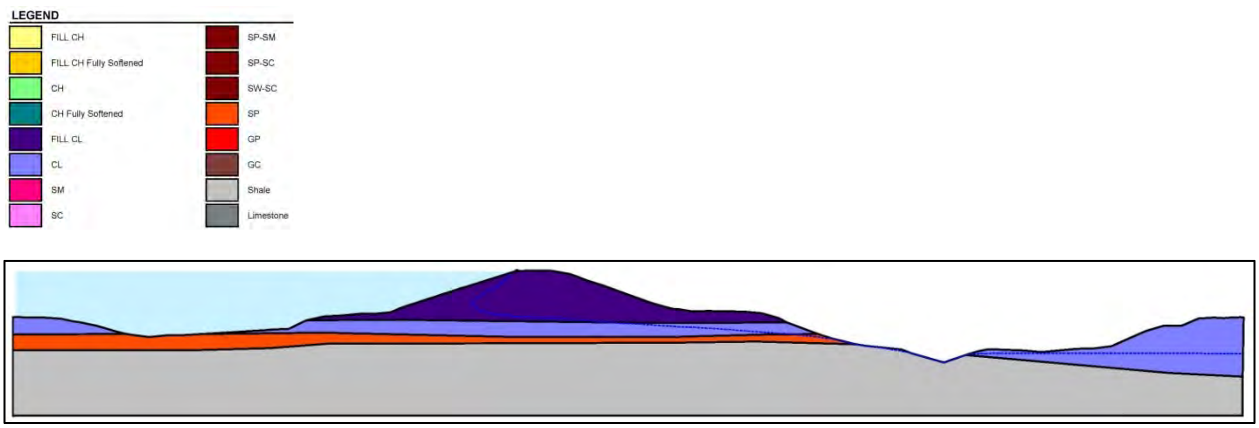
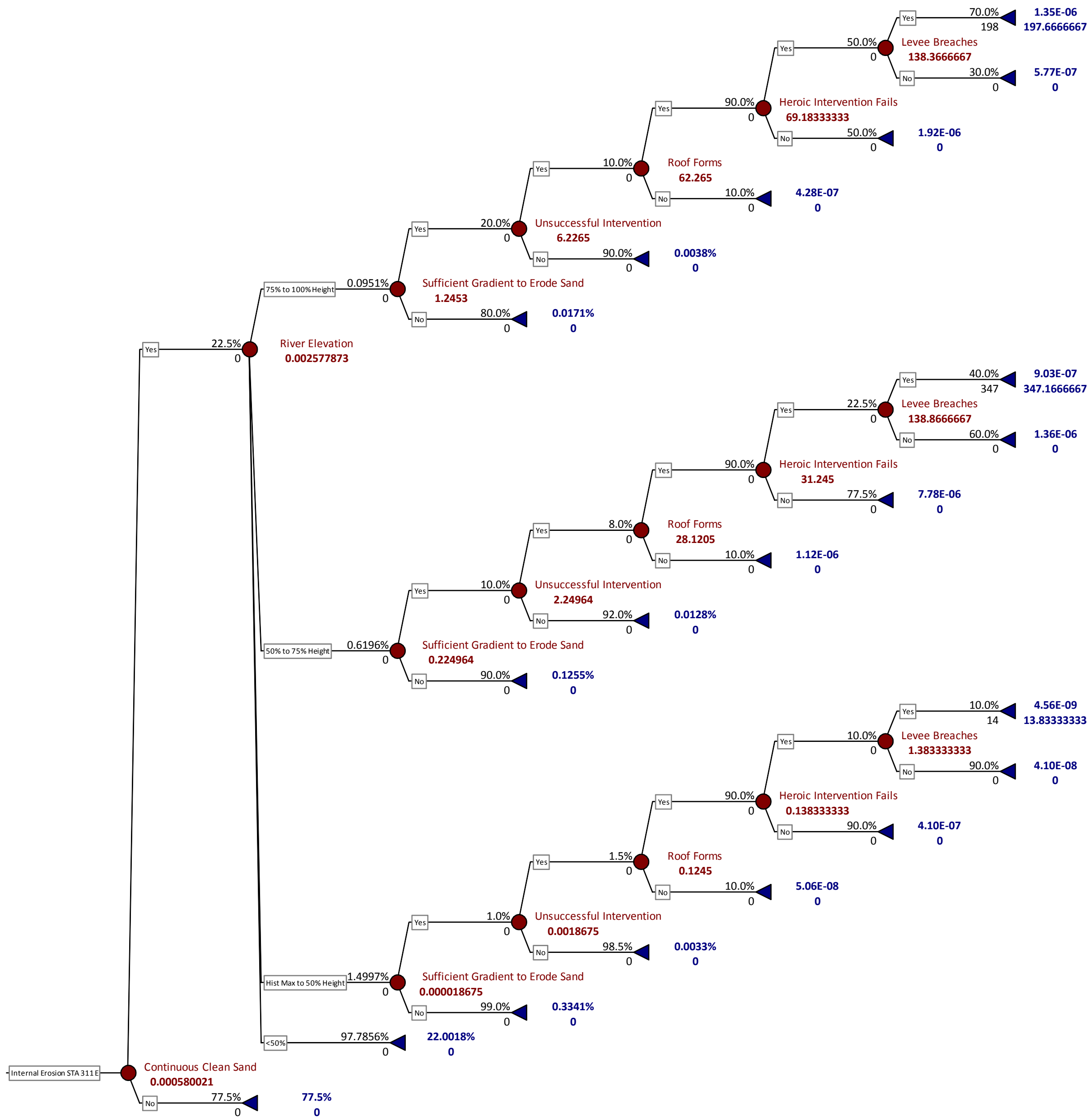


Figure 9 - Levee Section

Initiating Event

The initiating event for this failure mode is a flood that at least gets above the historical maximum river stage. During the historical maximum, no poor performance was observed in the East Levee system related to internal erosion. The team evaluated risks at 50% height, 75% height, and at the top of the levee.

Event Tree

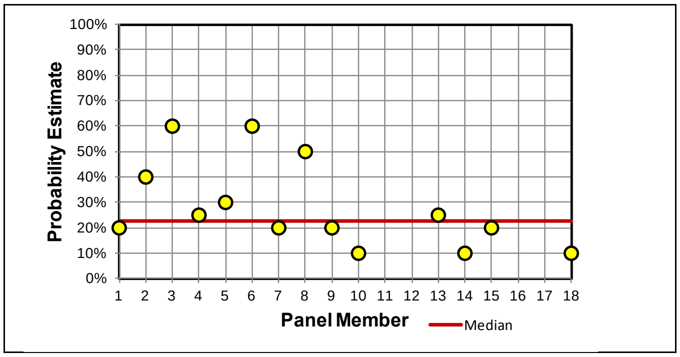


Event Tree

Continuous Sand Layer

Continuity might be more likely where gravel pits were developed and in the location of historical river crossings. The boring logs indicate a clean sand layer may include gravels and clays. Nearby piezometers indicate little response to the river stage. The team determined that continuity was unlikely in the section examined.

River Stage	Low	Median	High
All	0.10	0.23	0.6

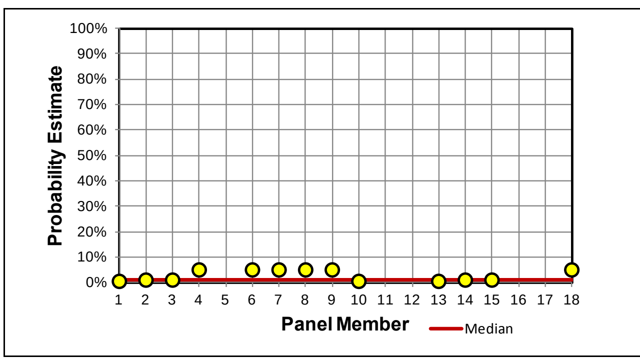


Continuous Sand Layer

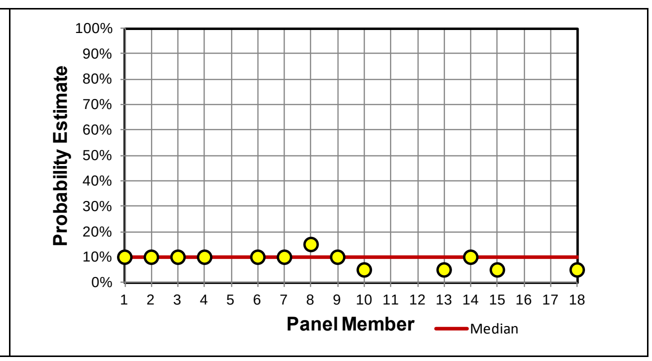
Sufficient Gradient to Erode Sand

The maximum gradient was calculated at 0.12. The presence of a terrace deposit could result in finer sands being on top of the layer which would be more erodible. However, the sand in the location of study is likely coarse. The critical gradient is 0.6, which is still higher than the calculated gradient of 0.12. The team determined that the sand would be unlikely to very unlikely to erode, but that erodibility is dependent on river stage.

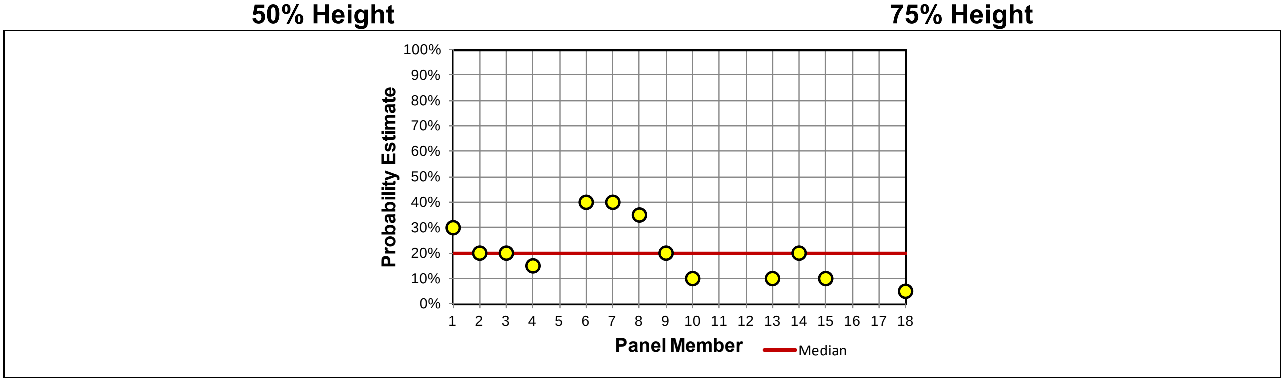
River Stage	Low	Median	High
100% Height	0.05	0.2	0.4
75% Height	0.05	0.1	0.15
50% Height	0.005	0.01	0.05



50% Height



75% Height

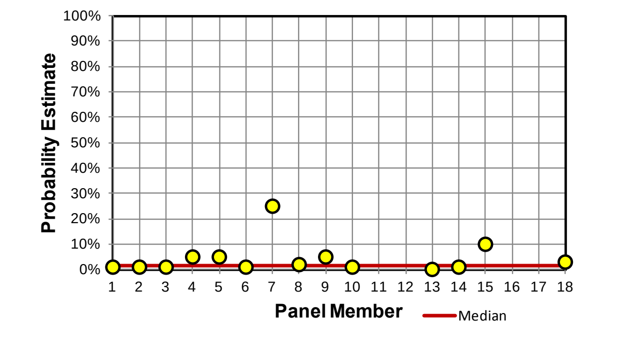


100% Height

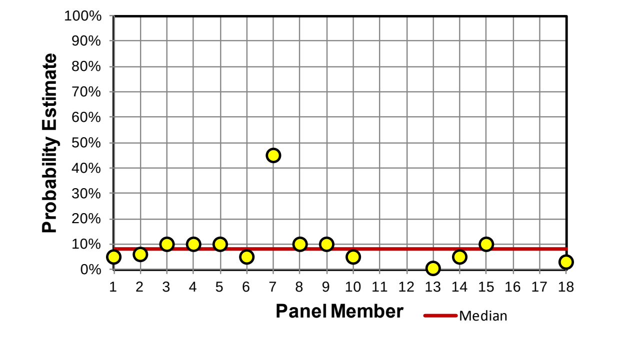
Unsuccessful Early Intervention

This area is near a pump station making it more likely that anomalous behavior would be observed. There is good access on the land side. If erosion began, it would occur in the side slope of the sump which is more observable. The team felt the ease or difficulties intervening early in the progression of erosion would be similar to PFM#8 and elected to use the same estimates.

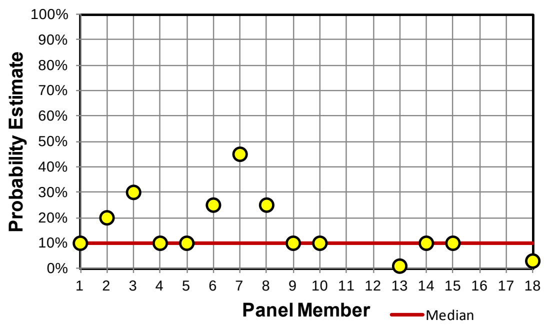
River Stage	Low	Median	High
100% Height	0.01	0.1	0.45
75% Height	0.005	0.08	0.45
50% Height	0.001	0.015	0.25



50% Height



75% Height

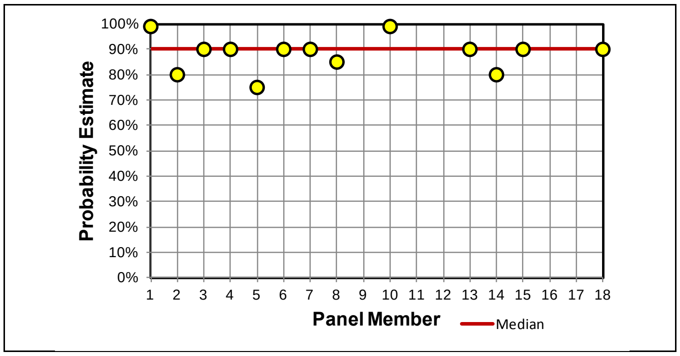


100% Height

Roof Forms

Examining the information, the team felt that the estimates from PFM #8 should be used. Although there is a shorter path, the path is still very long.

River Stage	Low	Median	High
All	0.75	0.90	0.99

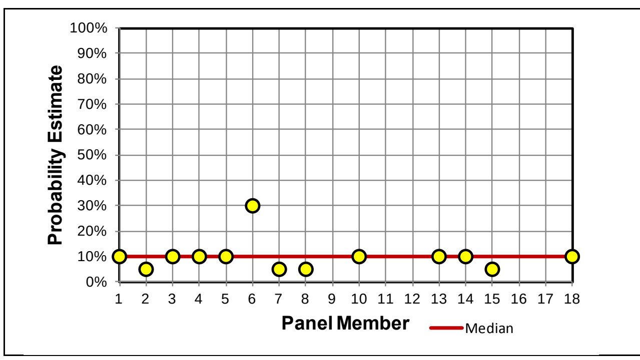


Roof Forms

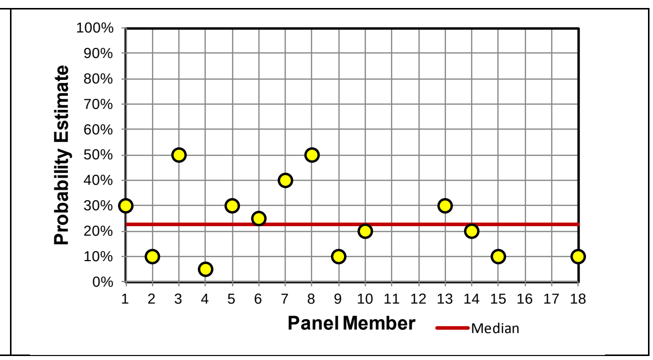
Heroic Intervention Fails

The team compared this location with that of a previous failure mode. Access is better in this location, and it might be possible to take some action on the river-side by dumping material. However, the team elected to use the same estimates as PFM #8 although it was recognized that the values could be slightly lower here.

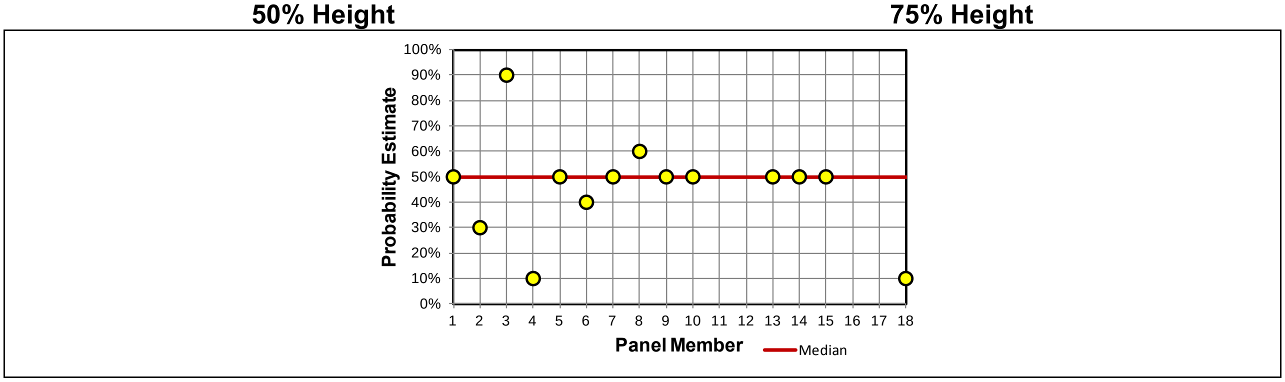
River Stage	Low	Median	High
100% Height	0.10	0.50	0.90
75% Height	0.05	0.23	0.50
50% Height	0.05	0.10	0.30



50% Height



75% Height

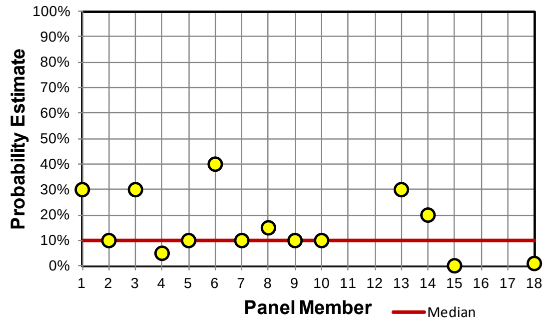


100% Height

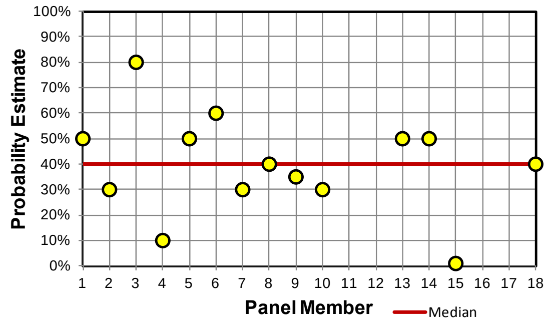
Levee Breaches

The team compared this location to a location from a previous failure mode. In this location, the sand layer is closer to the embankment and the crest elevation is 1.4 feet lower based on the 2003 survey. However, the sand layer is not as thick in this location. None of these factors were determined to be significant, and the team elected to use the same estimates from PFM #8.

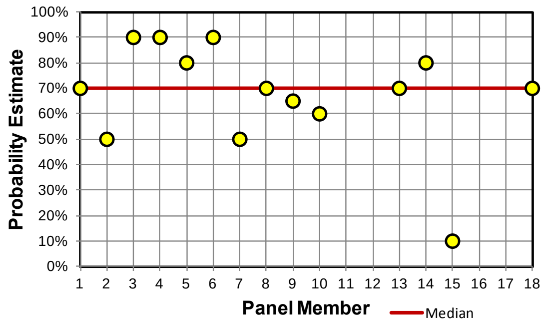
River Stage	Low	Median	High
100% Height	0.10	0.70	0.90
75% Height	0.01	0.40	0.80
50% Height	0.001	0.10	0.40



50% Height



75% Height



100% Height

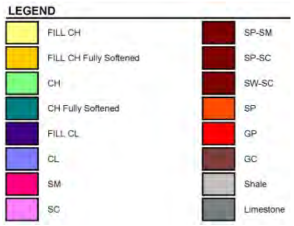
Consequences

Consequences were estimated for breach of both the East and West Levee embankments separately. The methods have been previously described. The results are summarized in the following table.

Loading	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
100% Height	19	7	90	46	1,451	172	66
75% Height	77	18	222	43	4,992	500	124
50% Height	0	1	4	6	22	14	5

Results

The team compared the cross-section, materials, and location with the location from a previous failure mode. The team felt there was no significant difference between the situations and elected to use the same probability estimates for this failure mode.



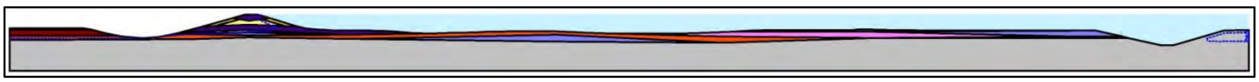


Figure 10 - Levee Section including river

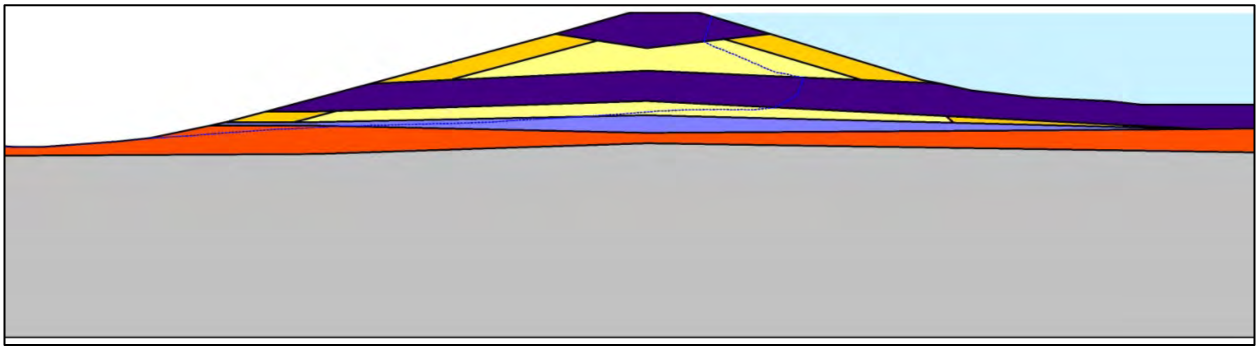
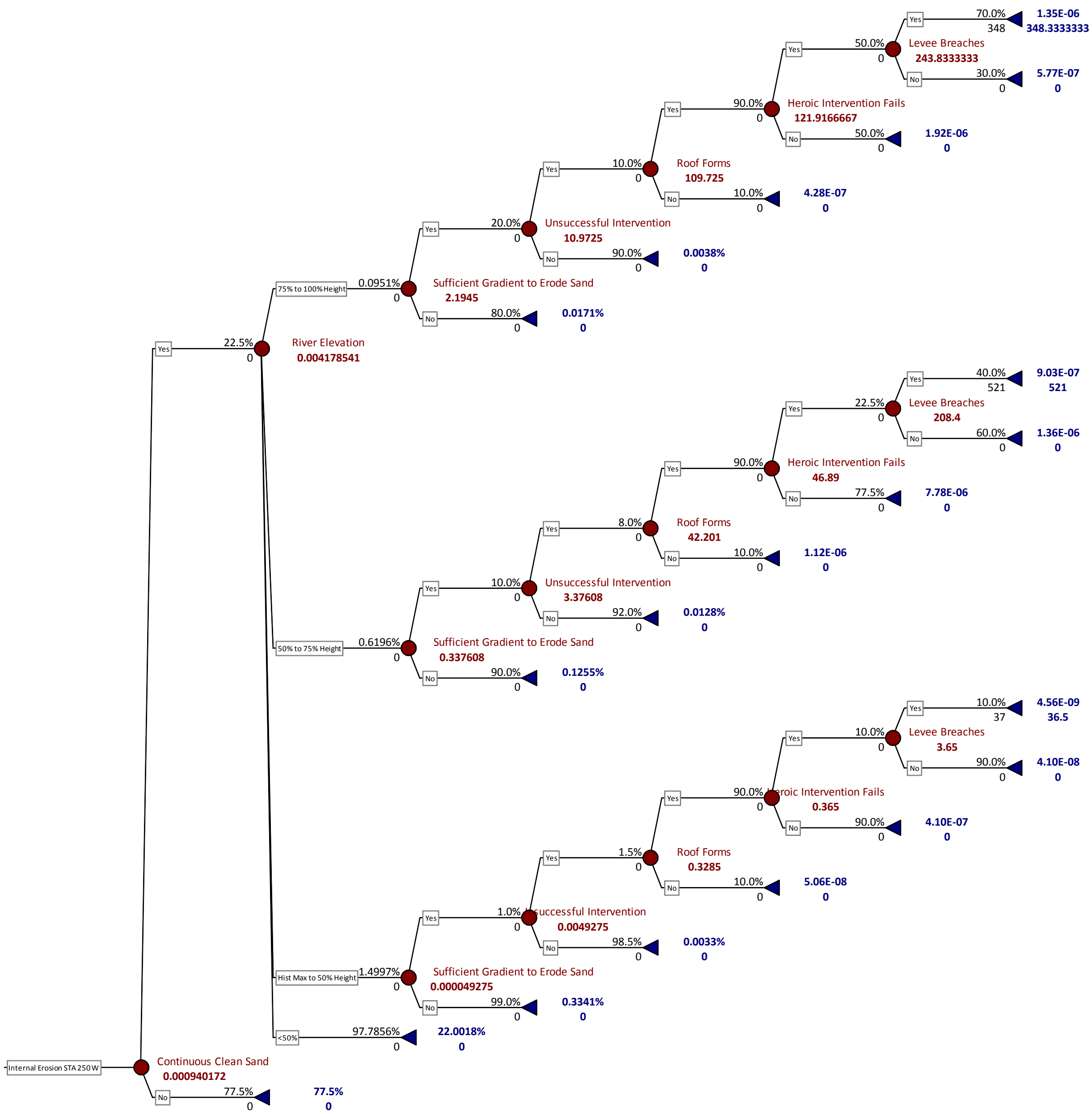


Figure 11 - Levee Section



Event Tree

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below. The risks are at the tolerable risk threshold.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee	2.3E-06	5.8E-04
West Levee	2.3E-06	9.4E-04

PFM #8 – Heave of the East Levee

There are locations on the East and West levee systems where a pervious basal sand layer exists on top of the foundation rock and are overlain by an impervious clay cap. In those locations, it is possible that foundation pressures could overcome the weight of the soil above, heave the soil and allow seepage to exit and eventually lead to breach of the system. In this scenario the following events would need to occur in order for a breach of the levee to result.

- There is a continuous sand layer connected to the river that allows water pressures to build up in the foundation
- The foundation pressures exceed the weight of the impervious cap causing the foundation to heave on the land side
- Early intervention is unsuccessful if the heave is observed
- The gradient is sufficient to move the basal sands
- A roof forms and allows erosion to progress under the levee section
- Heroic intervention fails
- Erosion progresses and leads to a breach of the levee

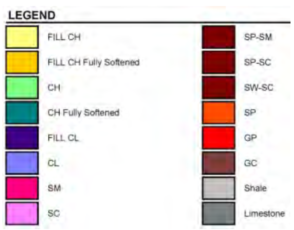


Figure 12 – East Levee Section including river

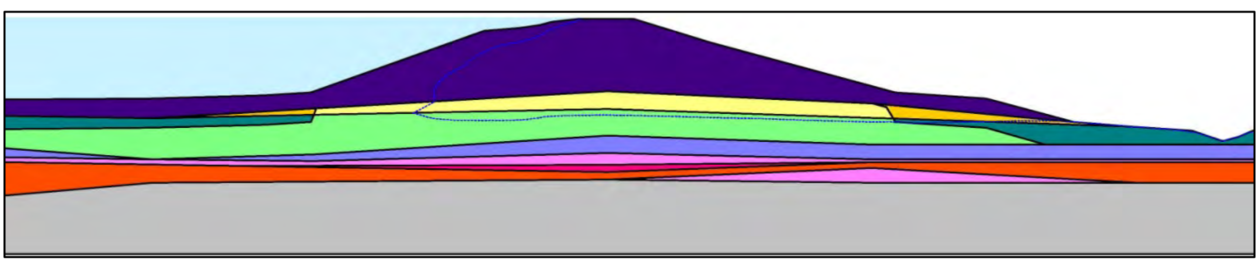


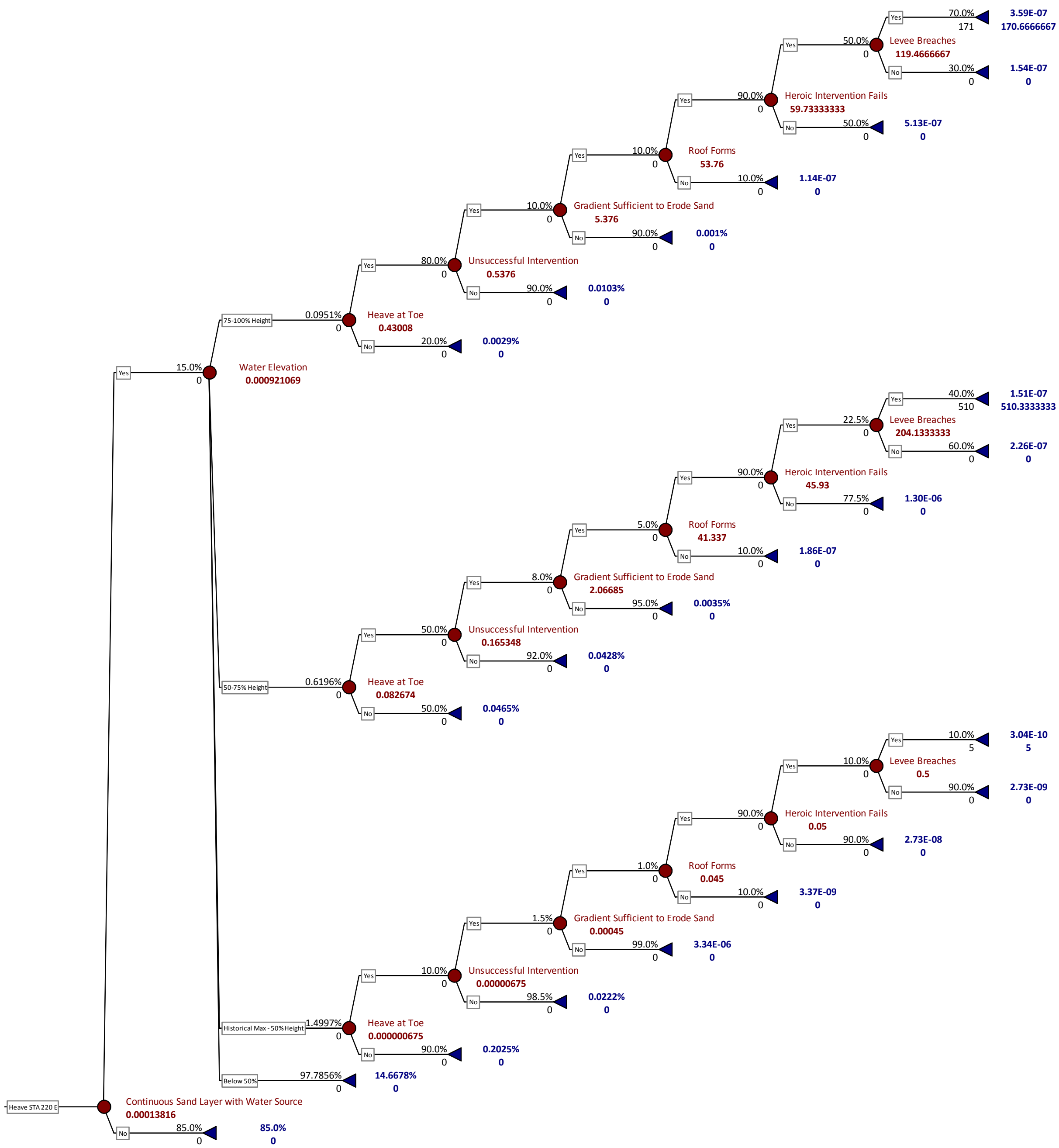
Figure 13 – East Levee Section

Initiating Event

The team discussed at what level the river stage would need to be before this potential failure mode would be of concern. It was concluded that this failure mechanism would be possible for all river stages. Probabilities were estimated for 50%, 75%, and 100% of the levee height.

Event Tree

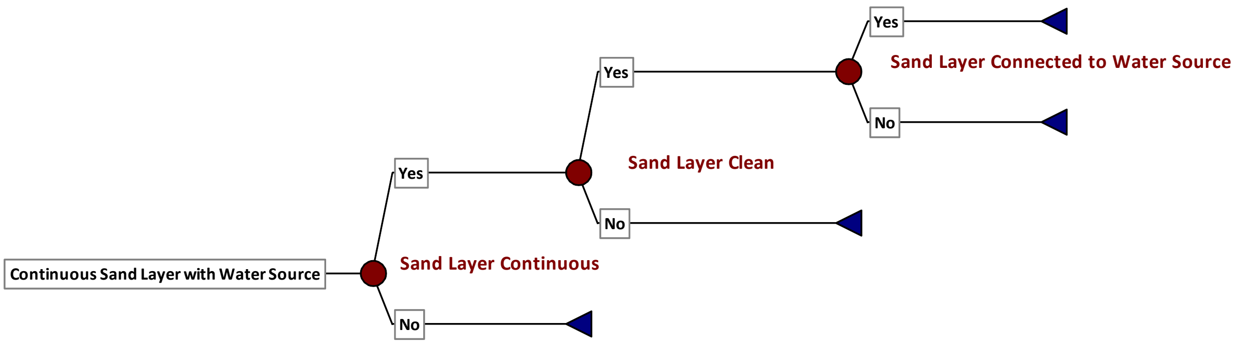
The event tree for this potential failure mode is shown below. Estimates for each branch of the event tree are discussed in subsequent sections.



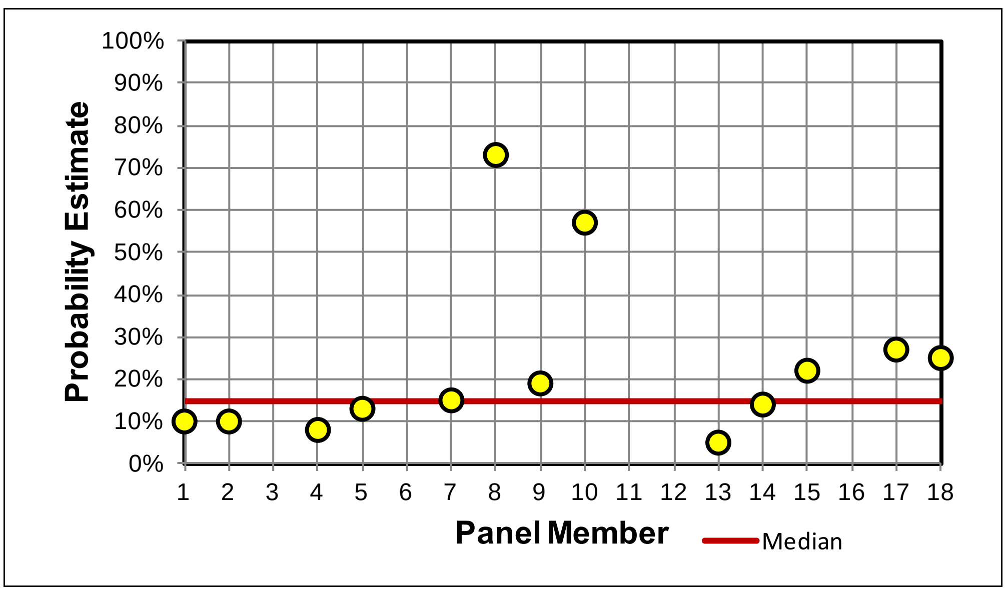
Event Tree

A Continuous Sand Layer Exists and is Connected to the River

The geomorphology of the Trinity River valley suggests that the depositional environment is conductive to large continuous sand deposits. There is ample boring coverage of the levee system which shows sand regularly. Although the sections are in two dimensions, there is likely continuity in three dimensions. Even though continuity was considered likely, several factors might limit the continuity. Gradations of the sand indicate a small percentage (5-10%) of the sands are clean. The depositional environment supports the mixing of materials and the borings indicate the sand layer may pinch out towards the land side. Piezometers also indicate a head drop from the river side to the land side piezometer in this location. Also, no seepage has been occurred to-date. Overall, there could be a circuitous path of cleaner sands beneath the levee even though evidence has not been found that has specifically indentified this feature with certainty. The team also estimated on their own the likelihoods of the following events and multiplied them together to obtain the likelihood of having a continuous sand layer with a water source.



Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East Levee	0.05	0.10	0.15	0.23	0.19	0.73



Continuous Layer

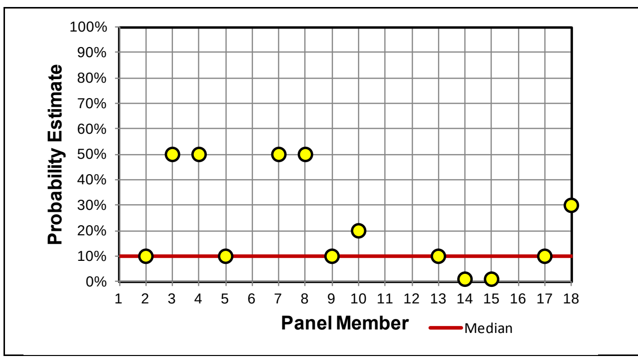
Foundation Pressures Cause Heave at the Toe

One of the reasons this particular section was thought to be critical is that the clay blanket is likely one of the thinnest in the system where a basal sand layer exists. Foundation pressures are related to the river stage, so these factors vary with river stage. Seepage analysis indicates low factors of safety against heave in this area:

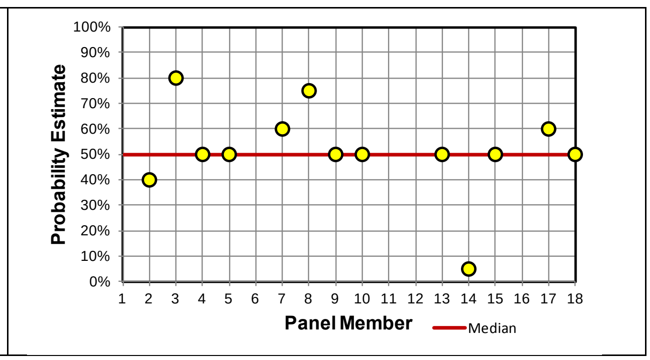
- With the river stage at 50% of the height of the levee, the factor of safety is between 0.5 and 0.6
- With the river stage at 75% of the height of the levee, the factor of safety is between 0.4 and 0.6
- With the river stage at 100% of the height of the levee, the factor of safety is between 0.4 and 0.5

The sumps are normally pumped down during flooding, so water on the land side would not be adding to the resisting forces. Historically, the flood loading has come within 2 feet of 50% of the levee height, and no problems such as increased seepage or boils were observed during that event. It's possible than natural drainage would occur in the sand layer. The clay blanket may also be thicker than it was modeled and the clay strength does not factor into the heave calculation. In the end, a lot of weight was put to the seepage analysis results because they are conditional on the continuous sand layer, but the team reduced the estimates somewhat because of the historical behavior.

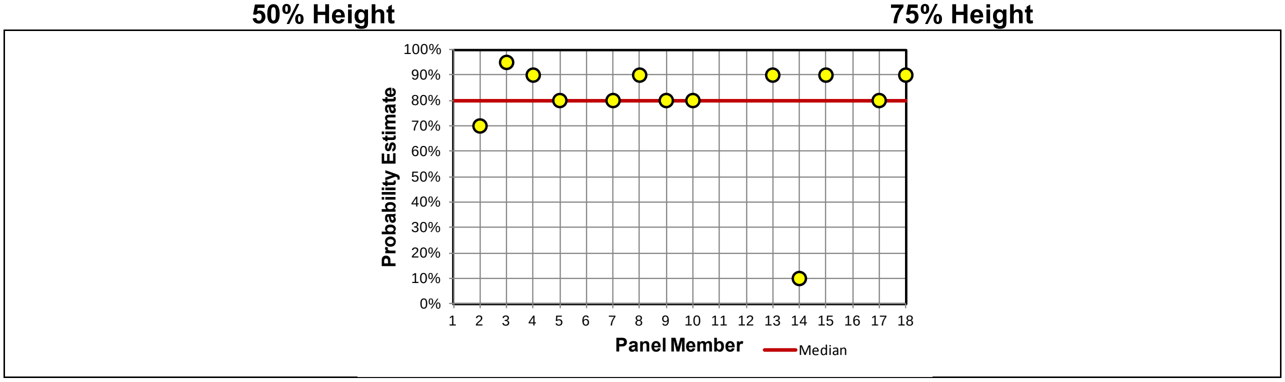
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.10	0.90	0.80	0.79	0.21	0.95
75% Height	0.05	0.50	0.50	0.52	0.17	0.80
50% Height	0.10	0.10	0.10	0.23	0.19	0.50



50% Height



75% Height



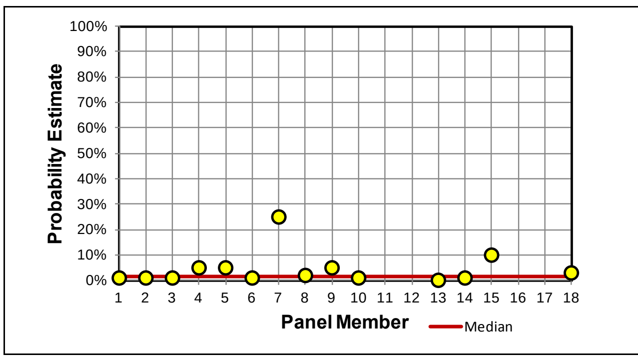
100% Height

Early Intervention is Unsuccessful

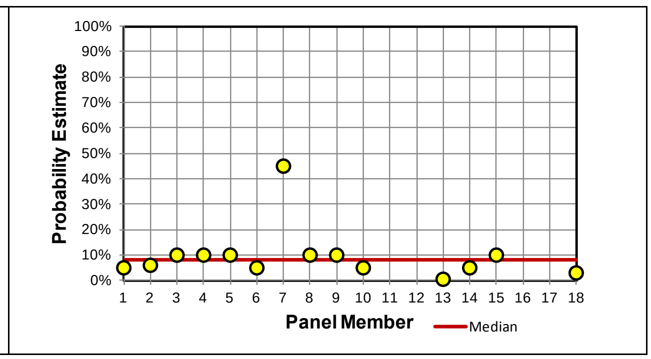
Intervention is likely to be successful at this location. Visual observation of the sumps begins once pumping begins. Mowing operations have increased the visibility at the sumps and the City of Dallas has tracked equipment. Access is excellent at the likely location of this failure mode. There are emergency stockpiles of sand and sandbags that could be used to build sand rings to reduce the gradient. The Corps has seen much success flood fighting sand boils across the country. The heads in this system are not large, so flood fighting should be successful and the boils should be obvious. The flood peak durations are relatively short (hours to days).

Despite these factors, there are other items that might indicate difficulties flood fighting. Treating a boil in one area might cause the pressures to move to another location. Visibility is less at night and vegetation and water could cause the boils to allow erosion to start unnoticed. Vigilant monitoring and abilities and potential success fighting boils were the most influential pieces of information gathered.

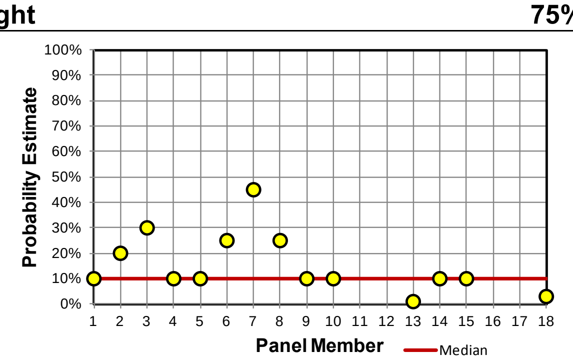
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.01	0.10	0.10	0.16	0.12	0.45
75% Height	0.005	0.10	0.08	0.10	0.10	0.45
50% Height	0.001	0.01	0.015	0.04	0.06	0.25



50% Height



75% Height



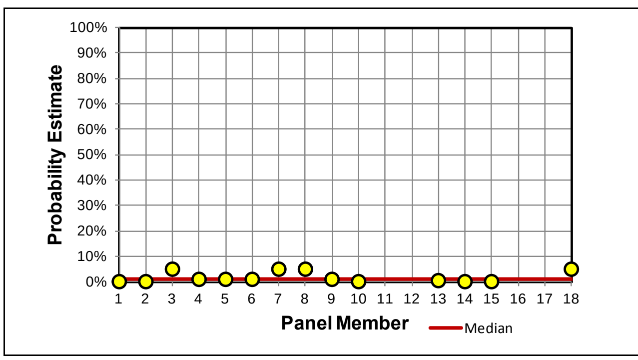
100% Height

Gradients are Sufficient to Erode the Basal Sands

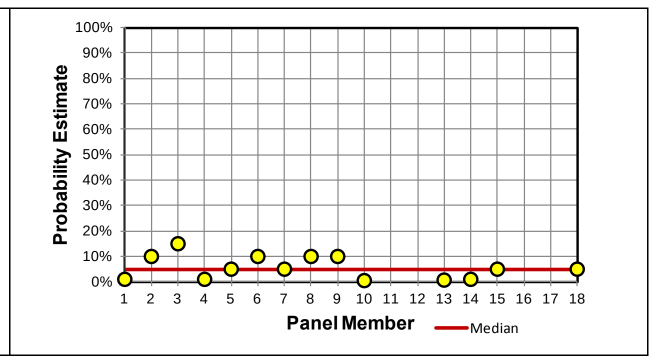
For erosion to progress, enough gradient must exist to begin to move material out of the basal sand layer so that the failure mode can progress. The gradients are unlikely to be high enough to move material at this location. The gradient is approximately 0.03 with water at the full height of the levee. The Coefficient of Uniformity calculations indicate values more than 3.0, with much of the data in the 10.0 to 20.0 range. That means the critical gradient is approximately 0.60. A large amount of sand would need to be moved to connect to the river. The gradient from the upstream toe (in the case where a flaw might exist right at the upstream toe) to the heel is approximately 0.10.

However, some of the fine sand is more erodible than the material that was assumed during the assessment. There could be a shorter seepage path at the bridge pier location and there is a lake on the river side – although the lake is shallow and likely silted in. The low average gradients and high coefficient of uniformity were the critical pieces of information evaluated by the team.

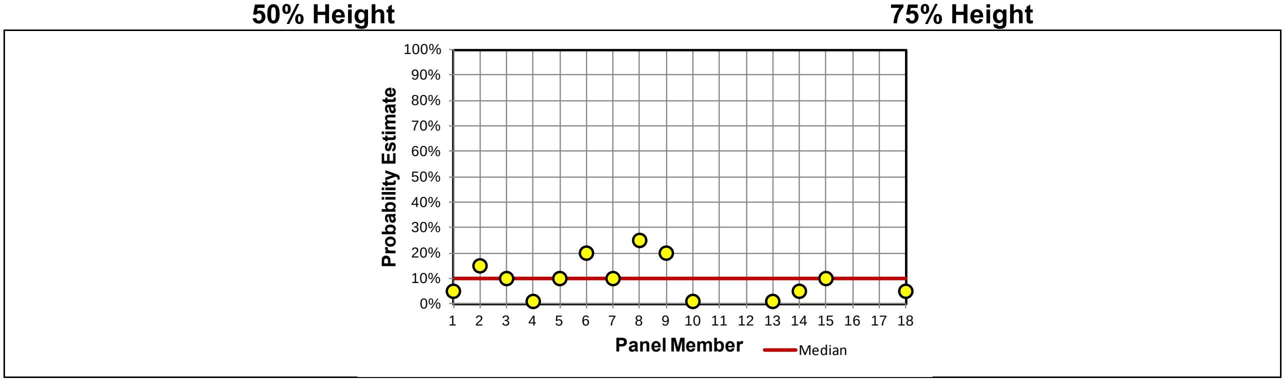
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.01	0.10	0.10	0.10	0.07	0.25
75% Height	0.005	0.10	0.05	0.06	0.05	0.15
50% Height	0.001	0.001	0.01	0.02	0.02	0.05



50% Height



75% Height

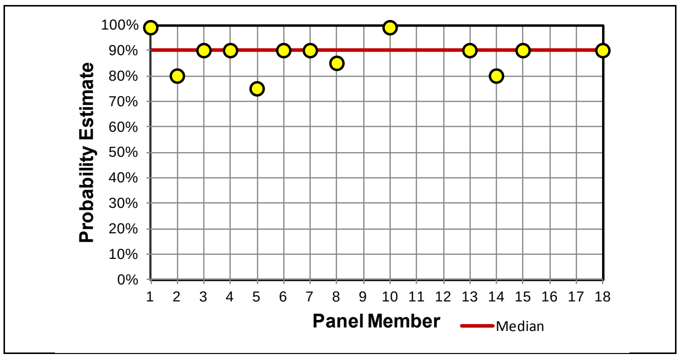


100% Height

A Roof Forms Allowing Erosion to Progress

In order for erosion to progress, a roof needs to form either in the foundation or the levee section that allows the erosion to eventually reach the river. The embankment and the foundation are both composed of clay material which is susceptible to holding a roof. In general the sand layers at this section are overlain by clay layers. Although the materials overlaying the sands may not all be clay and the fact that the roof needs to stay open for a long distance, the material composition was the primary factor behind the risk estimates in the table below.

Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East STA 220+00	0.75	0.90	0.90	0.88	0.07	0.99

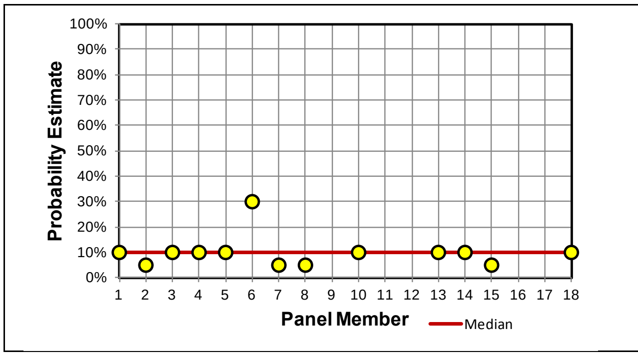


Roof Forms

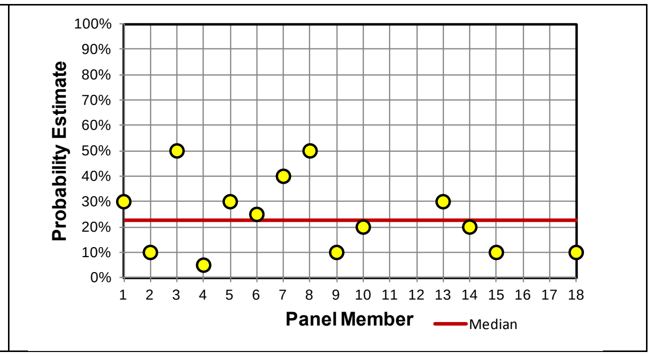
Heroic Intervention Fails

When erosion has progressed and an erosion tunnel has contacted the river, the team examined the potential to intervene and prevent a breach. A seepage berm could be built on the land side. The City of Dallas has material, equipment, and staff to do this. The pumps could be turned off and the sumps could be filled with water to help reduce the gradient. The problem would be pretty obvious and there is good access to the site. There would be no access on the river side where the seepage entrance would be and it make take time to intervene, so the team was not certain regarding the ability to intervene. Overall, the City has the ability to deal with major incidents, but the team was less certain about their potential for success at very high flood levels.

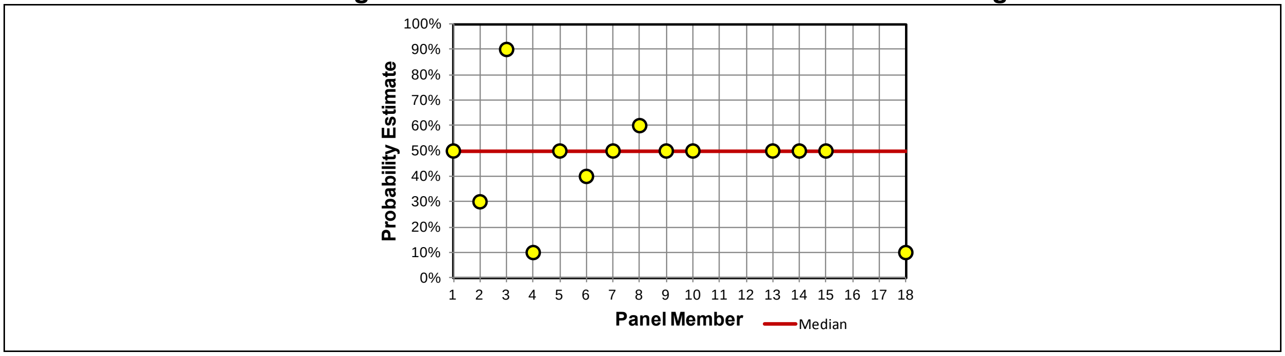
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.10	0.50	0.50	0.46	0.19	0.90
75% Height	0.05	0.10	0.23	0.24	0.14	0.50
50% Height	0.05	0.10	0.10	0.10	0.06	0.30



50% Height



75% Height

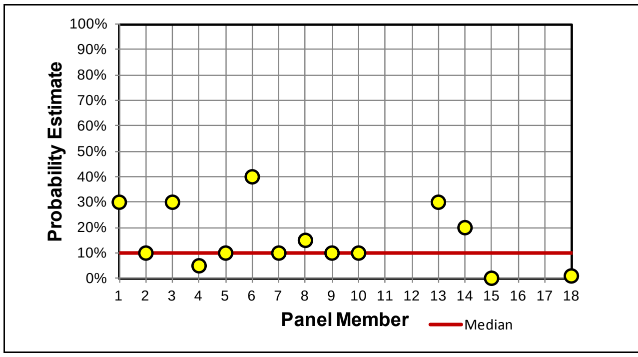


100% Height

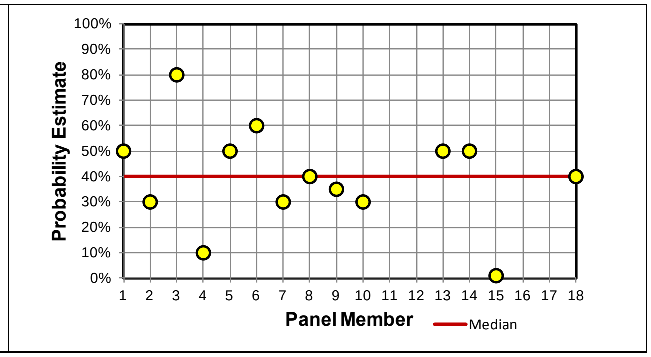
Erosion Progresses to a Breach of the Levee

If erosion has progressed and intervention failed, the team examined the likelihood of the levee breaching and catastrophically failing. The levee will likely be loaded for a short duration, the clay erodes slowly, and the smaller the flood, the less likely breach would happen. The sand layer is also relatively thin, which might limit the amount of flow and erosion. The desiccated clay might erode more quickly because of its cracked nature.

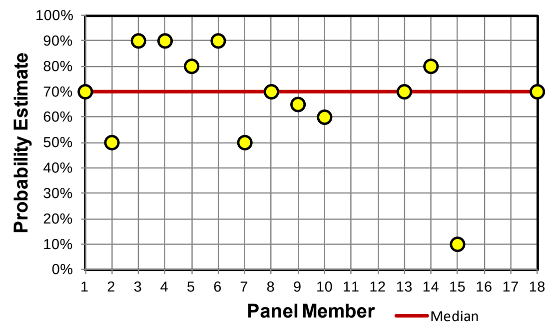
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.10	0.70	0.70	0.68	0.20	0.90
75% Height	0.01	0.50	0.40	0.40	0.19	0.80
50% Height	0.001	0.10	0.1	0.16	0.12	0.40



50% Height



75% Height



100% Height

Consequences

Consequences were estimated for breach of both the East Levee embankment. The methods have been previously described. The results are summarized in the following table.

Loading	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
100% Height	19	7	90	46	1,451	172	66
75% Height	77	18	222	43	4,992	500	124
50% Height	0	1	4	6	22	14	5

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below. The risks exceed tolerable risk guidelines for dams.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee STA 220+00	5.1E-07	1.4E-04

PFM #8 – Heave of the West Levee

The team examined the situation on the West Levee and elected to compare that to the East Levee. The team determined that the probability estimates would be identical with the exception of the two nodes listed below.

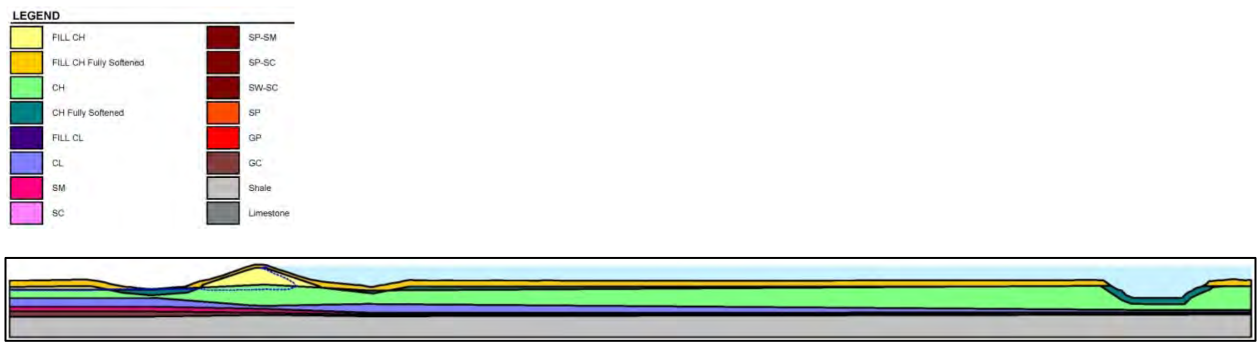


Figure 14 - West Levee Section including river

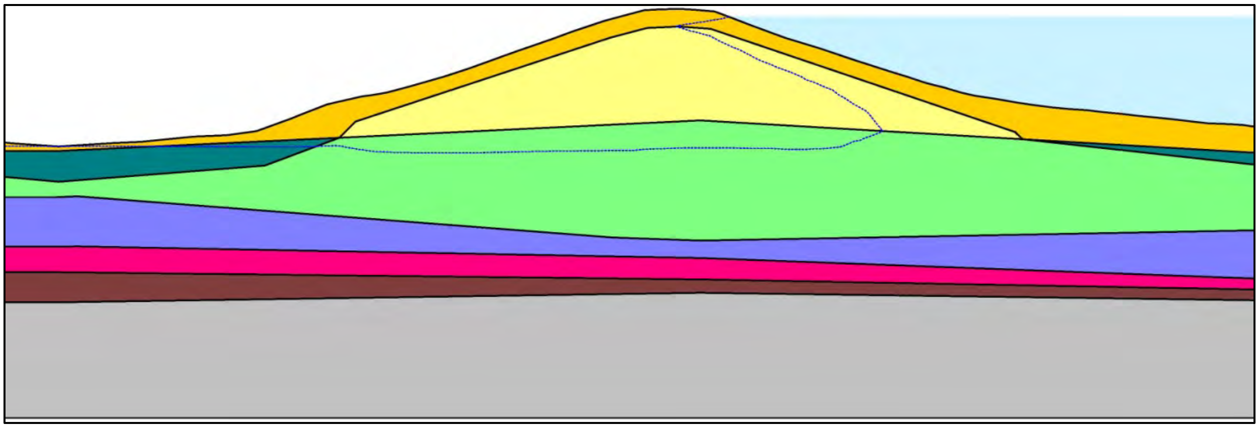
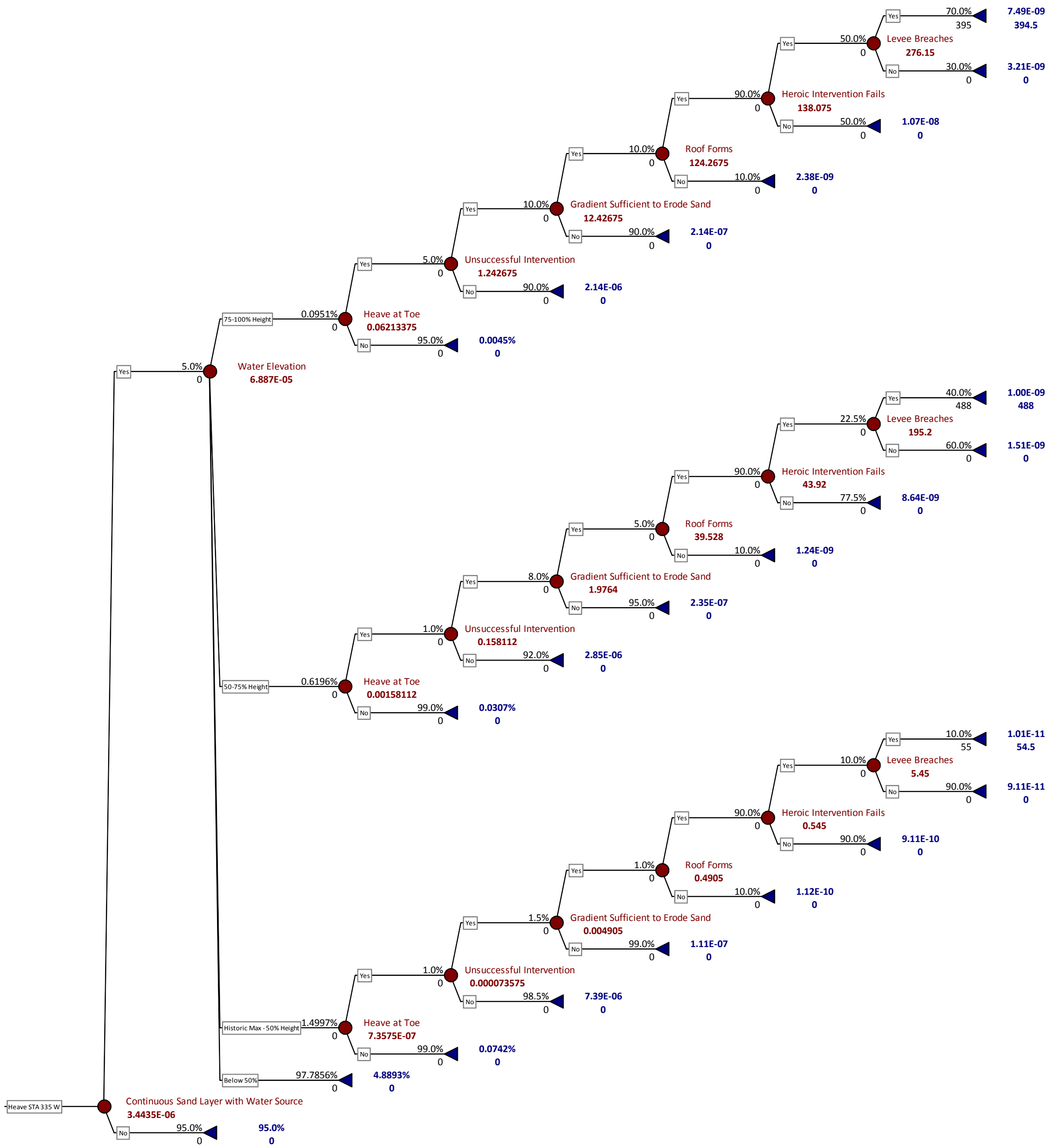


Figure 15 - West Levee Section

Event Tree



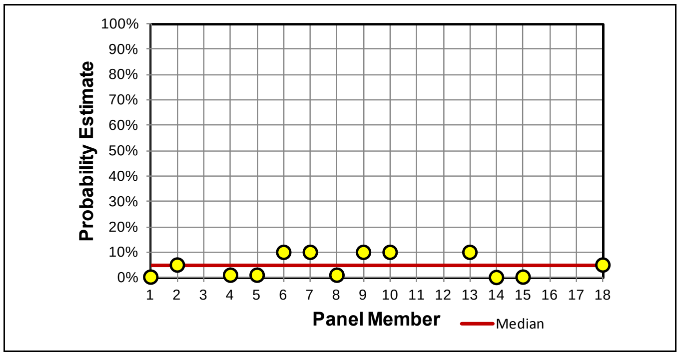
Event Tree

A Continuous Sand Layer Exists and is Connected to the River

In this location, the primary difference is that the sand layer does not daylight in the river channel. This was determined to be an important consideration. The estimates are listed below.

Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum

Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
West STA 335+00	0.001	0.10	0.05	0.05	0.04	0.10

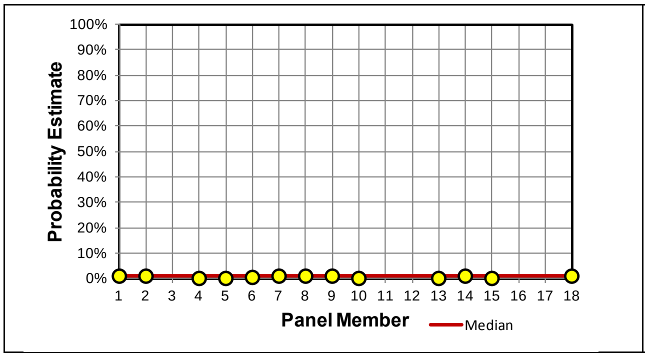


Continuous Layer

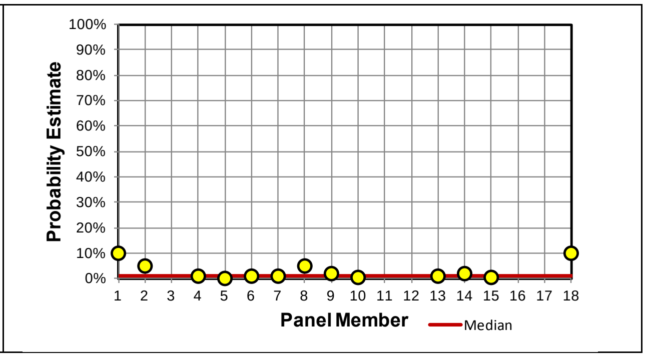
Foundation Pressures Cause Heave at the Toe

At this location, the clay layer is 30 feet thick on the land side as compared to 6 feet on the East Levee. This causes the factor of safety to be 1.3 as compared to 0.5 on the East Levee. The estimates were modified to the values listed in the table below.

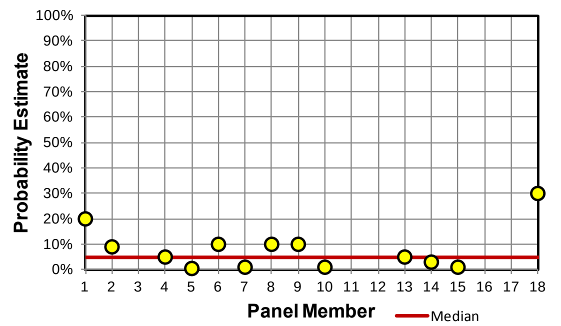
River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100% Height	0.005	0.10	0.05	0.08	0.08	0.30
75% Height	0.001	0.01	0.01	0.03	0.03	0.10
50% Height	0.001	0.01	0.01	0.01	0.004	0.01



50% Height



75% Height



100% Height

Consequences

Consequences were estimated for breach of the West Levee embankment. The methods have been previously described. The results are summarized in the following table.

Loading	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
100% Height	36	56	245	369	814	1,275	313
75% Height	63	115	220	371	1,153	2,009	303
50% Height	4	8	34	68	71	140	53

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below.

Location	Annualized Failure Probability	Annualized Life Loss
West Levee	8.5E-09	3.4E-06

PFM #9/10 – Internal Erosion around a Conduit

During the risk assessment, a sewage outfall tunnel was being constructed beneath the levee. This failure mode was considered during the PFMA session and determined to be a low risk. However, the team assumed these conduits were built as they were planned using the methods as they were approved. Before completion of the outfall tunnel, on 28 January 2012, a large area of the tunnel collapsed and stoped up to the ground surface near the river side of the levee. Flowable fill was placed into the sinkhole to stabilize the area. 350 to 450 cubic yards of flowable fill were placed in the sinkhole. A remedial grouting operation was immediately begun to fill the void and the collapsed area of the tunnel with low mobility grout.

Piezometers near the collapsed area with tips in the basal sand show regional groundwater dropping significantly at the time of the collapse and rebounding over several days. These instruments also show an identical response approximately one month after the initial collapse with no surface expression of distress. This could indicate a collapsed area, likely between the land-side toe of the levee and the initial collapsed area.

Currently, the land side of the collapsed tunnel area is full of water. The City of Dallas is considering options to safely replace the water, re-establish the tunnel, construct the outfall structure, and possible remediate any damage caused during the collapses.

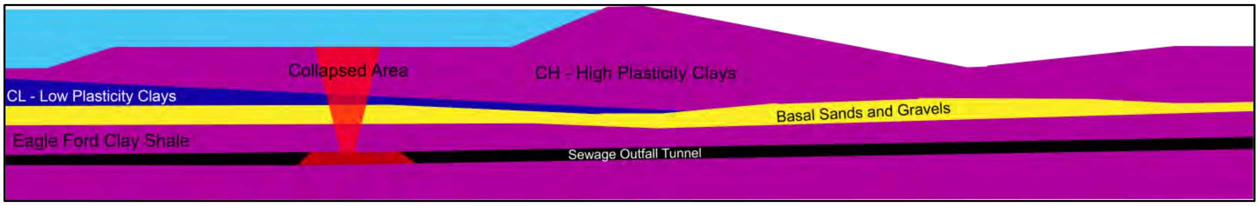


Figure 16 - Levee Section at the collapsed area

Figure 17 - Photo of the Initial Collapse

Figure 18 - Photo of the Collapse Progressing

Figure 19 - Photo of the Sinkhole

Figure 20 - Photo of the Remedial Grouting

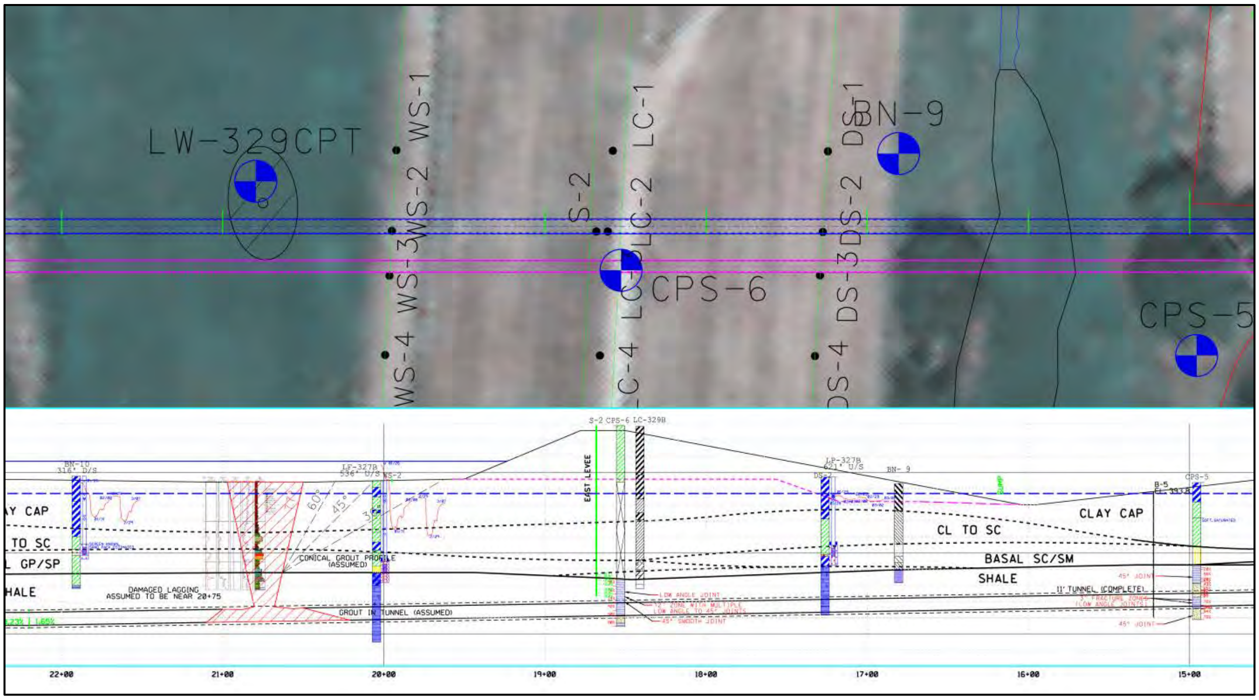


Figure 21 - Cross-section showing instrumentation and nearby borings.

The risk assessment team held a teleconference to discuss the impacts of the existing tunnel situation. The team decided to qualitatively assess the risks posed by the existing system. The team felt that there is a significant chance that a void exists below the levee itself. If this void were to lead to a collapse during a flood event, it could cause a 10-15 foot collapse of the crest of the embankment. If the flood was significant – even for storms with less than 100-year recurrence – this could cause the embankment to overtop and breach. The team used the original matrix from the PFMA, which is shown below.

Failure Likelihood	Consequences of Failure				
	Level 0	Level 1	Level 2	Level 3	Level 4
Very High					
High					
Moderate			FM #3	FM #2	
Low		FM #4	FM #1, FM #3, FM #6, FM #9,	FM #6, FM #7, FM #8, FM #10	
Very Low		FM #4	FM #5	FM #5	

The team believes that the risks posed by the tunnel are at least an order of magnitude higher than any other failure mode evaluated by the team. There is a significant amount of uncertainty with this situation.

PFM #13a – Global Slope Instability of the East Levee

Slides contained within the slopes of the levee embankments have occurred in some reaches of the levee embankments, typically where the embankment is constructed of high Plasticity and high Liquid Limit CH clays with relatively steep slopes (approximately 1 vertical on 3 horizontal). The City of Dallas has developed methods to repair these slides. In addition, some reaches have been repaired and the slopes flattened such that slides are no longer a major issue. However, two reaches remain prone to such slides. The concern over continued slides in these areas led to the development of the event tree shown below. In this scenario the following events would need to occur in order for a breach of the levee to result.

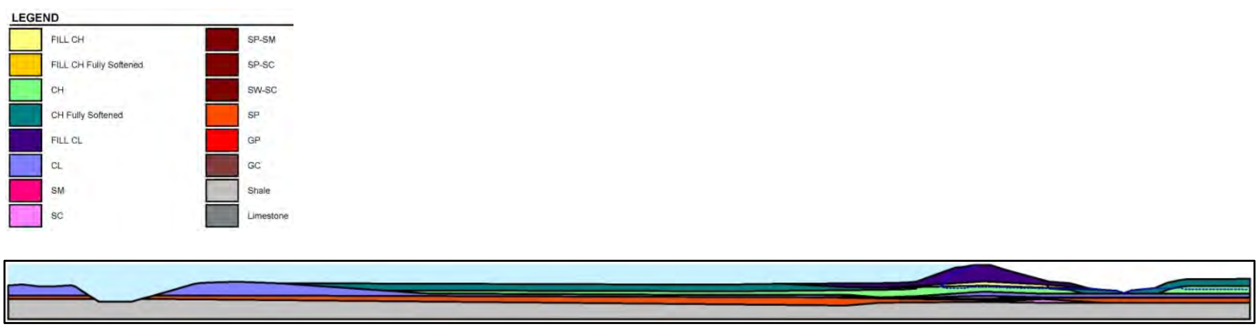


Figure 22 - East Levee Section including river

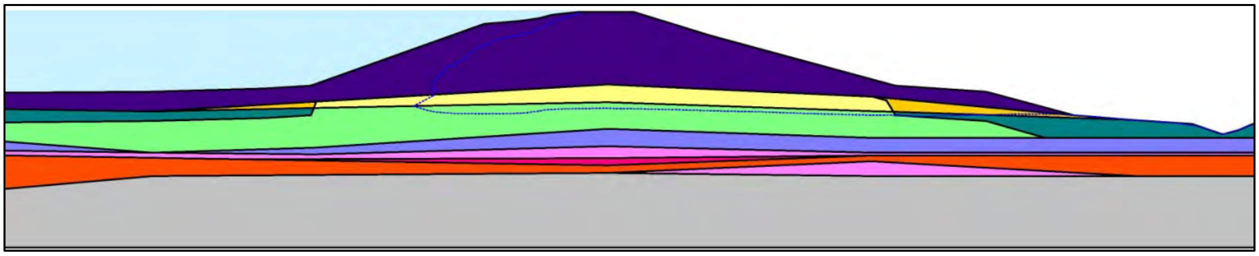


Figure 23 - East Levee Section

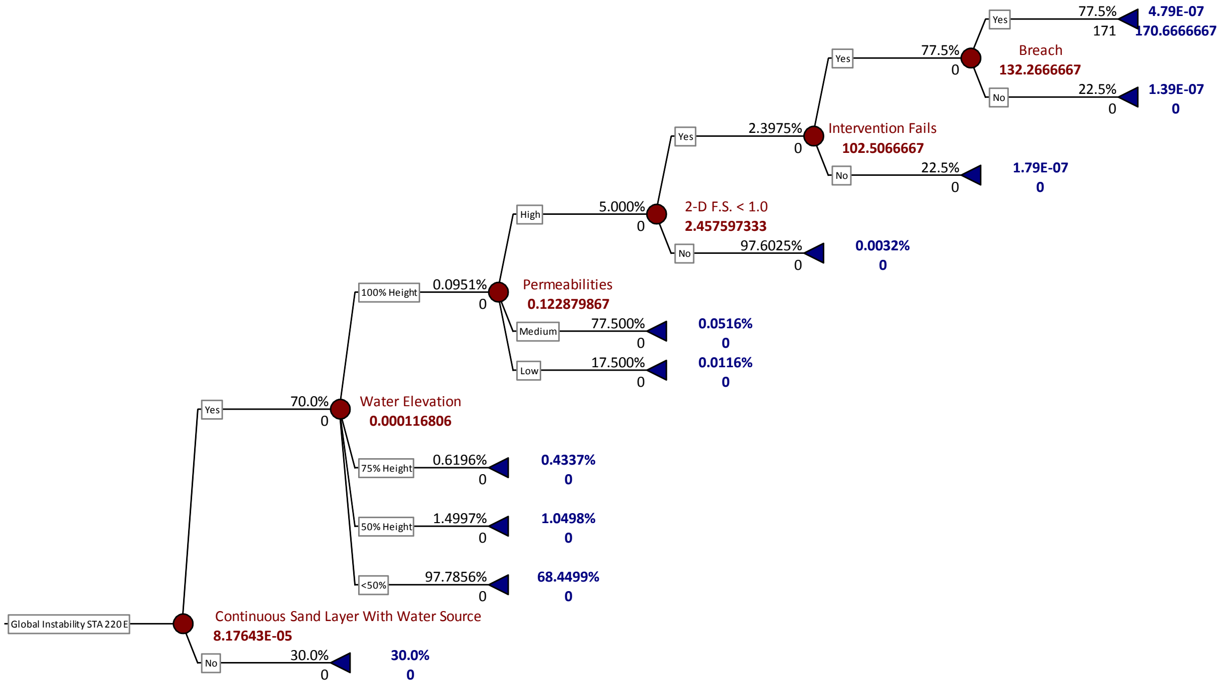
- There is a continuous sand layer connected to the river that allows the levee embankment and/or foundation to saturate
- The materials on-site are assumed to have low, medium, or high permeability
- The two-dimensional factor of safety drops below 1.0 (the driving forces exceed the resisting forces) for a failure surface that impacts most of the crest of the levee
- Intervention may be attempted, perhaps with sandbags or other materials, but it is unsuccessful
- The slide brings the crest of the levee embankment below the river stage and the embankment overtops and erodes leading to breach.

Initiating Event

The team discussed at what level the river stage would need to be before this potential failure mode would be of concern. It was concluded that the river would need to be very close to the levee crest before there would be significant concern for a breach by this mechanism. When the water surface is 75% of the levee height or lower, sliding surfaces that allow the levee to overtop are significantly more stable. This would lead to a more stable condition overall.

Event Tree

The event tree for this potential failure mode is shown below. Estimates for each branch of the event tree are discussed in subsequent sections.



Continuous Sand Layer with Water Source

The team examined this event tree branch and determined that the condition was identical to those examined for PFM #8 in the same location and elected to use the same probability estimates.

Material Permeabilities

Using the abundant data for the site, the team examined the available information and categorized the material properties in this section with respect to their layer permeabilities. The team had earlier selected the following correlations with respect to “low”, “medium”, and “high” permeabilities:

- Low = 10% of the permeabilities were at this value or less permeable from the available test results
- Medium = the median value of all recorded permeability tests in a single layer
- High = 10% of the permeabilities were at this value or more permeable from the available test results

Those permeability estimates were then used in conjunction with the stage hydrograph to determine the seepage front and pore pressures in a transient seepage analysis. That information was then used for the accompanying stability analyses. To simplify the assessment, the models assumed low, medium, or high permeabilities for all the layers rather than varying permeabilities for single layers or attempting any more complex permutations. The team estimated probabilities for low, medium, and high probabilities.

The high plasticity clay, and to some extent the low plasticity clay, whether it is in the levee embankment or in the foundation, is the key parameter for seepage and stability modeling.

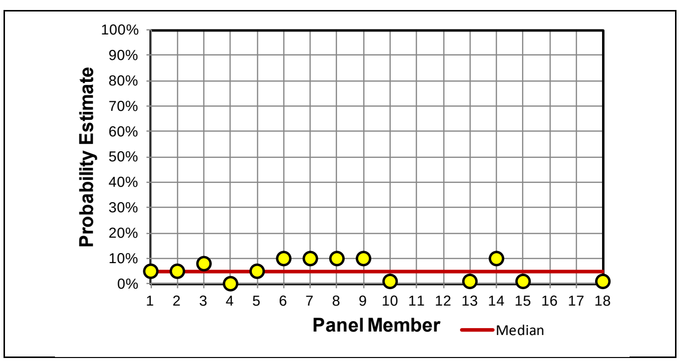
There is an abundant amount of permeability information for both the embankment and foundation.

Material	Low k (cm/sec)	Best Estimate k (cm/sec)	High k (cm/sec)	Basis for Estimate
Basal Sands	6.5E-05	1.3E-03	2.4E-03	HNTB SP
Point Bar Sands	6.5E-05	1.3E-03	2.4E-03	HNTB SP
High Plasticity Clay	1.0E-08	1.0E-07	1.0E-05	EM, HNTB Data
Desiccated Clay	1.0E-06	1.0E-05	1.0E-04	EM, HNTB Falling Head Data
Clean Basal Gravel	1.0E-03	1.0E-02	6.0E-02	HNTB GW, GP
Dirty Basal Gravel	4.0E-04	1.0E-03	3.0E-03	HNTB GW-GC, GP-GC
Lean Clay	2.4E-09	5.0E-07	5.3E-05	HNTB CL
Clayey Sand	1.0E-08	1.0E-06	3.0E-04	HNTB SC

High Permeability

The team assumed a desiccated zone in the upper portion of the embankment and the foundation. It is perhaps 5- to 10-feet deep, and both were modeled during the seepage and stability analysis. Although there is an opportunity for desiccated clay to exist in the upper portion of the levee embankment and the foundation clays, the team does not believe that the 90th percentile values are representative of the true permeability. The assumption that all the layers have high permeability is conservative, and some of the high estimates for the clays have permeabilities that resemble values that would normally be found in sands.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.001	0.10	0.05	0.055	0.04	0.10



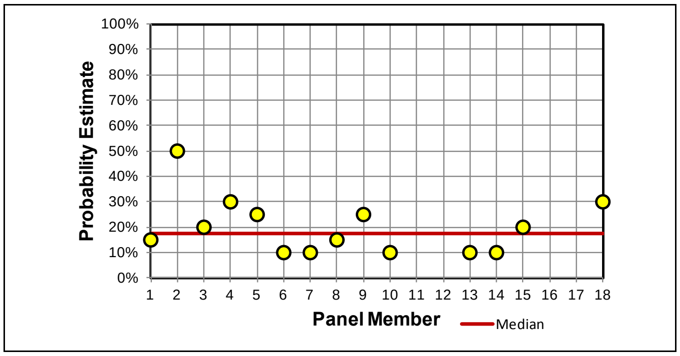
High Permeability

Low Permeability

The team felt similarly about the likelihood of all the layers having a low permeability, although the team felt that a low permeability layer could control the overall permeability and that compacted material would likely have lower permeability values than natural material.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.10	0.10	0.175	0.20	0.11	0.50

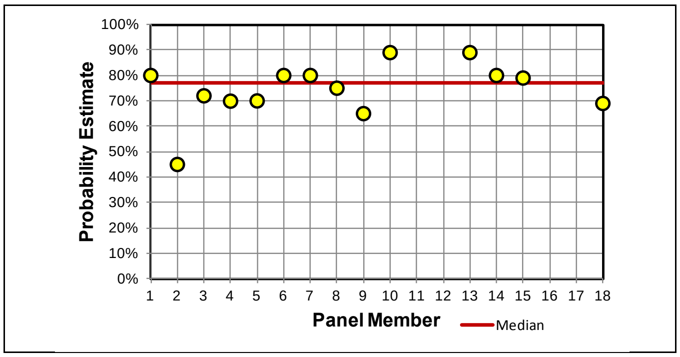


Low Permeability

Medium Permeability

The team calculated the medium permeability layer as 1 – (low permeability + high permeability). The result is the table below:

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.45	0.80	0.77	0.74	0.11	0.89



Medium Permeability

Two-Dimensional Factor of Safety

The team used the permeability estimates and the stage hydrographs to evaluate the actual phreatic surfaces and pore pressures in the system for each of the 7 cross sections. They also ran numerous sensitivity studies to ensure a critical analysis factor was not missed. The results of those seepage and stability analyses can be found in Appendix B and Appendix C. Using low, medium, and high permeability values and varying the strength values in SLOPE/W using the probabilistic stability analyses, the team was able to eliminate all sections except for one on the East Levee assuming all the layers had high permeabilities and all sections where the river elevation was less than 100% of the height. For all other stability analyses, the probability of failure was zero and the factors of safety were high.

The team evaluated the factors that would indicate the probabilistic analysis was a good representation of the actual likelihood that the driving forces would overcome resisting forces for a significant portion of the levee. The team felt that the linear strength envelopes were reasonable given for the normal stresses that are critical for stability. The monte carlo simulation assumed a uniform distribution of strengths between the minimum and maximum. This is likely conservative. The team elected not to include the presence of a tension crack in the stability analyses, which is slightly unconservative. Sensitivity was evaluated using several sections the factor of safety dropped from 1.77 to 1.66 when a tension crack was assumed. Also, the best estimate shear strength was slightly skewed towards the lower bound, but when modeled as a uniform distribution the mean value is likely higher than the test data. The full list of stability analyses completed and the underlying permeability and strength assumptions are shown in the three tables below.

Table 7 - 1990 Hydrograph, River Elevation at Crest of Levee

	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**1.89**	**1.66**	**2.86**	**2.35**	**1.20**	**2.37**	**1.86**	**2.50**
Low k, Prob Str	Mean	**2.19**	**1.84**	**2.87**	**2.06**	**2.71**	**2.71**	**2.04**	**2.38**
	P(f)	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min	1.65	1.43	1.89	1.69	2.00	2.03	1.47	1.77
Best k, Prob Str	Mean	**2.18**	**1.77**	**2.63**	**2.30**	**1.38**	**2.38**	**1.7**	**2.38**
	P(f)	0.00%	0.00%	0.00%	0.00%	13.00%	0.00%	0.00%	0.00%
	Min	1.58	1.31	1.84	1.65	0.95	1.98	1.21	1.77
High k, Prob Str	Mean	**1.91**	**1.24**	**2.35**	**1.88**	**1.89**	**2.24**	**1.56**	**2.15**
	P(f)	0.00%	1.67%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min	1.48	0.82	1.58	1.53	1.41	1.87	1.12	1.61

Table 8 - 1990 Hydrograph, River Elevation at 75% Height of Levee

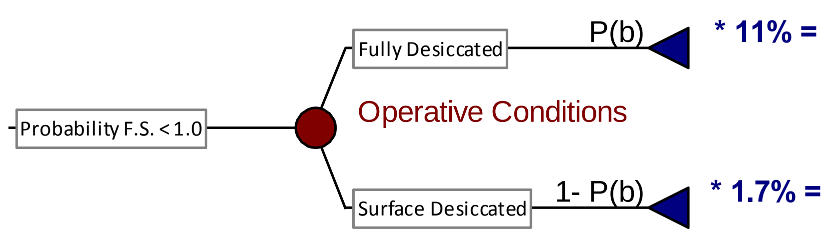
	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**2.22**	**1.84**		**2.35**	**1.46**	**2.43**		**2.50**
Low k, Prob Str	Mean		**2.05**		**2.09**	**2.71**	**2.71**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.71		1.72	2.00	2.03		1.77
Best k, Prob Str	Mean		**1.89**		**2.30**	**1.60**	**2.42**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.56		1.66	1.15	2.02		1.77
High k, Prob Str	Mean		**1.56**		**1.95**	**2.05**			**2.31**
	P(f)		0.00%		0.00%	0.00%			0.00%
	Min		1.15		1.60	1.54			1.77

Table 9 - 1990 Hydrograph, River Elevation at 50% Height of Levee

	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**2.11**	**1.95**		**2.36**	**1.76**	**2.49**		**2.50**
Low k, Prob Str	Mean P(f) Min		**2.08** 0.00% 1.71		**2.30** 0.00% 1.66	**2.71** 0.00% 2.00	**2.71** 0.00% 2.03		**2.38** 0.00% 1.77
Best k, Prob Str	Mean P(f) Min		**2.01** 0.00% 1.56		**2.30** 0.00% 1.6568	**1.86** 0.00% 1.38	**2.5** 0.00% 2.06		**2.38** 0.00% 1.77
High k, Prob Str	Mean P(f) Min		**1.82** 0.00% 1.15		**2.01** 0.00% 1.65	**2.09** 0.00% 1.70			**2.38** 0.00% 1.77

The event tree for global slope instability includes a node for the probability of factor of safety less than 1.0. The team decided that a factor of safety less than 1.0 as calculated by normal two-dimensional slope stability analyses would be a reasonable representation for a failure condition. The software (SLOPE/W) used for the analyses has the capability to calculate the probability of factor of safety less than 1.0 using a Monte-Carlo approach if distributions are input for shear strength. These analyses and input distributions are described in Appendix C.

Probabilistic slope stability analyses were performed for two basic cases. The first included fully desiccated higher permeability clay and fully softened strength estimates in the entire embankment CH foundation material (the embankment in the critical locations is CL material and not thought to be subject to full desiccation). The second included only a surficial layer with these properties. In the first case, the probability of factor of safety less than 1.0 was calculated as 0.11. In the second case, it was calculated as 0.017. The team decided to accept these as reasonable estimates. However, estimates were then required for the probability of each basic case being the operative controlling condition. To facilitate these estimates, a simple event tree was set up, as shown below.

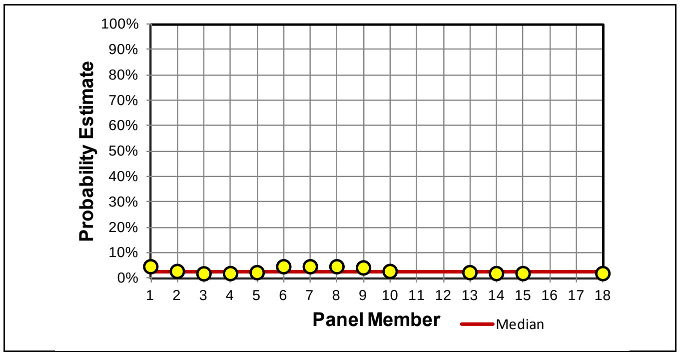


Total P (FS<1.0) = Σ Above

There is only one variable in the mini-tree, the probability of fully desiccated and fully softened clay material within the embankment foundation. The team was asked to estimate this probability, and then the probability of factor of safety less than 1.0 for use in the main event tree was calculated as follows:

$$P(FS < 1.0) = P(b) * 0.11 + [1-P(b)] * 0.017$$

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100%	0.02	0.045	0.024	0.029	0.012	0.045

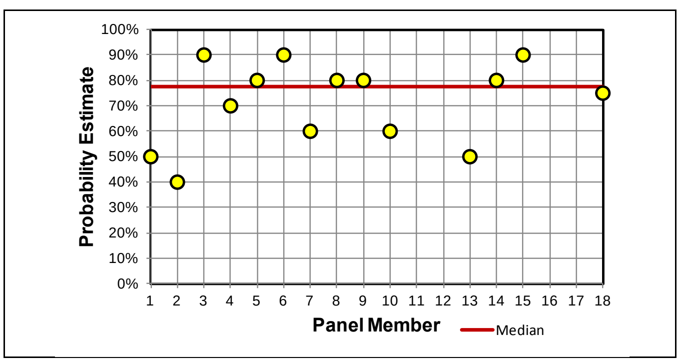


Factor of Safety

Heroic Intervention Fails

The team generally believed that intervention would likely fail if the levee slumped even slightly when flood waters were near the crest of the levee. It would take a major effort to deal with the situation at this point, and the safety of workers might be in jeopardy. Dumping material on the crest would provide additional driving forces and could actually make the situation worse. There is equipment and material on site. There is also frequent monitoring. Perhaps there would be precursors such as cracks or seepage on the land-side face. The sumps could be flooded to increase water load on the land side. The team did not have confidence that intervention would be successful.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100%	0.40	0.80	0.775	0.71	0.16	0.90

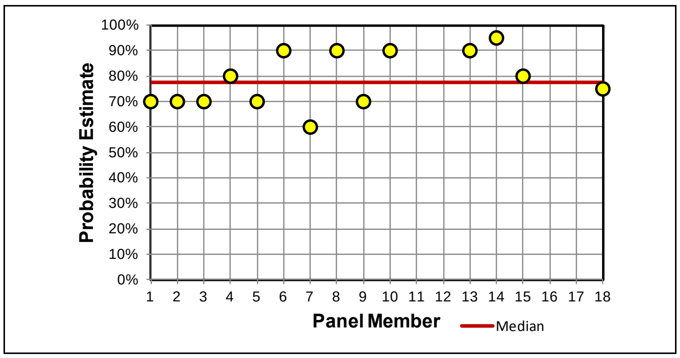


Factor of Safety

Progression to Failure

Once the failure progresses to this point, a large amount of water is likely to be pouring over the crest of the embankment. The disturbed slide mass would likely erode faster than an undisturbed clay embankment. The low notched area would concentrate flow and accelerate the erosion process. However, the slide could have occurred at the peak water level or the descending limb of the hydrograph which might limit the duration of overtopping and limit the breach size. It also takes time to erode a compacted clay embankment. However, the team consensus is that the levee would erode faster than the river would drop.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
100%	0.60	0.70	0.775	0.79	0.10	0.95



Factor of Safety

Consequences

These consequences at the threshold of overtopping were used for this potential failure mode to represent a breach on the East Levee. Refer to the section on Consequences for a description of how these consequences were estimated. The consequences in terms of estimated fatalities are summarized in the following table.

Location	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
East 220+00	19	7	90	46	1,451	172	66

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee STA 220+00	4.8E-07	8.2E-05

PFM #13a – Global Slope Instability of the West Levee

The team also examined the risks posed by global stability on the West Levee. All the factors are the same for this section as for East Levee.

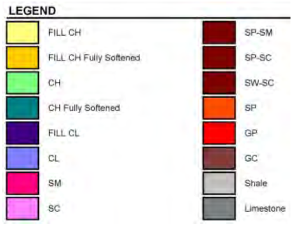


Figure 24 - West Levee Section including river

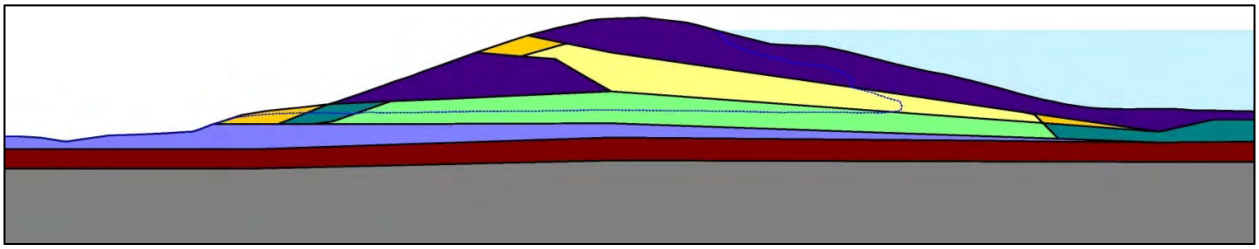
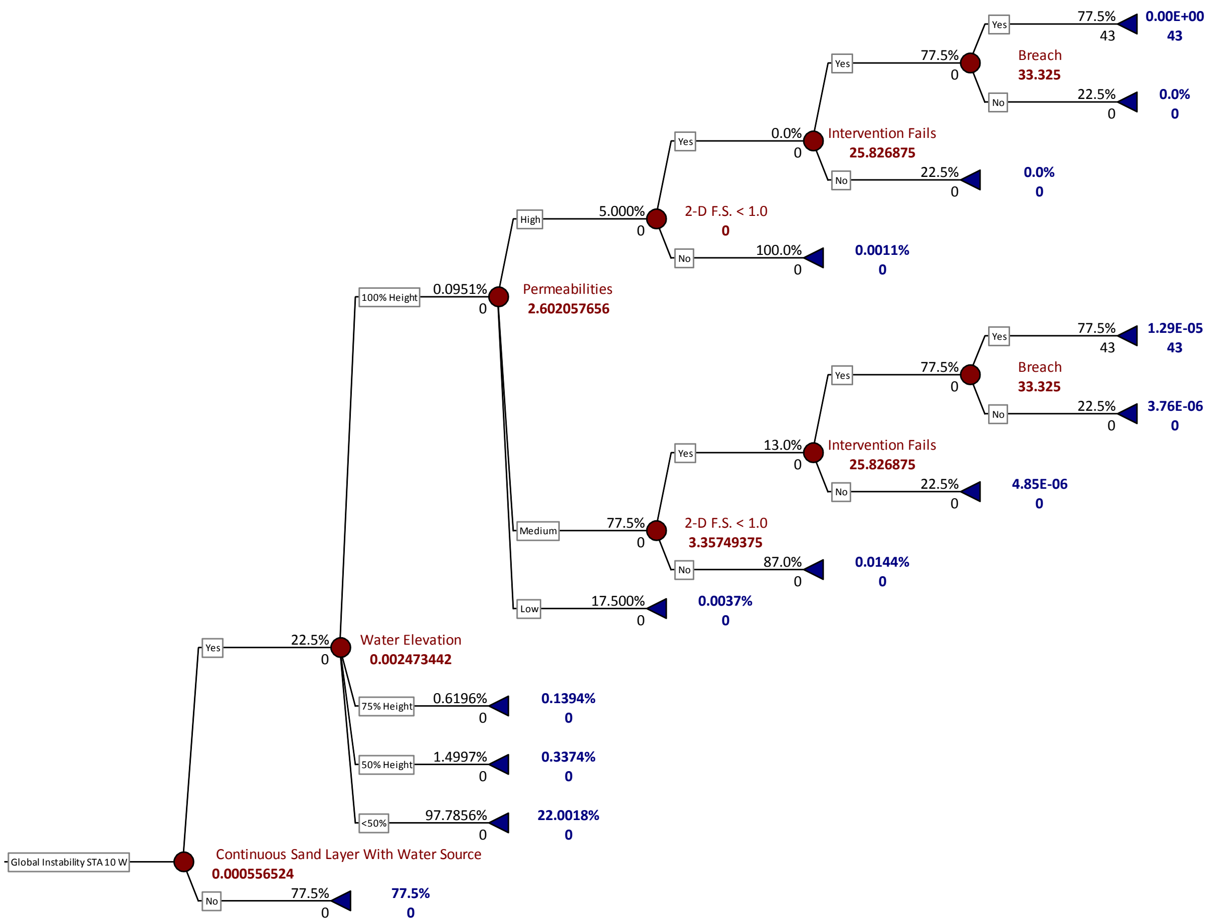


Figure 25 - West Levee Section

Event Tree

The event tree for this potential failure mode is shown below.



The only changes from the previous failure mode was to manually change the 2-D Factor of Safety event tree branch to 13.0%, which is an order of magnitude higher than at the East Levee.

Consequences

These consequences at the threshold of overtopping were used for this potential failure mode to represent a breach on the West Levee. Refer to the section on Consequences for a description of how these consequences were estimated. The consequences in terms of estimated fatalities are summarized in the following table.

Location	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
West 10+00	4	5	35	49	260	175	43

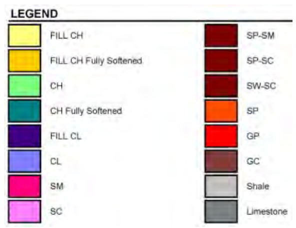
Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below.

Location	Annualized Failure Probability	Annualized Life Loss
West Levee	1.3E-05	5.6E-04

PFM #13a – Global Slope Instability of the East Levee

In an attempt to find the absolute most critical combination of material properties and structural behavior, a section was developed on the East Levee. In this failure mode, an extremely low permeability clay layer exists on the land side acting as a partial aquatard, allowing pressures to build below it during large flood events leading to potential instability of the levee. It was postulated that this situation could occur at any point in the system, but this section was likely the most critical and could be extended to represent the potential for this scenario at any location in the system.



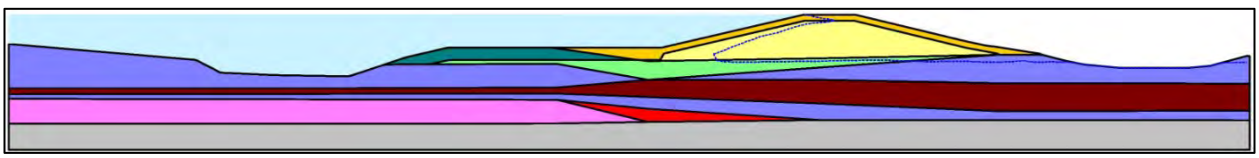


Figure 26 - East Levee Section including river

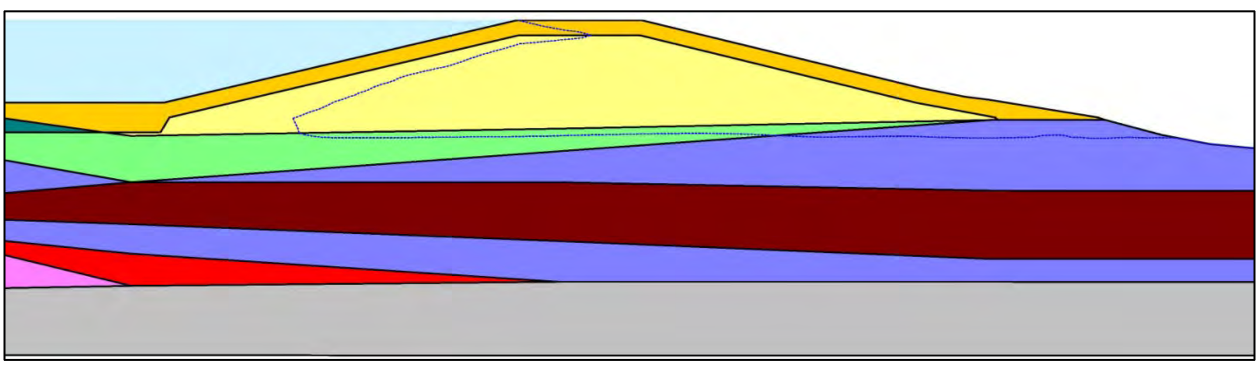


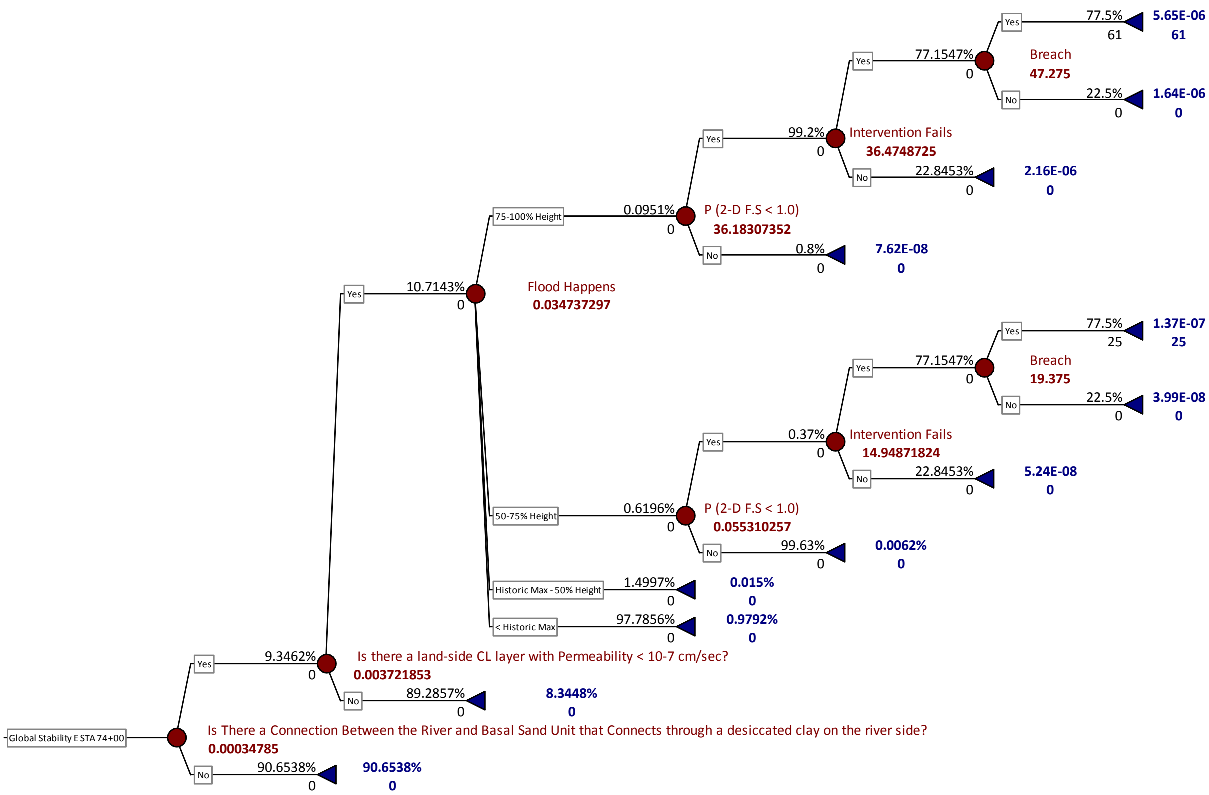
Figure 27 - East Levee Section

Initiating Event

The team discussed at what level the river stage would need to be before this potential failure mode would be of concern. It was concluded that the river would need to be fairly high before there would be significant concern for a breach by this mechanism. When the water surface is 50% of the levee height or lower, sliding surfaces that allow the levee to overtop are significantly more stable. This would lead to a more stable condition overall.

Event Tree

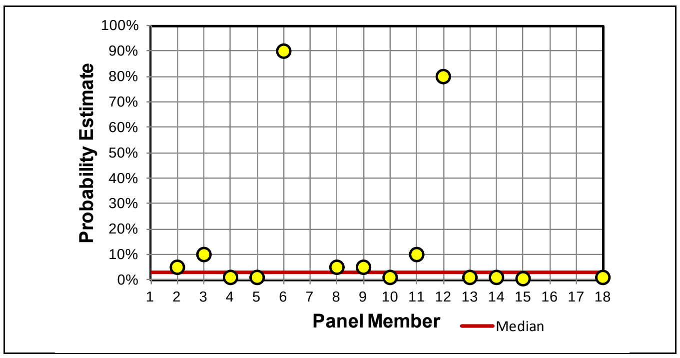
The event tree for this potential failure mode is shown below. Estimates for each branch of the event tree are discussed in subsequent sections.



Fully Desiccated Foundation Connects to Basal Sand Layer

In order for this failure mechanism to occur, a pervious clay layer needs to allow water to move into the basal sand layer and build up pressures below the low plasticity clay layer on the land side of the levee foundation. The desiccation cracks in the high plasticity clay which lines the river side of the foundation would tend to increase the vertical permeability. Measured desiccation cracks in the CH material have been measured as deep as 6 feet, although the layer is approximately 25 feet deep. On the nearby Ft. Worth levee system, clays with moistures below the shrinkage limit have seen cracks as deep as 25'. Even though the layer was modeled as a high plasticity clay, the liquid limit is on the border of a low plasticity clay. Desiccation is more limited in low plasticity clays. Silty layers would tend to limit cracking and vertical permeability. The Cone Penetration Testing (CPT) indicates silty layers within the clay layers, although there were some issues with CPT calibration. The existence of groundwater can also limit the depth of desiccation cracking, as it is a phenomena associated with drying clays. The baseline piezometer levels are at approximately elevation 383, which is approximately 13 feet above the contact between the CH and the layers below which would make it less likely that water pressures would enter the foundation. A nearby piezometer with its tip in the basal sand layer shows a response to river elevation which might indicate contact with the river, although the response is more consistent with a low permeability clay and no direct river connection. Additionally, there are locations where the CH is close to the basal sand layer as shown in LW-33B, BN-10, and LW-331. The model indicates a thick basal sand layer underneath the CL on the land side. The actual geomorphologic conditions and boring logs indicate interbedded layers of sand and clay, so the model is likely conservative compared to the actual conditions. The conclusion was that although there was experience on the Ft. Worth levee system that indicated deep and extensive cracking, the fact that the material is borderline CH/CL, the depth to the sand is so far, and the high water table make it less likely that water pressures will be easily conveyed between the river and the basal sand layer.

Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East STA 74+00	0.005	0.01	0.01	0.10	0.21	0.80



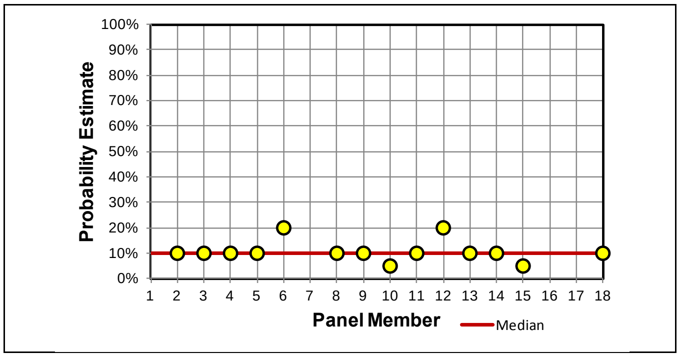
Fully Desiccated Layer (high permeability)

Existence of a Land-Side CL Layer with Very Low Permeability

For the stability of this section to become critical, a low permeability clay layer must exist in the foundation beginning at approximately centerline of the levee and continuing towards the land side that allows pore pressures to build in the basal sand layer below it. Analysis indicates that if the layer is more permeable than 1E-07 cm/sec, it provides enough drainage to limit water pressures to acceptable levels.

There were tests in the data set for the low plasticity that had permeabilities that were this low, although they were likely in compacted embankment material. In order for this to be plausible, the entire layer would need to have a low permeability. Logs indicate some sand lenses in the CL material. The lab testing values might be an order of magnitude less permeable than actual field conditions. The model assumes the permeability on the land side and the river side would be different (high permeability on the river side, low permeability on the land side). The estimates are listed in the table below.

Location	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
East STA 74+00	0.05	0.10	0.10	0.11	0.04	0.20



CL Permeability < 10E-7

Two Dimensional Factor of Safety
The two dimensional factor of safety for the 100% height and 75% height river elevations were calculated using the probabilistic functions within SLOPE/W.

Heroic Intervention Fails
The team examined this event tree branch and determined that the condition was identical to those examined for PFM #13a and elected to use the same probability estimates.

Progression to Failure
The team examined this event tree branch and determined that the condition was identical to those examined for PFM #13a and elected to use the same probability estimates.

Consequences
These consequences at the threshold of overtopping were used for this potential failure mode to represent a breach on the East Levee. Refer to the section on Consequences for a description of how these consequences were estimated. The consequences in terms of estimated fatalities are summarized in the following table.

River Height	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
100% Height	4	2	16	13	226	33	14
75% Height	12	4	26	10	344	37	18

Results
The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee	5.8E-06	3.5E-04

PFM #13b – Progressive Instability of the East Levee System
Slides contained within the slopes of the levee embankments have occurred in some reaches of the levee embankments, typically where the embankment is constructed of high Plasticity and

high Liquid Limit CH clays with relatively steep slopes (approximately 1 vertical on 3 horizontal). The City of Dallas has developed methods to repair these slides. In addition, some reaches have been repaired and the slopes flattened such that slides are no longer a major issue. However, two reaches remain prone to such slides. The concern over continued slides in these areas led to the development of the event tree shown below. In this scenario the following events would need to occur in order for a breach of the levee to result.

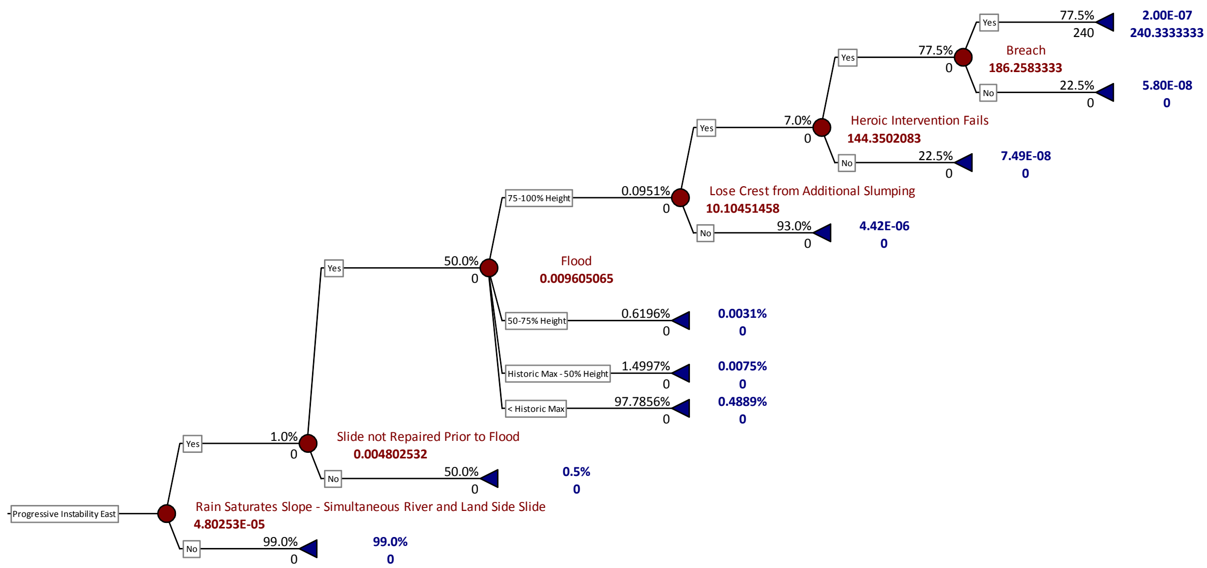
- A soaking rain follows a period of drought resulting in saturation of the portion of the embankment that is desiccated, leading to simultaneous slides on both the river side and protected side of the levee embankment.
- A major flood follows closely behind such that repairs cannot be made to the slides.
- As a result of the flood, additional saturation and slumping occurs in the area previously damaged by upstream and downstream slides.
- Intervention may be attempted, perhaps with sandbags or other materials, as additional slumping occurs, but it is unsuccessful.
- The slumping eventually brings the crest of the levee embankment below the river stage and the embankment overtops and erodes leading to breach.

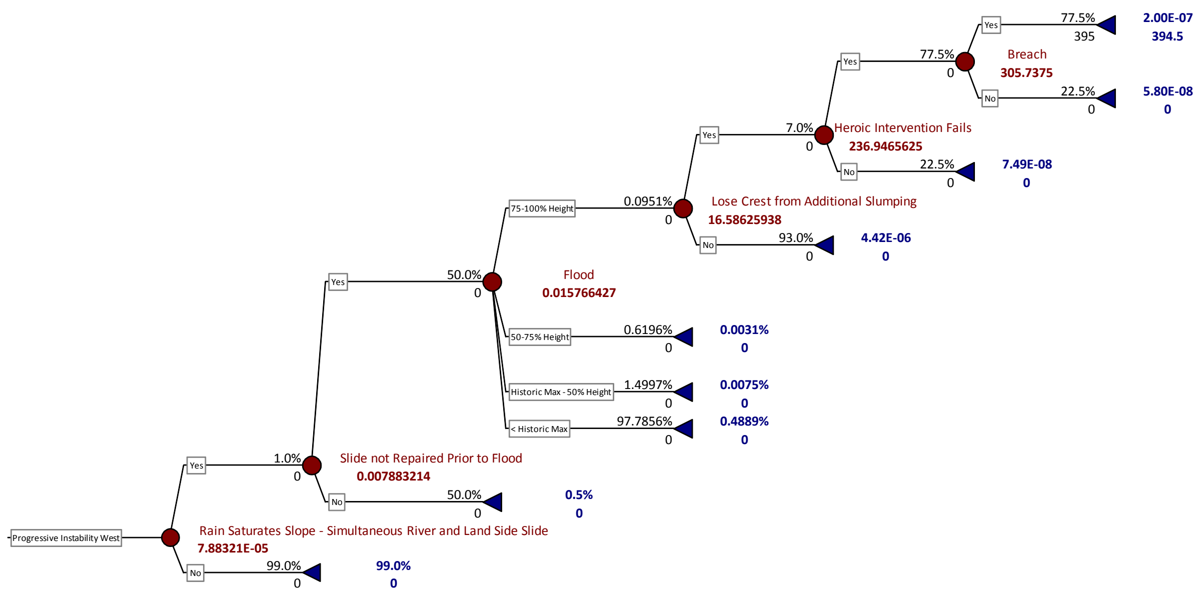
Initiating Event

The team discussed at what level the river stage would need to be before this potential failure mode would be of concern. It was concluded that the river would need to be very close to the levee crest before there would be significant concern for a breach by this mechanism. The basis for this conclusion was the fact that slumping of this nature would put material near the toe of the levee and would remove material from near the crest. This would lead to a more stable condition overall. In addition, slides of this type have typically left a large embankment remnant near the crest. Although an over-steepened section near the crest creates a potential failure condition, only the flood load range from 75% to 100% of the levee height was considered capable of causing a levee breach by this mechanism.

Event Tree

The event tree for this potential failure mode is shown below. Since there was little difference between the East and West Levees for conditions that might lead to this type of failure, the same event tree was used to evaluate both sides. Estimates for each branch of the event tree are discussed in subsequent sections.





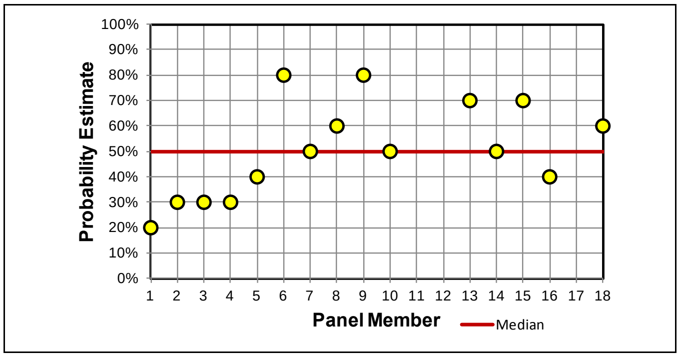
Rain Saturates Slopes – Simultaneous Slides on River and Protected Sides

This node was estimated using statistics of previous slides. In approximately 50 years of record keeping, there have been about 300 documented slope slides, typically during a wet period following a period of drought. However, in only 4 cases were slides located on both the river side and protected side in close proximity. The slides on the landside were closer to the toe of the embankment. Based on this information, the team judged there was approximately a 1% chance of this scenario occurring at any given time. Since the slide statistics drove the estimate, individual estimates were not made for this node.

Slides not Repaired Prior to Major Flood

Although it typically takes a month or two to repair slides when they occur, the actual repair time is only a few days. Thus, if a large storm was forecast, the repairs could be accelerated. Local thunderstorms can occur quickly, and may result in limited warning time and ability to repair the slides in time. It may be possible to perform temporary repairs such as covering the area with tarps anchored into shallow ditches to prevent the infiltration of water. From this discussion, it can be seen that there is significant uncertainty related to this node, and the individual team estimates, summarized in the following table, reflect this.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.20	0.30	0.50	0.51	0.18	0.80

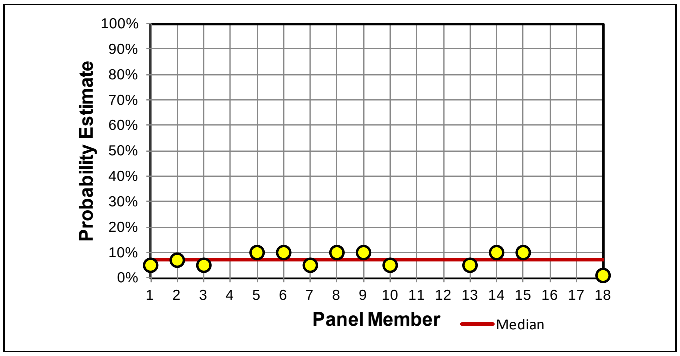


Slides not repaired

Crest Effectively Drops Below River Stage due to Progressive Slumping

The team generally thought it would be unlikely that the crest would be lost to progressive slumping. Although a steeper scarp would be exposed on both sides of the crest and subjected to additional rain, there was no compelling evidence to suggest the scarps would not be stable for the relatively short duration of flooding to this level. Progressive slumping of slide scarps has not been observed historically. In addition, the crest is typically wide (~ 16 feet, although it is somewhat narrower in some locations) and it would take time for progressive slumping to continue across the crest. A summary of the individual estimates is provided in the following table.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.01	0.10	0.07	0.07	0.03	0.10

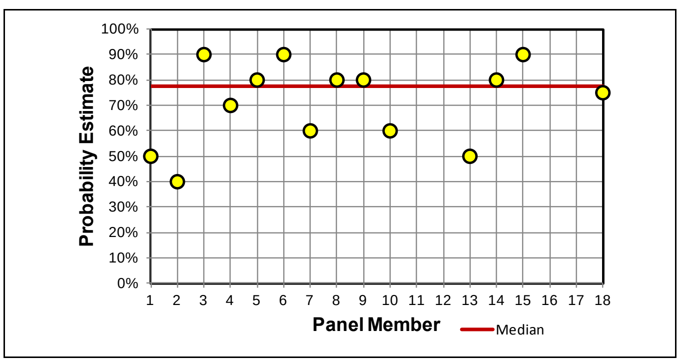


Crest Drops

Heroic Intervention Fails

The team generally believed that intervention would likely fail if the situation got to the point of progressive slumping where the crest elevation dropped below the river stage. The reasoning and estimates basically followed those of PFM 13a, Global Instability at the 100% stage height. It would take a major effort to deal with the situation at this point, and the safety of workers might be in jeopardy. Dumping material on the crest would provide additional driving forces and could actually make the situation worse.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.40	0.80	0.78	0.71	0.16	0.90



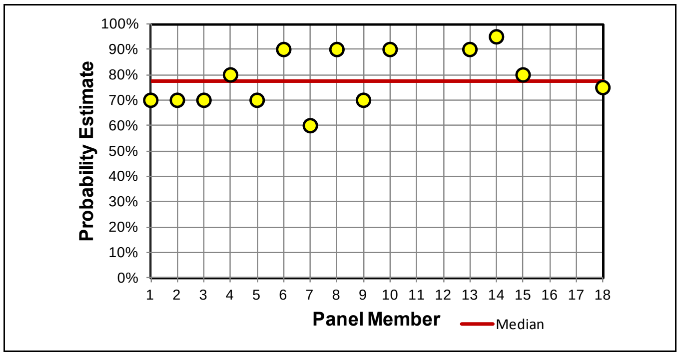
Heroic Intervention Fails

Progression to Breach

Similar to the previous node, the reasoning and estimates for this node essentially followed that for PFM 13A, Global Instability. The disrupted crest would likely concentrate the flow through an area of disturbed soil. Although the clay embankment material should be erosion resistant, most team members thought erosion would occur more quickly than the river would drop. Therefore, the estimates, summarized in the following table, indicate the belief that breach would be likely once the failure mode progressed to this point.

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum

River Stage	Minimum	Mode	Median	Mean	Standard Deviation	Maximum
All	0.60	0.70	0.78	0.79	0.10	0.95



Progression to Breach

Consequences

Consequences were not actually estimated for Reach 5 on the East Levee. They were compared to a nearby location where consequences were estimated. These values at the threshold of overtopping were used for this potential failure mode to represent the East Levee. Consequences were estimated on the West Levee. These consequences at the threshold of overtopping were used for this potential failure mode to represent a breach on the West Levee. Refer to the section on Consequences for a description of how these consequences were estimated. The consequences in terms of estimated fatalities are summarized in the following table.

Location	Best Case Day	Best Case Night	Most Likely Day	Most Likely Night	Worst Case Day	Worst Case Night	Expected Value
East 410+00	42	17	141	81	1,891	235	242
West 355+00	36	56	245	369	814	1,275	394

Results

The expected values for Annualized Failure Probability and Annualized Life Loss, using median values from the team estimates and expected value consequences are summarized below.

Location	Annualized Failure Probability	Annualized Life Loss
East Levee	2.0E-07	4.8E-05
West Levee	2.0E-07	7.9E-05

Uncertainty and Sensitivity Analyses

Uncertainty

Uncertainty was modeled using distributions contained in @Risk. For the hydrologic loading, the 5th, 50th, and 95th curves were input using a lognormal distribution. Several distributions were tried, but none matched the median, mean, and range of uncertainty as well as the lognormal distribution. Cumulative distributions were examined, particularly for the flood loading, but distribution type did not have a significant effect on the results.

For each event tree branch, the team estimates were used to quantify the uncertainty. For nearly all distributions a normal distribution was selected, truncated at the team's minimum and maximum estimate, and the standard deviation was changed to make the mean and median as close as possible. Figure 30 shows an example of what that distribution looks like for a single event tree branch. All the distributions are not included in this report.

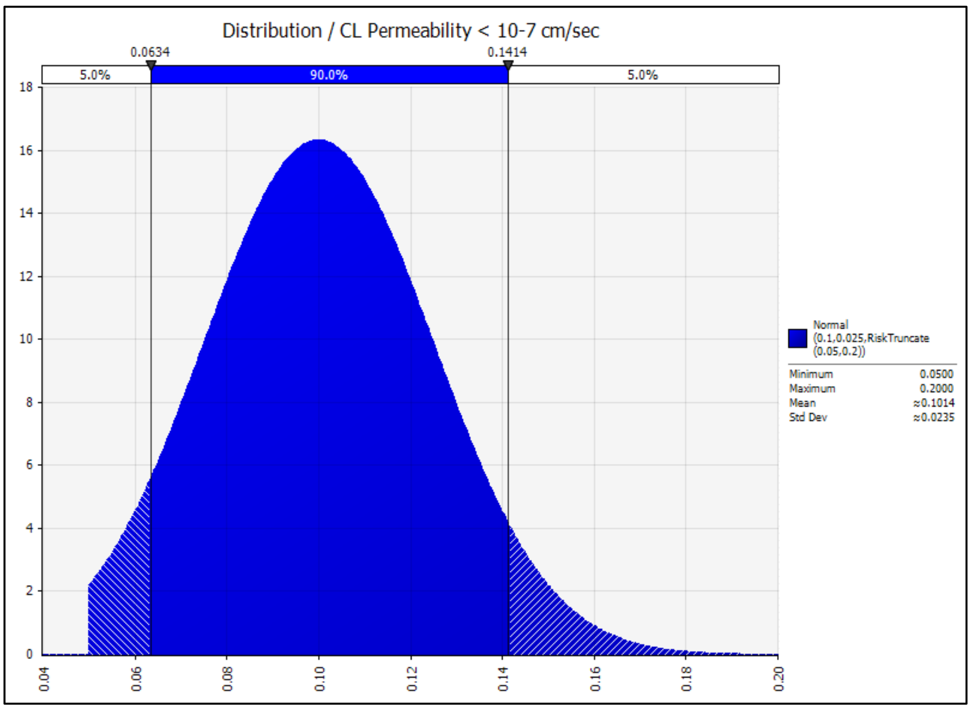


Figure 28 - Example Truncated Normal Distribution

These distributions were used and included in a Monte Carlo simulation with 10,000 runs per failure mode. Other distributions were examined (Triangular, Lognormal, Gaussian, and Uniform), but they had very small effects on the uncertainty. Appendix F – Uncertainty Results contains the Monte Carlo simulation results.

Sensitivity

Seepage and Stability

The team evaluated the sensitivity of various factors related to seepage and stability. Those cases are shown in the two tables below.

Table 10 - East Alignment Sensitivity

Variation	STA 74+00	STA 311+00	STA 220+00	STA 410+00	Reasoning
5-ft Desiccated Layer on Levee Surface	E74-mid w varying desiccation depths				Models the effect (primarily on seepage) of having desiccation layers of varying depths.
10-ft Desiccated Layer on Levee Surface	E74-mid w varying desiccation depths				Models the effect (primarily on seepage) of having desiccation layers of varying depths.
15-ft Desiccated Layer on Levee Surface	E74-mid w varying desiccation depths				Models the effect (primarily on seepage) of having desiccation layers of varying depths.
Tension Crack - 5 ft			E220_High; Mid; Low; East 220+00 12-13-11		Models the effect of desiccation cracks on stability
Tension Crack - 10 ft			E220_High; Mid; Low; East 220+00 12-13-12		Models the effect of desiccation cracks on stability
Failure Surface Extent			E220_High		Restricted to remove infinite slope failures from the results and to limit the failure surface to slides that would take out the levee crest.
Levee section assumed as all CH			E220_High, Mid, Low Desc		Models the potential of a levee section that is susceptible to desiccation.
Desiccated Layer Under Levee	E74-desiccated fdn		E220 Best Desiccated; E220_Low Desc; E220_Mid Desc		Models the effect of a foundation layer that desiccated before construction of the original levee section.

Variation	STA 74+00	STA 311+00	STA 220+00	STA 410+00	Reasoning
Blocked seepage entrance			E220_High Desc		Modeled the effect of removing the direct seepage entrance at the river. Slightly increased stability but seepage still enters via the long, relatively thin clay in the free-field between the river and the levee toe. Sand layer was still allowed to charge up.
Modeling SP as CH			E220_High Desc		Modeled the effect of removing the primary seepage path from under the levee. Served to reduce the pore pressures under the protected side of the levee where the sand layer should be and as a result the failure surface did not advance as deep and stability was increased.
Materials applied to end boundaries	E74-SWF perm-NO END BOUNDARIES				Established how GeoStudio reacts to no infinite boundaries (can cause a back-up of pore pressures that can substantially reduce stability.
Modeled Shale as fully softened clay		East 311+00 RMC			Models the foundation layer as a CH clay layer to determine the effect this has on stability.
Block Failure surface		East 311+00 RMC	East 220+00 12-13-11		Compares another way of determining failure surfaces beyond the entry/exit method. When optimized, it results in a similar failure surface.
Time steps outside of peak		East 311+00 RMC	East 220+00 12-13-11		Investigates the effect that post-peak seepage investigation time steps have on stability. Peak time step was critical in all situations.
Undrained Stability Analysis		East 311+00 RMC	East 220+00 12-13-11		Additional stability analyses done to determine if undrained strengths were critical compared to drained strength parameters should there be saturated conditions present. Drained strengths are critical over the best estimate of undrained strengths. The transient seepage analysis model indicates that modeling stability with drained strengths is the proper method of assessing stability.

Variation	STA 74+00	STA 311+00	STA 220+00	STA 410+00	Reasoning
Ru to model PWP					This was done to simulate the effect of a saturated levee slope surface due to rainfall. This was a way of reproducing the fully softened strength surface slides seen regularly around the levee system.
FSS infinite slope failure					This was done to simulate the effect of a saturated levee slope surface due to rainfall. This was a way of reproducing the fully softened strength surface slides seen regularly around the levee system.
Protected side failure surface					Investigates the effect of the fully softened strength surface slides on both sides of the levee, eventually leading to loss of the crest due to intersecting failure surfaces. Was not found to be a viable failure mode.
All materials best k, CL low k	E74-Fully Desiccated, Low CL, Varying Floods				Investigates the condition in which seepage can enter at the flood side of the levee and then penetrate below the levee section and have nowhere to exit on the protected side, creating a condition that charges up pore pressure beneath the levee. 4 orders of magnitude difference in the clay layers from flood side to protected side. Stability significantly reduced.
All materials best k, CL varying k	E74-Fully Desiccated, Varying CL k				Investigates the effect of varying flood heights on a levee section where pore pressures can get trapped and charge up reducing stability.

Table 11 - West Alignment Sensitivity

Variation	STA 335+00	STA 250+00	STA 188+00	Reasoning
5-ft Desiccated Layer on Levee Surface		W250_Mid-levee all CH		Models the effect (primarily on seepage) of having desiccation layers of varying depths.

Variation	STA 335+00	STA 250+00	STA 188+00	Reasoning
10-ft Desiccated Layer on Levee Surface	W335+00 Mid Expanded Desiccation			Models the effect (primarily on seepage) of having desiccation layers of varying depths.
15-ft Desiccated Layer on Levee Surface	W335+00 Mid Expanded Desiccation			Models the effect (primarily on seepage) of having desiccation layers of varying depths.
Tension Crack - 5 ft	W33 Best-Shallow Failure; W335+00 Mid, Desc Fdn			Models the effect of desiccation cracks on stability
Tension Crack - 10 ft				Models the effect of desiccation cracks on stability
Failure Surface Extent				Restricted to remove infinite slope failures from the results and to limit the failure surface to slides that would take out the levee crest.
Levee section assumed as all CH		W250_Mid-levee all CH		Models the potential of a levee section that is susceptible to desiccation.
Desiccated Layer Under Levee	W335+00 Mid, Desc Fdn			Models the effect of a foundation layer that desiccated before construction of the original levee section.
Blocked seepage entrance				Modeled the effect of removing the direct seepage entrance at the river. Slightly increased stability but seepage still enters via the long, relatively thin clay in the free-field between the river and the levee toe. Sand layer was still allowed to charge up.
Modeling SP as CH				Modeled the effect of removing the primary seepage path from under the levee. Served to reduce the pore pressures under the protected side of the levee where the sand layer should be and as a result the failure surface did not advance as deep and stability was increased.
Materials applied to end boundaries				Established how GeoStudio reacts to no infinite boundaries (can cause a back-up of pore pressures that can substantially reduce stability.
Modeled Shale as fully softened clay				Models the foundation layer as a CH clay layer to determine the effect this has on stability.
Block Failure surface	W335 Best-Shallow Failure	West 250+00 RMC		Compares another way of determining failure surfaces beyond the entry/exit method. When optimized, it results in a similar failure surface.

Variation	STA 335+00	STA 250+00	STA 188+00	Reasoning
Time steps outside of peak		West 250+00 RMC		Investigates the effect that post-peak seepage investigation time steps have on stability. Peak time step was critical in all situations.
Undrained Stability Analysis		West 250+00 RMC		Additional stability analyses done to determine if undrained strengths were critical compared to drained strength parameters should there be saturated conditions present. Drained strengths are critical over the best estimate of undrained strengths. The transient seepage analysis model indicates that modeling stability with drained strengths is the proper method of assessing stability.
Ru to model PWP	W335 Best-Shallow Failure			This was done to simulate the effect of a saturated levee slope surface due to rainfall. This was a way of reproducing the fully softened strength surface slides seen regularly around the levee system.
FSS infinite slope failure	W335+00 Mid; W335 Best-Shallow Failure			This was done to simulate the effect of a saturated levee slope surface due to rainfall. This was a way of reproducing the fully softened strength surface slides seen regularly around the levee system.
Protected side failure surface	W335+00 Mid, Desc Fdn; W335 Best-Shallow Failure			Investigates the effect of the fully softened strength surface slides on both sides of the levee, eventually leading to loss of the crest due to intersecting failure surfaces. Was not found to be a viable failure mode.
All materials best k, CL low k				Investigates the condition in which seepage can enter at the flood side of the levee and then penetrate below the levee section and have nowhere to exit on the protected side, creating a condition that charges up pore pressure beneath the levee. 4 orders of magnitude difference in the clay layers from flood side to protected side. Stability significantly reduced.
All materials best k, CL varying k				Investigates the effect of varying flood heights on a levee section where pore pressures can get trapped and charge up reducing stability.

Individual Risk Estimates

Before the risk assessment began, the team was prepared to use an independent group of risk estimators where were both very experienced in their field and also in subjective probability estimates. In the end, estimates from the entire group of 16 individuals who consistently participated in the group discussions were included because the variation in individual estimates was quite small. Generally, the maximum and minimum estimates are also eliminated, but this was found to have a negligible effect on the results. The overall results of the estimates are shown in Figure 31.

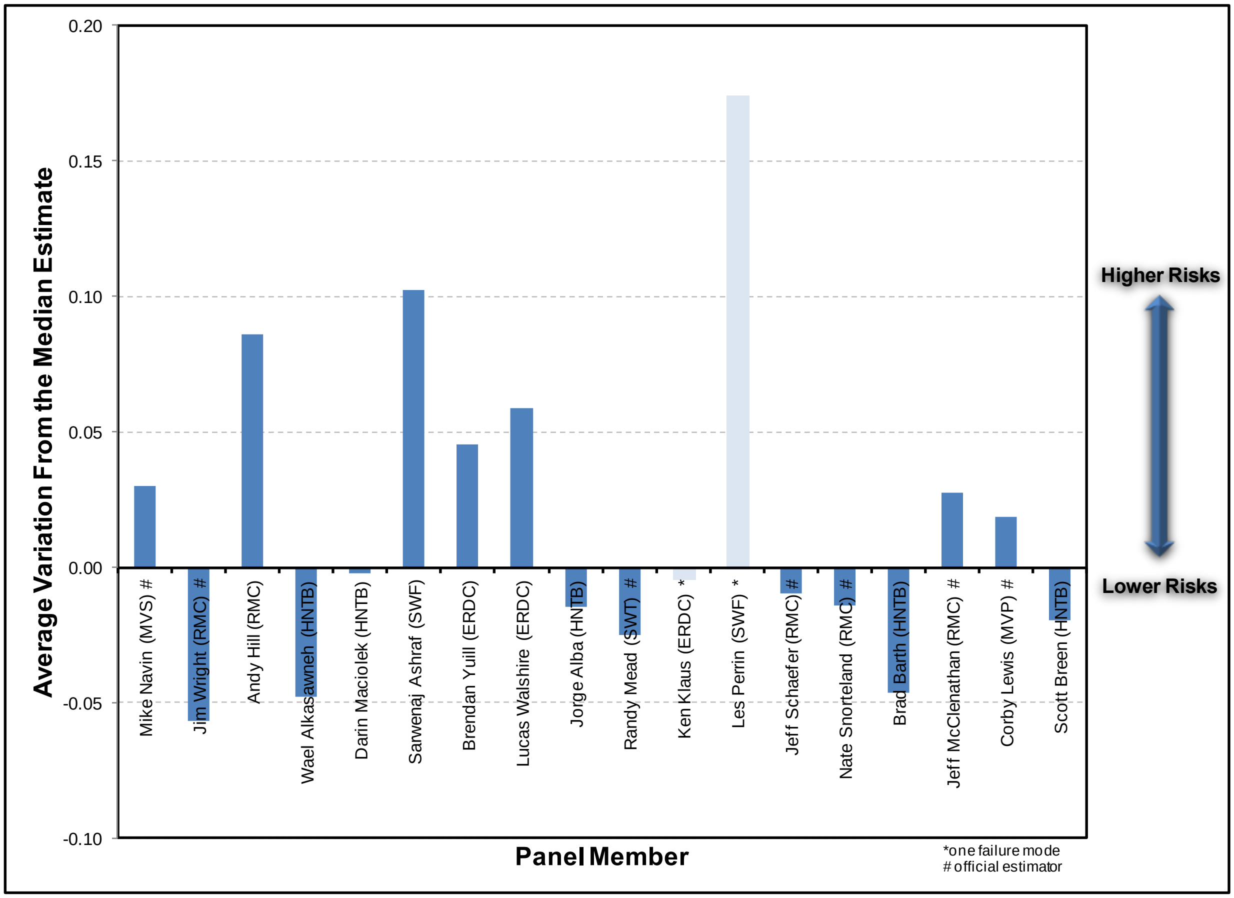


Figure 29 - Summary of Individual Risk Estimates

System Risk – Common Cause and Length Effects

Common cause adjustments were examined, but because the conditional probabilities of failure were so low for all the failure modes, the adjustment was so small that it was not included in the individual event trees.

The Dallas Floodway system of levees consists of several miles of protection. It is not too difficult to recognize that a structure miles long has a higher probability of failure than an identical structure that is only a few hundred feet long. There are more opportunities for flaws or unexpected conditions in a long structure. A completely satisfactory method for dealing with length effects has not yet been developed. The typical approach is to divide a long structure in to reaches of similar geometry and geologic conditions, and then to evaluate each reach. That approach was adopted here. The reaches are shown in the following figure and were broken out based primarily on similar geologic conditions.

The team evaluated the relevance of each "risk-driver" potential failure mode in relation to each reach to make sure nothing was missed. This evaluation is shown in the following table.

Reach	Internal Erosion	Heave	Global Instability	Progressive Instability
1	N/A	East Levee	East Levee	Improvements Made
2	N/A	N/A	N/A	N/A
3	N/A	East Levee	East Levee	N/A
4	East Levee	N/A	N/A	N/A
5	N/A	N/A	N/A	Yes
6	N/A	West Levee	West Levee	Improvements Made
7	West Levee	N/A	N/A	N/A
8	N/A	West Levee	All Monte Carlo Factors of Safety > 1.0	Yes
9	West Levee	N/A	N/A	N/A

As a result of this exercise, it was concluded that the team had identified most of the critical areas. However, no studies had been performed for heave or global instability for Reach 6, where conditions were conducive to their development. Therefore, a section was analyzed at another location following the team activities and those studies are also documented in this report.

Since different populations would be affected by breach of the East Levee in comparison to the West Levee, risks will be accumulated separately for East reaches and West reaches, including not only overtopping erosion risks, but also the risks identified in the above table.

Major Findings and Understandings

Major findings from the Dallas Floodway Base Condition Risk Assessment were captured in three ways. For the first, each team member was asked to provide the factors that stuck in their mind as a result of participating in two weeks of potential failure mode analysis and risk assessment. For the second, a summary was prepared by the facilitators at the end of those sessions for the out-briefing. The third came at the end of compiling the risk assessment report after the risk calculation results were studied in more detail.

Important Information Regarding This Study

This report is a comprehensive evaluation of the risks posed by the existing East and West levee systems. This document should be used as the base condition risk assessment for the system. If new information is discovered or evaluated in the future, these risks should be updated and evaluated to incorporate that information. Flooding poses the primary risks to the east and west levee systems.

Numerical modeling and analyses are engineering artifacts developed to describe processes using science and empirical correlations. These analyses are imperfect in all cases. The East and West Levee systems are complicated and the perceived risks depend on our ability to incorporate information we don't know and the information we do know into an overall perspective of the safety of the system. It would be impractical – if not impossible – to eliminate the possibility of a flaw existing in the current system that we have not yet identified. However, Karl Terzaghi's observational method ("Keen observation is at least as necessary as penetrating analysis") is a good guide for the best approach to handle these potential unknowns. USACE and the City of Dallas have been practicing this method for more than 50 years – when problems are identified, they are addressed expeditiously. Slides have been fixed. Pump stations have been improved or replaced. By that process, many of the most vulnerable areas have been identified and improved.

The ability for the City of Dallas to take steps to intercept and address internal erosion, heave, and land-side stability failure modes early – what USACE considers non-heroic intervention – is important and affects the perception of risk and the risks posed by the levees. The risk assessment team both commends the City for having the capacity and intention to take significant actions when unexpected performance is observed and simultaneously cautions the City to ensure that these potential actions are continually considered and exercised. Having monitoring in critical locations, having personnel available who are educated and capable, and having material and equipment available in critical areas during flood events are essential to the safety of the City. These efforts should be continued and augmented whenever possible.

f-N Plots

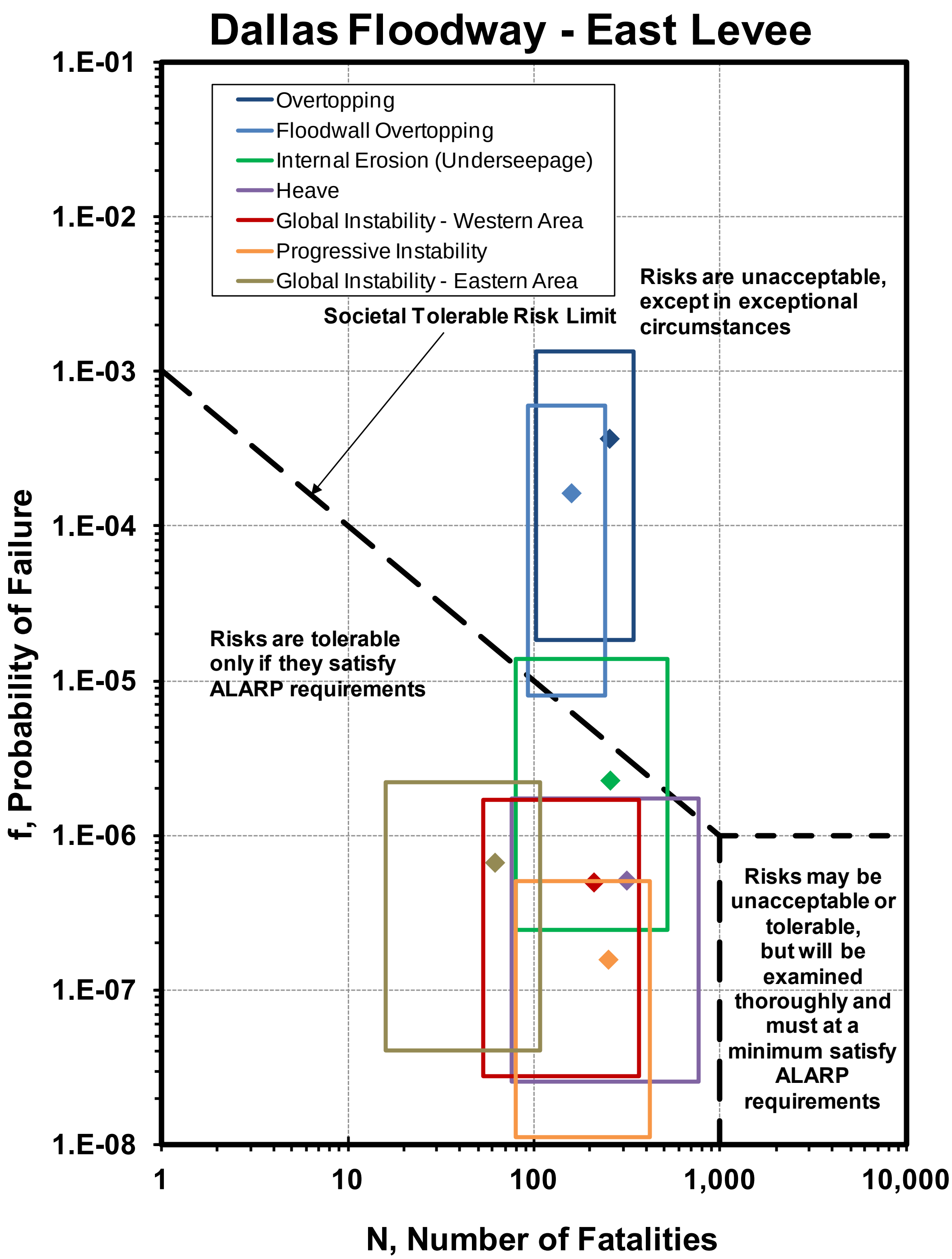


Figure 30 – f-N plot for the East Levee System. Points represent mean risk estimates. Boxes represent uncertainty of those mean estimates and roughly correspond to 5th and 95th limits.

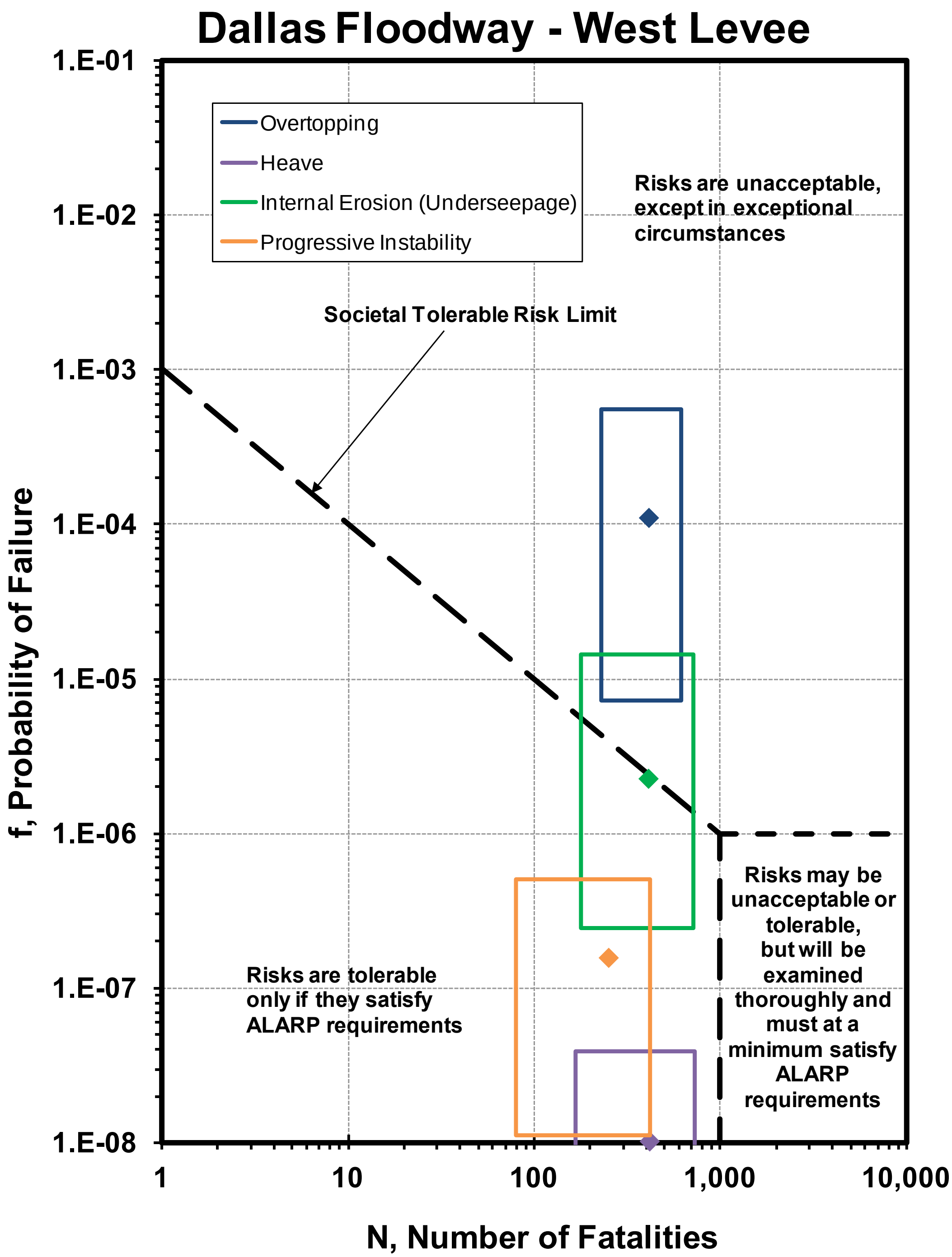


Figure 31 - f-N plot for the West Levee System. Points represent mean risk estimates. Boxes represent uncertainty of those mean estimates and roughly correspond to 5th and 95th limits.

F-N Plots

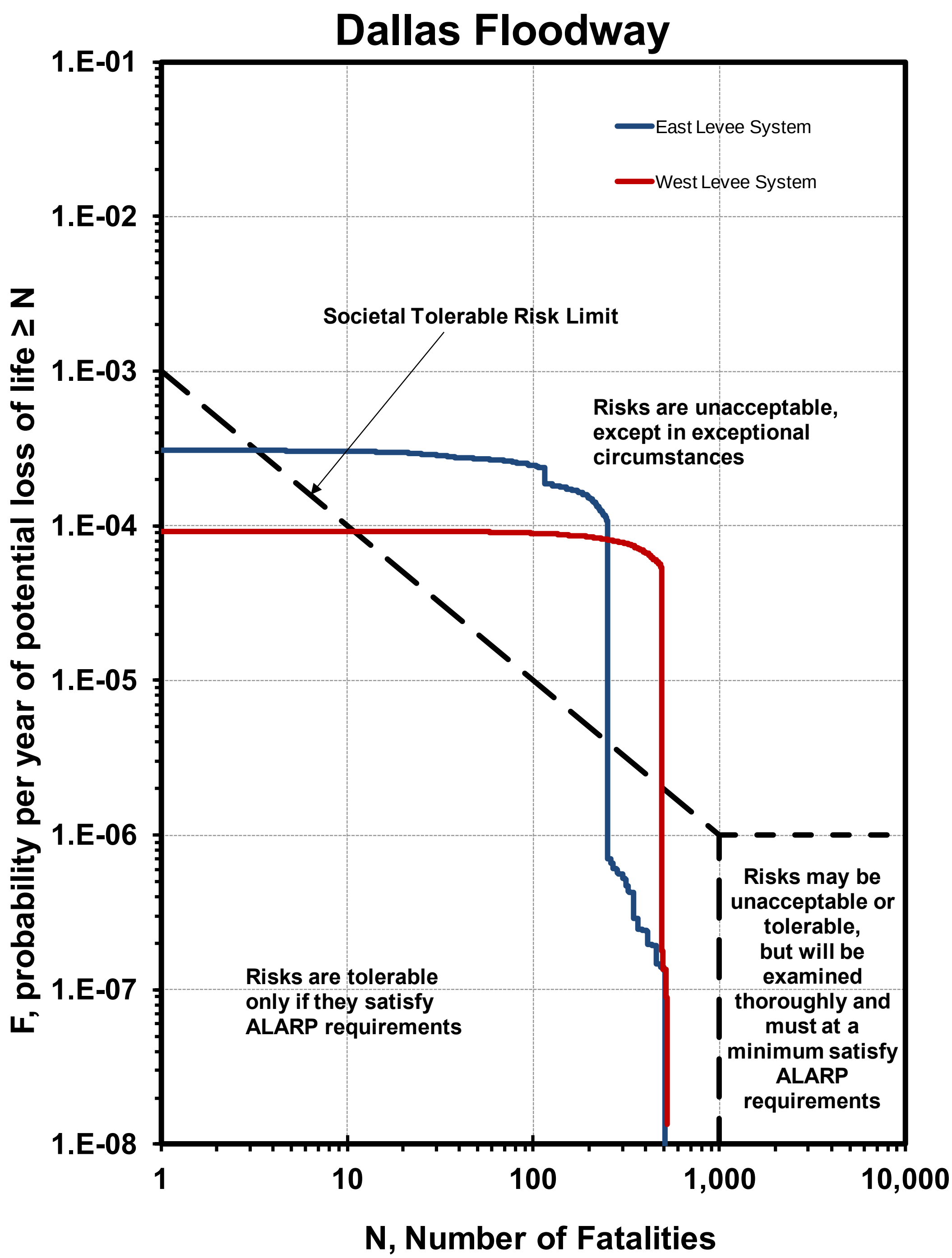


Figure 32 - F-N plot of the East and West Levee Systems. Curves represent total risks posed by each system from all failure modes.

Major Findings from the Risk Assessment

Unacceptable Risks

Risks for overtopping of the East Levee embankment, West Levee embankment, and the floodwall on the East Levee are unacceptable using Societal Tolerable Risk Guidelines for life safety from ER 1110-2-1156.

Tolerable Risks

Risks for all other failure modes would likely be tolerable, although uncertainty related to internal erosion indicates risks could potentially be unacceptable.

Major Findings from Out-briefing

The potential failure mode risks appear to rank out from highest risk to lowest risk in the following order:

- Overtopping erosion of the levee embankments
- Overtopping and undermining erosion of the concrete floodwall
- Backward erosion piping of a sand layer connected to the river and exposed in a landside sump
- Blowout or heave of a clay confining layer in a landside sump followed by backward erosion piping of the underlying sand layer
- Global instability of a levee embankment slope that takes out the crest in a single slip through the embankment and foundation
- Progressive instability of a levee embankment slope due to localized slumping, saturation, and more slumping

With respect to embankment overtopping erosion, the following factors are pertinent:

- If a major flood occurs that brings the river stage to the levee crest, the crest will likely be overtopped in multiple locations.
- Overtopping of the embankments will likely occur first in the center reaches where surveys show the crest to be lowest relative to the anticipated river stages.
- Breach of the embankments due to overtopping is not certain due to the plasticity of the clay materials forming the levees and their foundations, and the limited duration of overtopping anticipated from the critical hydrograph shapes.
- If a breach does develop, it will likely do so slowly, again due to the somewhat erosion resistant soils.
- The risk for overtopping embankment erosion breach is strongly driven by the frequency of the overtopping floods (based on peak flow), even though the soils are somewhat erosion resistant.
- Areas will begin to flood upon overtopping of the embankments, but the depths and areas of inundation will increase significantly following breach.

The following findings were captured relative to flood overtopping and undermining erosion of the concrete floodwall:

- The concrete floodwall represents the lowest point in the line of protection, and thus will overtop first.
- Even though the floodwall will overtop first, it is not as tall of a structure as the levee embankments, and thus the breach inundation and consequences will not be as severe.
- The chance of intervention is better for this potential failure mode than for overtopping breach of the levee embankments due to good access and the limited reach (~ 1,000 feet) that would need to be protected.

Major findings related to the potential for backward erosion piping of a sand layer leading to breach of a levee embankment are as follows:

- The seepage path for such a potential failure mode would extend from the river to a sump on the protected side of the levee. This results in a long seepage path and low gradients such that progression of the failure mode would be unlikely.
- A continuous clean sand layer extending from the river to the sump is unlikely. Although sandy zones and layers are present in the foundations of the levees, they appear to have significant fines in many locations.
- The uniformity coefficient of the sand is such that a low critical exit gradient is unlikely.

With respect to the potential for heave or blowout of a confining clay layer in a sump area leading to backward erosion piping of a sand layer, the following major finding was captured.

- Analysis indicates pore pressures are high enough in the sand layers beneath the clay caps to initiate and produce boils. However, backward erosion is unlikely to progress to the river for reasons cited above.

In terms of global instability, the following major finding was captured:

- The stability of the levee embankments is primarily dependent on developed pore pressures, which are likely to be low due to the transient nature of the flood loading and the generally low permeability of the clay embankments and their foundations.

In terms of progressive instability, the following major findings were identified:

- Progressive instability would likely require simultaneous sliding on the upstream and downstream slopes of an embankment.
- Progressive instability is driven by saturation of the desiccated embankment zones.
- It is unlikely that the entire crest would be breached during the progression of a flood.
- Progressive slumping would need to gradually eat away at the crest, and a very high river stage would be needed to result in breach.

Major Findings from Individual Team Members

- The worst case scenarios (i.e. complete levee saturation and major slope failure) are not as probable as originally thought due to limited duration of flood loading and multiple steps/conditions that would need to occur before breach could result.

- Once a failure mode initiates, it will likely take some time for a breach to develop due to the plasticity in the clays forming the embankments and foundations.
- The saturation of the levee embankments is controlled by the peak of the hydrographs and not by the duration of the hydrograph "benches" or tails, since the water levels represented by the benches and tails are quite low on the embankments when compared to the levels of the peak flows.
- Failure likelihood is not as sensitive to variations in subsurface conditions as originally thought. Even with conservative assumptions on clean sand layer continuity, the transient nature of the flood loading still controlled the modeled behavior.
- Re-evaluation of the hydrology suggests the level of protection (against an overtopping flood) is higher than previously thought.
- The system has not been tested for floods higher than about 1/50 annual exceedance probability.
- If overtopping of the levees occurs, it will likely overtop over significant reaches and at multiple locations.
- Overtopping without breach will not likely fill up the inundated area due to limited volume in the expected hydrographs. However, if the levee breaches the incremental depth and area of inundation will be significant.
- The City of Dallas has been pro-active in managing risks. They have a good surveillance plan and a good Emergency Action Plan. They have established good lines of communication with the public (including neighborhood focus groups) and emergency management officials. This should all result in good warning for any potential issues.
- The population at risk behind the East Levee is largely in commercial areas and there is good potential for evacuation. The population at risk behind the West Levee is largely residential with more special needs citizens. Evacuation could be more difficult in these areas.
- Seepage issues related to sand layers and exposed sumps are mitigated by long seepage paths and limited potential for continuity of clean sand layers.
- Strength parameters did not drive the stability analysis results. The permeability values in relation to transient seepage analysis had more effect.
- Saturation of the embankments from river flow is unlikely. Rainfall saturating desiccated clay could be more problematic.
- The performance of the levee system to date would not suggest any issues related to seepage and piping. No boils or seepage have been observed in the sumps to date.
- Estimating likelihoods for "what-if" scenarios and focusing on the most reasonable scenarios helped put things in better perspective.
- The new hydrologic information was enlightening in terms of how large regional storms really affect the basin and flood-stage durations. The flood risk management dams help to attenuate the peaks such that the local storms that produce high peak flows control the risks.

Recommendations

The Risk Management Center has several recommendations regarding the outcomes of this report.

1. Life safety risks for overtopping of the East and West Levee systems exceed Tolerable Risk Guidelines. Alternatives to reduce these risks should be explored. Water elevation is the primary driver of this risk.
2. The team believes that the way USACE and the City of Dallas have approached managing the system is the most prudent way to proceed in the future, as other steps that would need to be considered to eliminate performance uncertainty would be so expensive that they would outweigh the benefits currently provided by the system.
3. The risk assessment used seepage and stability models that depended on our ability to model the situation adequately. The team believes instrumentation options should be explored to be able to confirm those assumptions in critical areas during flood events.
4. The sewage outfall tunnel situation warrants close attention and the investigations related to that collapse should be incorporated as an addendum to this risk assessment if the findings are significant.

Appendix A – Cross Section Selection

Cross Section Selection

A total of eight cross-sections were selected for seepage and stability analysis from the east and west levee reaches of the Floodway. They were selected to be representative of the most critical conditions on the levee system to the best of our knowledge. Reasonable amounts of uncertainty were factored in to the analysis for parameters that displayed varying results during field and laboratory testing. Any gaps in data were typically bridged with reasonably conservative assumptions. The stationing corresponding to the cross sections selected can be seen below.

East Levee Alignment

- 74+00
- 220+00
- 311+00
- 410+00

West Levee Alignment

- 10+00
- 188+00
- 250+00
- 335+00

The cross sections were selected based on several factors. The major factors include:

- Their proximity to the river thalweg
- A small levee cross-section
- The presence of a continuous sand layer extending from a flood-side entry point and continuing under the levee section to the protected-side.
- Their proximity to a pump station
- The presence of near surface sands
- The presence of seepage entry points closer than the typical river banks (lakes, sand/gravel mining operations, low spots, etc.)

These sections provide reasonable coverage over the areas that have been previously identified as areas that are more critical with regard to seepage and stability. They also provide coverage over all the geotechnical reaches over the Dallas Floodway except for Reaches 5 and 9.

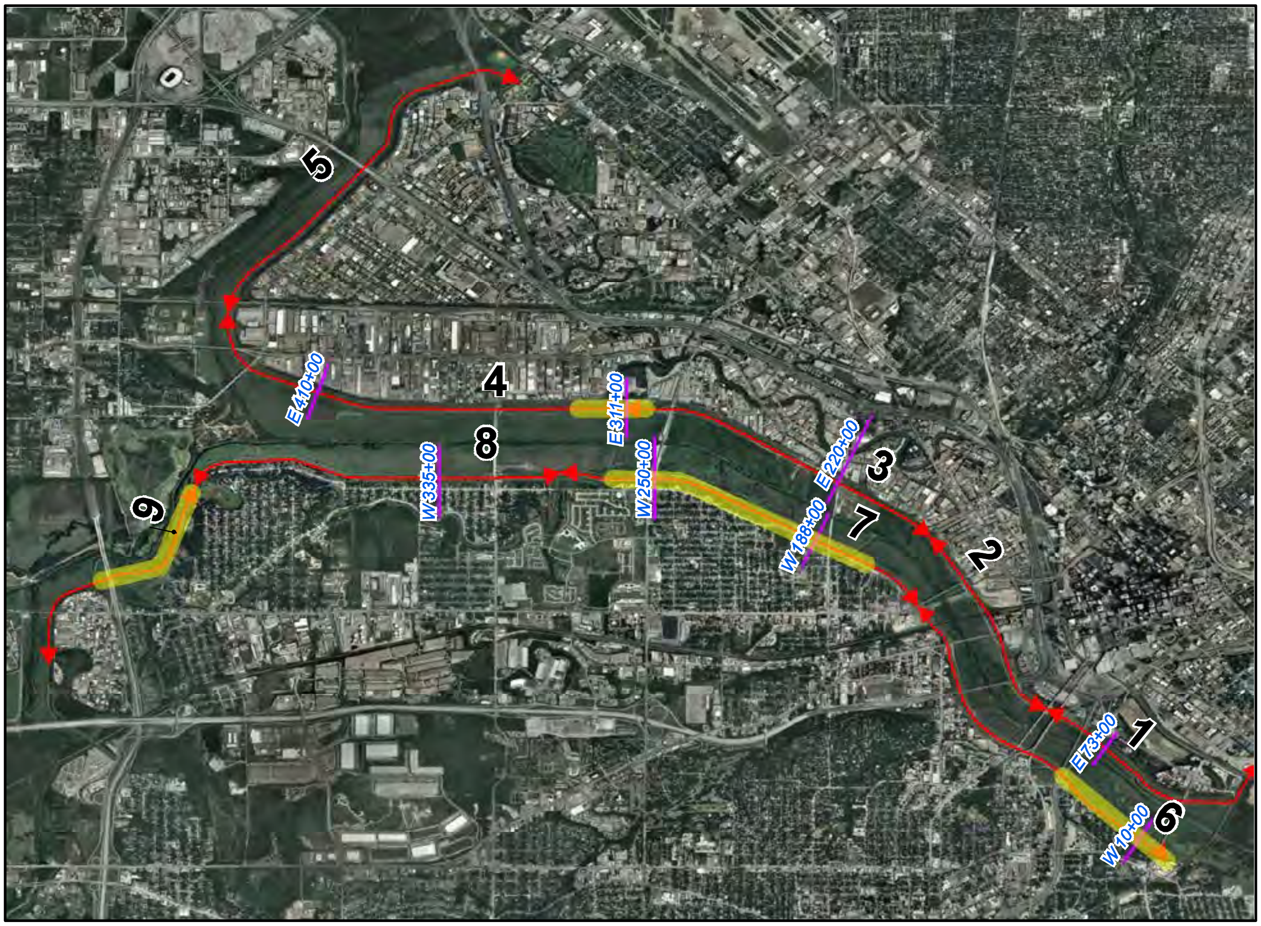


Figure 33 - Plan map of the Dallas Floodway with near surface sands indicated in yellow.

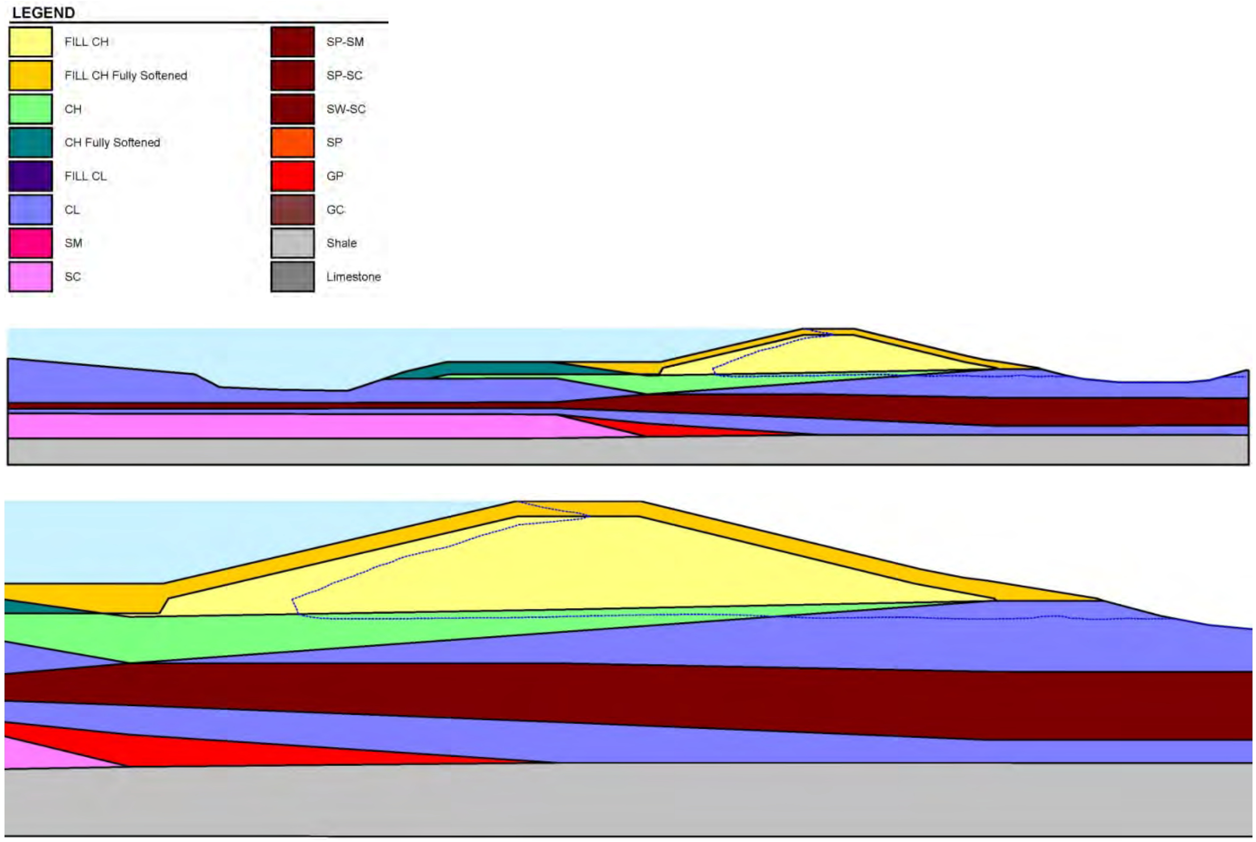
Cross Section Construction

Cross-sections for seepage and stability analysis were constructed in GeoStudio using the HNTB GeoStudio files as a starting point. The flood-side of each section selected was extended to include the Trinity River thalweg in order to accurately model all seepage entry points that can influence the performance of the flood protection (see the figure below showing the section corresponding to Station 220+00). This was done by using the most recent Trinity River surveys to capture the topography of each section between the river thalweg and the flood-side toe of the levee. The soil stratigraphy in the extended area was determined based on boring and CPT data collected in the "free-field" area between the flood-side levee toe and the Trinity River. If no such data was available, the stratigraphy was interpolated based on the nearest subsurface information available, typically from flood-side borings taken near the levee toe. The soil stratigraphy in and under the levee section was at times modified from the HNTB file to reflect additional information that was provided by surrounding borings and CPT's. If surrounding subsurface data indicated the presence of a continuous sand layer in the area, the seepage model was modified to capture the more critical condition.

Initial piezometric surfaces were defined by piezometer readings provided by HNTB. Often the piezometer data would indicate there were two consistent water levels being measured that more or less correspond to summer and winter time levels. If this is the case, the higher of the two water levels were selected as the initial piezometric level as a conservative measure. Water levels measured in bore holes and CPT probe holes were also used.

The paragraphs below specifically discuss each of the analysis sections.

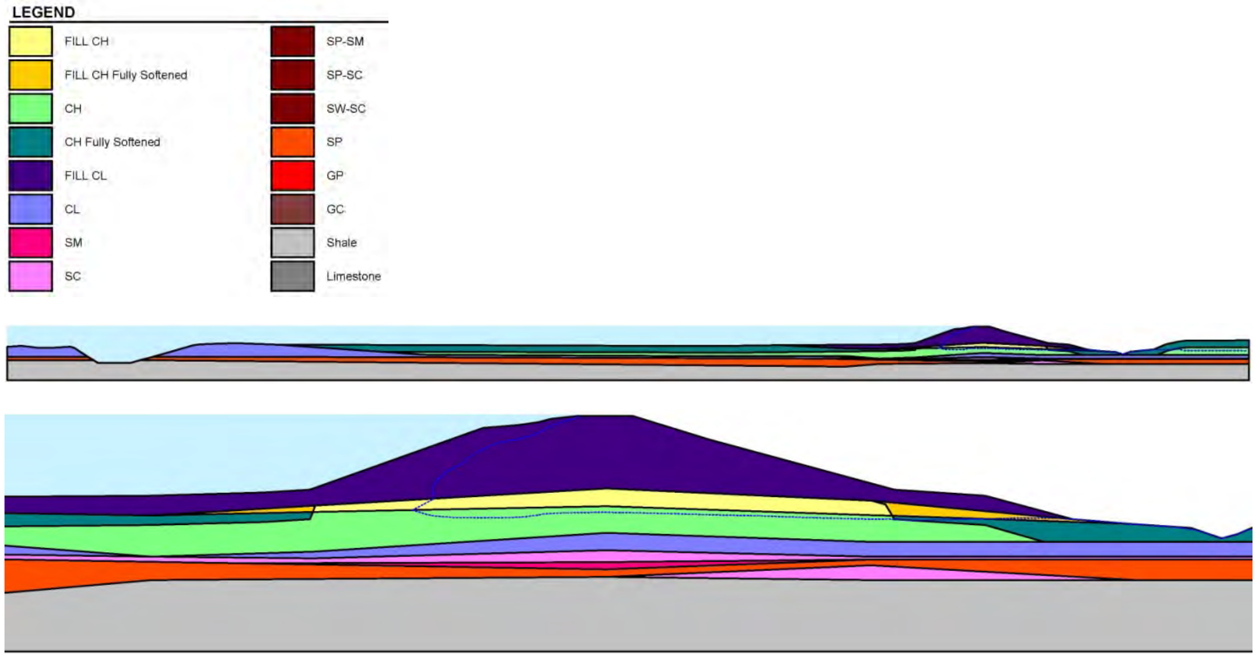
Station 74+00, East Alignment



A section at station 74+00 on the east alignment of the Dallas Floodway was selected for stability and seepage analysis due to its proximity to the river thalweg and the presence of a continuous sand layer under the levee section. There are sections further downstation (southeast) on the east alignment that are as close or closer to the river thalweg, but station 74+00 represents the station closest to the population center that has a close proximity to the river thalweg. In addition, there is an approximately 5-foot thick continuous layer of sand (SP-SM) that runs under the levee from the flood-side to the protected-side that is not present further downstation where the levee centerline gets closer to the river thalweg (see Borings LW-335B, LW-337B, and LW-339AB which show no samples of sand, only CL and CH soils overlying shale and/or limestone). Further upstation (northwest), the thalweg moves away from the levee centerline, providing for a more stable section.

- There appears to be a clay (CL) aquatard at the river thalweg above the SP-SC sand layer that is on the order of 10 ft thick that will prevent a direct seepage connection from occurring through the continuous sand layer under the levee section.
- Water level information was not available from the borings advanced through the levee section, but a piezometer (BN-10) located 260 ft to the riverside of the levee as well as a boring advanced on the south side of the river (N-29) provide some indication of what a starting piezometric head level may be.

Station 220+00, East Alignment



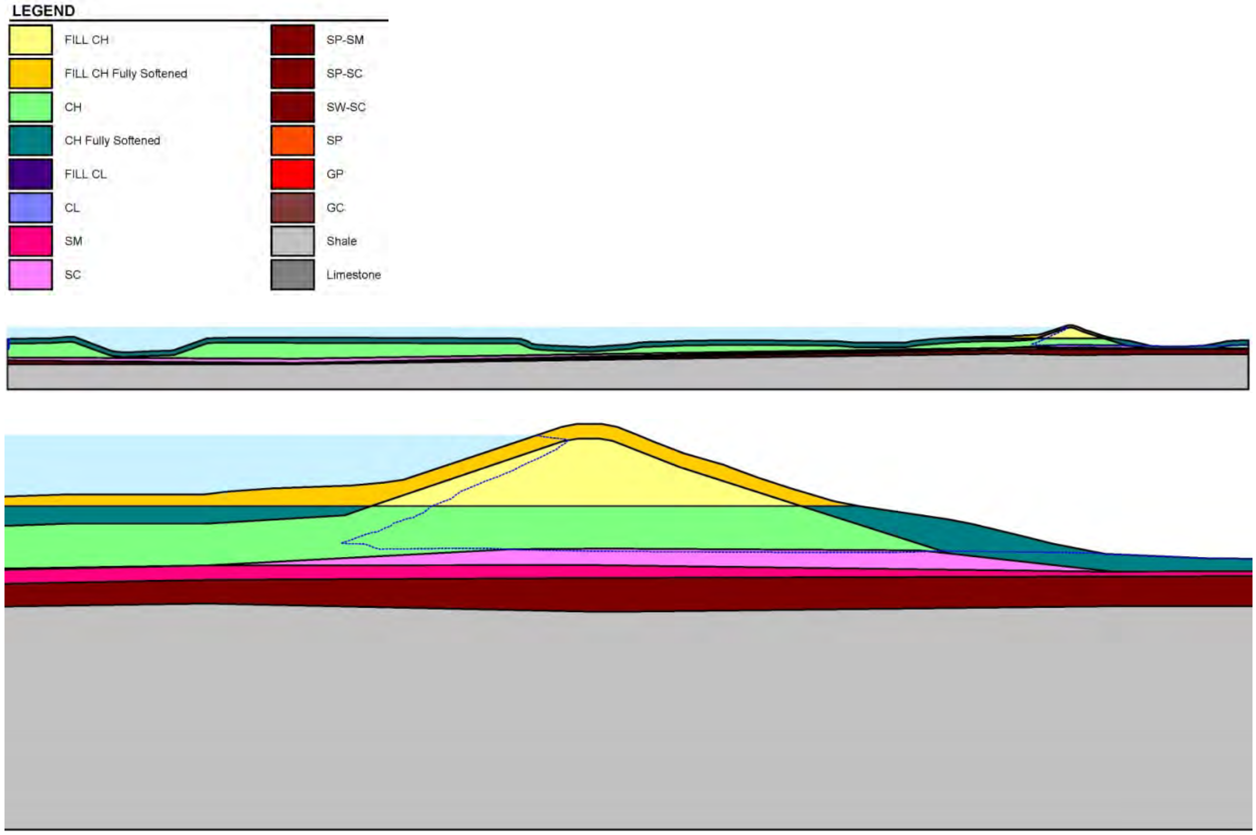
The area of the Dallas Floodway around station 220+00 on the east alignment was selected for investigation due to its proximity to the Pumping Plant Baker and the presence of a continuous sand layer under the levee section in the area that extends from around station 210+00 to beyond station 240+00. The area has a consistent 3 to 5-ft thick layer of SP material passing under the levee. Free field borings in the area of the river [B-11, -12, -13, -18 (SYLBR)] indicate this SP layer of sand is carried all the way through to the river at an approximate top elevation of 385 ft.

- Sylvan Bridge passes over the levee at station 224+75. The roadway is bridged over the east levee and is supported by an earthen embankment over the majority of the floodway except where the river thalweg passes through. The roadway is supported by another bridge deck in this area.
- The aerial photography/exploration location plan shows a shallow lake that extends from station 210+00 to 220+00 that is approximately 300 ft from the riverside toe of the levee. No bathymetry of this area was provided.

Station 311+00, East Alignment

The area of the Dallas Floodway around station 311+00 on the east alignment was selected for investigation due to its proximity to a pump station and the presence of a seepage entrance close to the flood-side levee toe. The seepage entrance corresponds to an effluent point of the pump station. The subsurface explorations in this area indicate the presence of a continuous poorly graded sand layer approximately 3 ft thick at its thinnest point under the levee section that extends from the pump station effluent pipe on the flood-side of the levee and daylights into the to the protected side ditch.

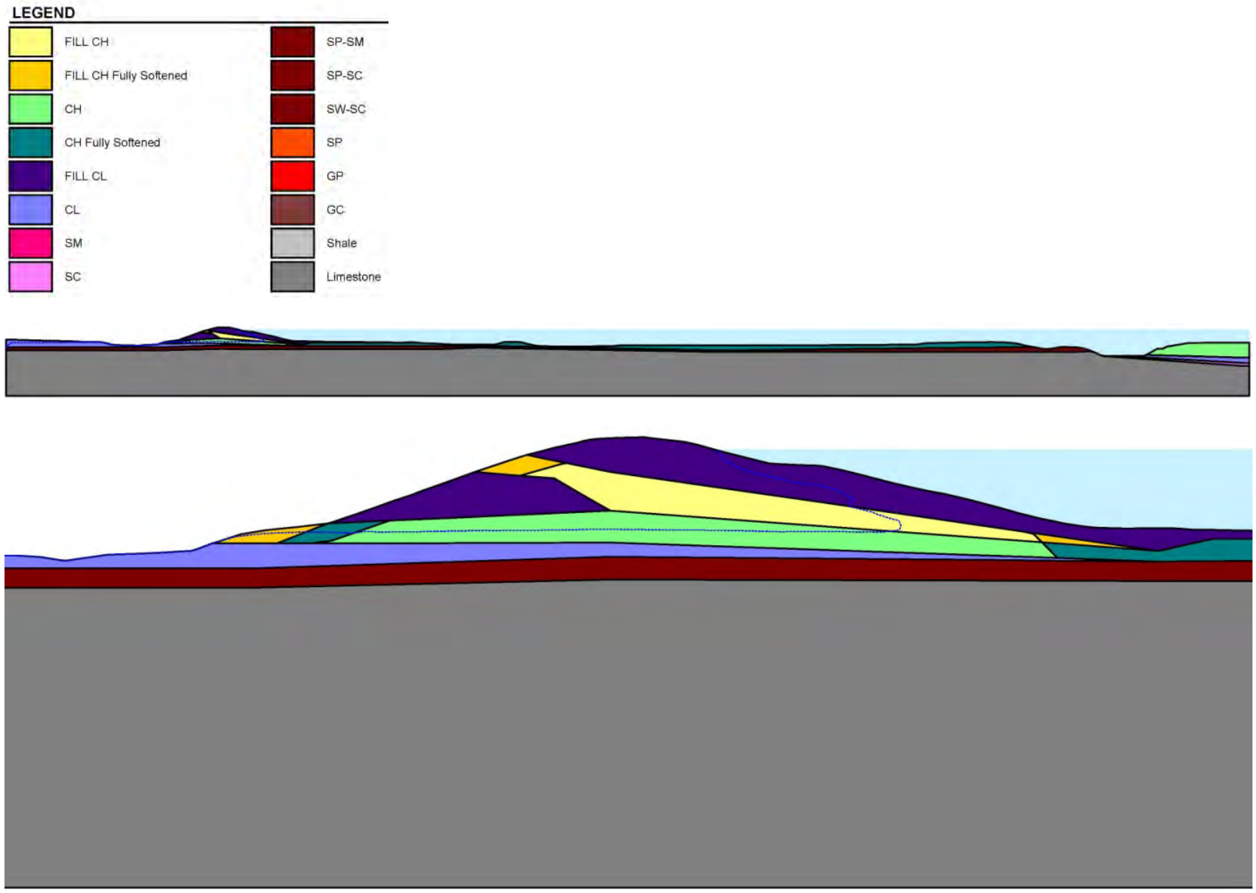
Station 410+00, East Alignment



A section at station 410+00 of the east alignment of the Dallas Floodway was selected for its analysis primarily due to the presence of an approximately 10 to 15-ft thick continuous sand layer that passes beneath the levee section. This area of the Dallas Floodway was examined around stations 410+00 and at 402+00. Both sections have a low spot in the floodway between the river and the levee. The major difference between the two is the location and depth of each from the flood-side levee toe. Station 410+00 has a low spot approximately 975 ft from the toe of the levee that extends to an elevation of 391 ft while station 402+50 has a low spot approximately 275 ft from the toe of the levee that extends to an elevation of 396 ft. Both sections have continuous sand layers that pass beneath the levee section. Both sections have upper layers of sand that classify as either SC or SM and range from 2 to 10 ft thick. Underlying those layers both sections also have sand layers that classify as either SW or an SP content that range from 5 to 15 ft thick. However, the explorations in proximity to station 410+00, the FER-10-11 series borings and CPTs, indicate the sand layers are as much as 5 feet higher in elevation than those seen at station 402+00 (FER-10-10 series borings and CPTs) making the low spot at station 410+00 the more critical with respect to seepage.

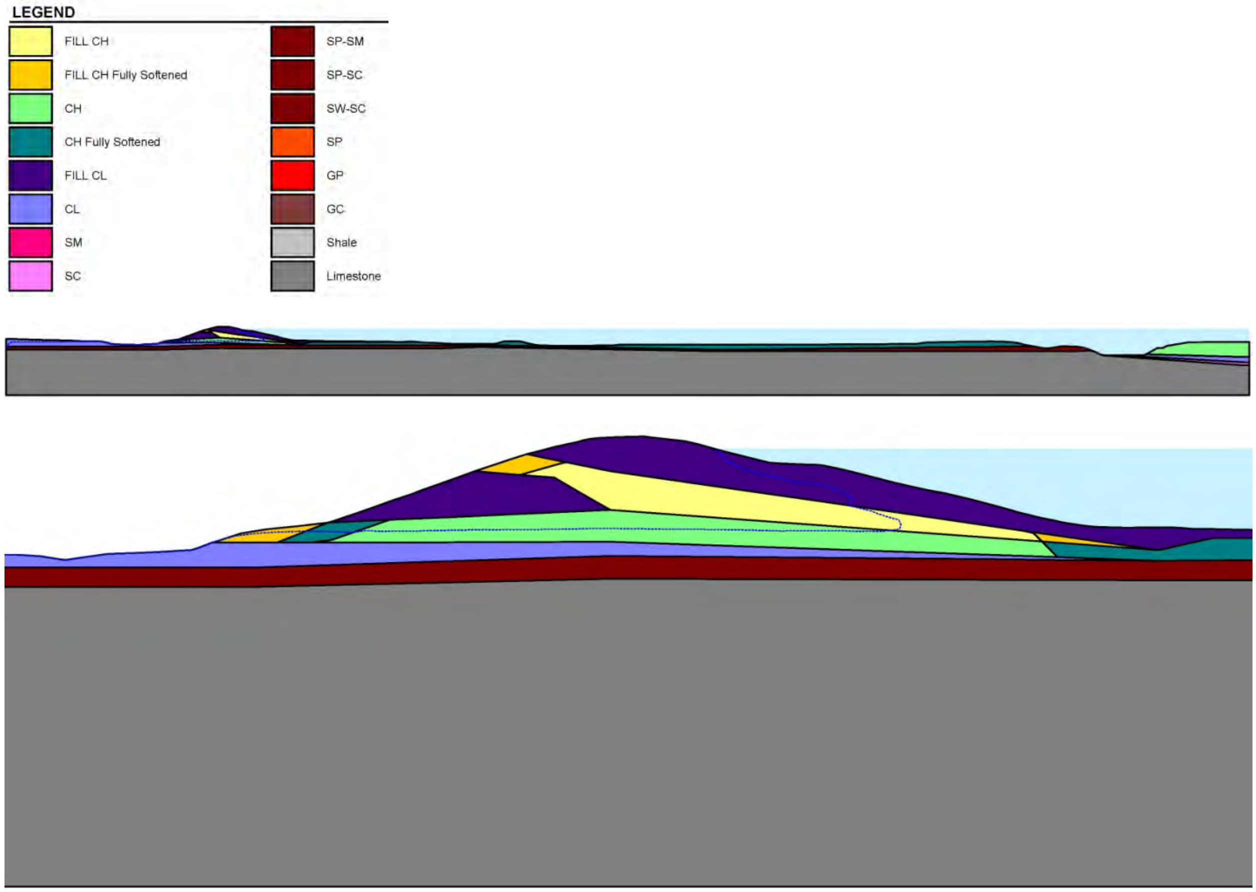
- It should be noted that no free field borings within the floodway between the river and levee have been located for either section. However, the graph of isolines depicting clay blanket thickness provided in Appendix F of the HNTB 408 Application Report indicates the blanket thickness on either side of the river is between 53 and 54 ft in this area. Toe borings from the west levee section opposite our cross-sections (FWR-08-18-WB, FWR-08-20-WB) indicate there is a sandy lean clay (CL) blanket down to elevation 361 ft overlying an SP/SW material. This would seem to indicate the clay blanket thickens as you move away from the east levee alignment.
- Stratigraphy lines determined from the east levee section borings were carried across the entire section in the drawings. This may not represent actual stratigraphic conditions.
- The protected-side crest of section 410+00 was artificially raised in a conservative measure to make the protected side topography data match what was available from the HNTB SLOPE/W file. Otherwise, data from the HNTB SLOPE/W file matched the topography for station 402+00.

Station 10+00, West Alignment



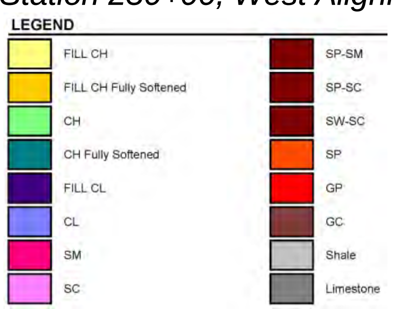
Station 10+00 on the west alignment was selected for analysis due to the presence of near surface sand layer in the area. It has an open seepage entrance near the Trinity River thalweg that connects to a continuous SP-SC material that passes under the levee section. There is an approximately 2-ft thick clay layer overlying the sand layer at the protected-side ditch.

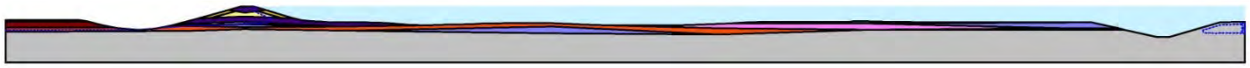
Station 188+00, West Alignment

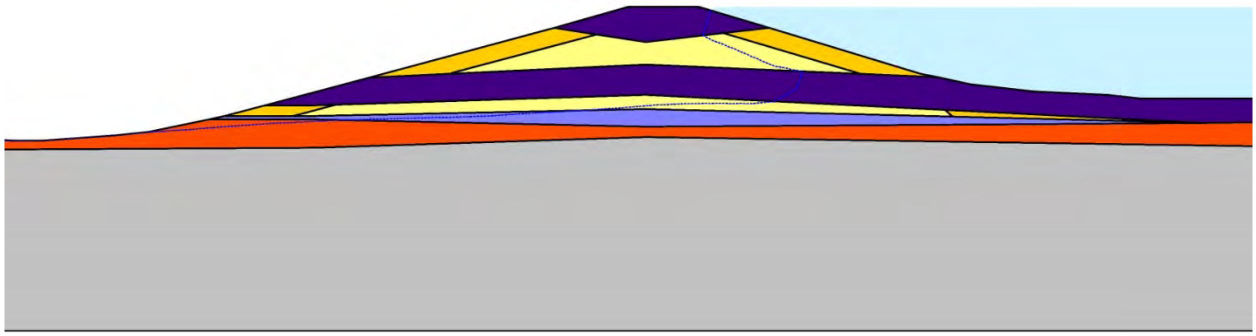


Station 188+00 on the west alignment was selected for analysis due to the presence of near surface sand layer in the area. It has an open seepage entrance near the Trinity River thalweg that connects to continuous SP, SP-SM/SC, SW-SC materials that pass under the levee section. The levee material at this section is made up of alternating layers of high and low plasticity clay.

Station 250+00, West Alignment



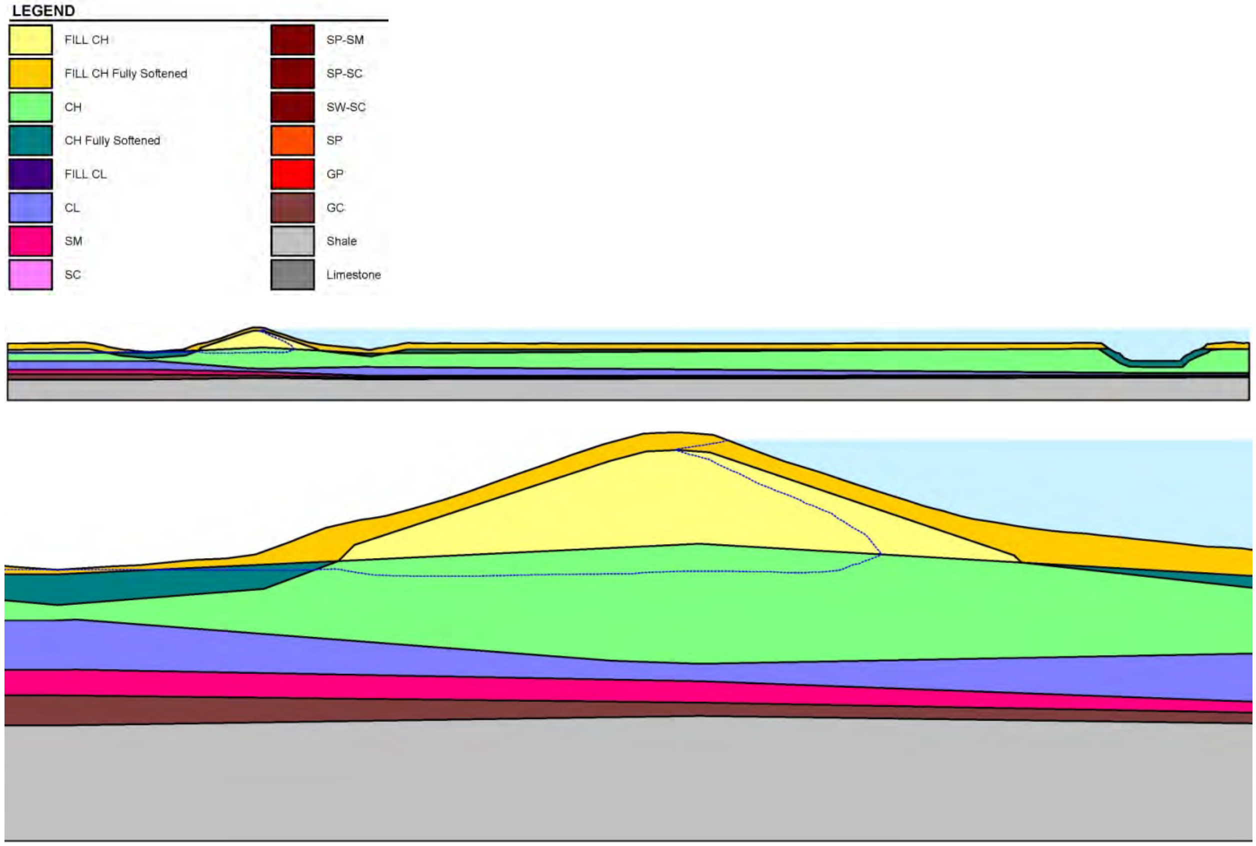




The section at station 250+00 on the west alignment of the Dallas Floodway was selected for analysis primarily due to a continuous SP and SC sand layer that extends beneath the levee. The sand layer daylights into the protected-side sump. Upon further investigation of the explorations that were advanced in the freeway for the Hampton Road Bridge (HR-4 through HR-7), it seems that the sand materials daylight in several areas between the toe of the levee and the Trinity River thalweg.

- Boring logs for the Hampton Road Bridge identified the clay and sand materials encountered as either “Fill Type A” or “Fill Type B” materials. This could mean that the driller/logger assumed that this was fill material, but it is more likely that the material was classified based on what type of material was available as a potential borrow source. Both SC and SP materials were classified as “Fill Type B”.
- The HNTB Geostudio file for this section used data from the FWR-07-09 series borings. The stratigraphy for the risk assessment model was slightly revised to take into account the FWR-07-08 borings located the same distance away but downstation rather than upstation. The principle difference is a more prevalent sand layer that occurs at the riverside toe.
- Borings A-7 and FWR-07-08-WB show an SP sand layer on the order of 7-ft thick with a top elevation of 399 ft. CPT’s FWR-07-08-WCPT and FWR-07-09-WCPT do not show conclusive evidence of the presence or absence of such a layer. It was not included in the HNTB analysis. The sand layer was included in the risk assessment analysis due to the presence of it noted in two borings.
- The continuous sand layer present in this section was classified as an SP rather than either an SC or an SP-SC due to the evidence of SP traits displayed in borings FWR-07-08-WB, -CB, FWR-07-09-CB. A 6-ft thick SP-SC layer was noted in boring FWR-07-09-DB (with an SC material overlying the SP-SC material from elevation 408 to 398 ft). However, the seeming lack of fines in the CPT logs for FWR-07-08-DCPT and -09-DCPT provided a rational basis to conservatively assume the SP material carried all the way through the section.

Station 335+00, West Alignment



The section at station 335+00 on the west alignment of the Dallas Floodway was selected for analysis primarily due to the continuous sand and gravel layers that extend beneath the levee. Stratigraphic information for area between the river thalweg and the levee toe was provided by borings WD-107, -67, and -28.

- The HNTB Geostudio file for this section used data from the FWR-08-15 series borings. The stratigraphy for the risk assessment model was slightly revised to take into account the FWR-08-14 borings located the same distance downstation rather than upstation.

Appendix B – Seepage Analysis

Seepage and stability analyses were performed on various sections of the Dallas Floodway Levee System in support of the risk assessment. These analyses were carried out before and during the assessment as a tool for use by the risk cadre to provide a greater understanding of how the performance of the levees will be affected by varying flood loads, varying material permeability and strength, and various deficiencies. The results provided reference points for an informed discussion by the entire risk analysis group during the elicitation process. All analyses were carried out using GeoStudio 2007, Version 7.17.

Seepage analyses were carried out on each cross section to provide an estimate of seepage through the levee section, gradients, and an estimate of pore water pressures for subsequent stability analyses. Each cross section has a suite of analyses developed for it that use three different sets of permeability estimates for each soil in each model and use two different historical storms scaled to three different heights to calculate 18 different seepage regimes. Following the calculation of each set of pore water pressures, a stability analysis is carried out to see how different hydrologic conditions affect the performance of the Dallas Floodway Levee System.

The levees are made up of either low or high plasticity clayey materials (or a mixture of both). Both of these materials have a relatively low permeability in comparison to coarser grained materials. Hydrologic records of the levee system indicate the Trinity River typically stays within its primary banks near the river thalweg the majority of the year and water is only against the levees during flood events. Therefore, it’s prudent to assume that flood waters will not have enough time to fully penetrate the levees and their foundations and subsequently develop steady state conditions during a Standard Project Flood (SPF) event or during a modified historical event that has a relatively long duration. Consequently, transient seepage analyses were performed for all sections instead of steady state seepage analyses. The transient analyses showed that the piezometric grade did not have an opportunity to stabilize to a steady state type of surface and failed to penetrate the more impervious areas of the levees and foundations.

Seepage Parameters

During the Probable Failure Mode Analysis (PFMA) session for the Dallas Floodway, three estimates for hydraulic permeability were provided for each material in the analysis by the risk cadre. The estimates were developed from a combination of laboratory testing and pump test data and reflect the team’s low, best, and highest reasonable estimates for permeability. These values are shown in Table 14. The permeability estimates for high plasticity clay were used to model the foundation shale. Each set of permeability estimates were used in a seepage analysis for each cross section in order to further understand how changes in permeability affect the stability of the levee sections.

Table 12 - Seepage Parameter Estimates

Material Type	ft/s			cm/s		
	Low	Best	High	Low	Best	High

SP	6.50E-05	1.30E-03	2.40E-03	1.98E-03	3.96E-02	7.32E-02
CH	1.00E-08	1.00E-07	1.00E-05	3.05E-07	3.05E-06	3.05E-04
CH FSS	1.00E-06	1.00E-05	1.00E-04	3.05E-05	3.05E-04	3.05E-03
GP, GW	1.00E-03	1.00E-02	6.00E-02	3.05E-02	3.05E-01	1.83E+00
GC	4.00E-04	1.00E-03	3.00E-03	1.22E-02	3.05E-02	9.14E-02
CL	2.40E-09	5.00E-07	5.30E-05	7.32E-08	1.52E-05	1.62E-03
SC	1.00E-08	1.00E-06	3.00E-04	3.05E-07	3.05E-05	9.14E-03
SM	1.70E-09	1.50E-04	4.50E-04	5.18E-08	4.57E-03	1.37E-02
SW-SC, SW-SM	1.40E-05	1.00E-03	5.20E-03	4.27E-04	3.05E-02	1.58E-01
Shale	1.00E-09	1.00E-09	1.00E-09	3.05E-08	3.05E-08	3.05E-08
Limestone	1.00E-09	1.00E-09	1.00E-09	3.05E-08	3.05E-08	3.05E-08

All materials were modeled in GeoStudio as "Saturated/Unsaturated" materials having volumetric water content and hydraulic conductivity functions to define their behavior in the model. The volumetric water content functions were generated with sample functions in GeoStudio that use an estimate of the zero-pressure saturated volumetric water content. Different sample functions are available for different soils such that the sample function for clay was used to develop the water content function for the clays, the silty sand function was used for silty sand, and so on. The estimates of zero-pressure saturated volumetric water content used for the different soils in the seepage model ranged from 0.25 to 0.4 ft3/ft3 with the coarse grained material using the lower value and fine grained material trending to the higher value. The values were based on typical volumetric water contents found on Figure 4-2 in the SEEP/W manual. Values of volumetric compression (Mv) were left at zero. The hydraulic conductivities were estimated using the Van Genuchten estimation method, the volumetric water content functions just described, and the permeabilities from Table 14.

Boundary Conditions

The flood loading was applied to the free field ground surface between the river and the flood side toe as well as to the flood side of the levee itself. This boundary condition was defined by scaled hydrographs of two previous storm events. The base hydrographs were provided by the USACE Fort Worth District and are shown on Figure 9. The first storm event was the May 1990 flood that resulted in the record pool for the levee system and provided a peak mean daily flow of approximately 72,100 cfs. The second storm was the June 2007 event which was a smaller storm with a slightly shorter duration having a peak mean daily flow of 35,700 cfs. Both storms were scaled to provide hydrographs that have peak water level elevations that correspond to 100, 75, and 50% of the total levee height at each of the sections that were evaluated. Independent hydrographs were produced for each section that was analyzed. This resulted in 6 different boundary conditions compiled for each cross section based on the fact that there were 2 storm events and 3 different water level elevations. Each hydrograph was run with a duration of 2136 hours, or 89 days.

Initially, seepage analyses were done using both storm events and all 3 water levels for each section. After analysis suites of several sections were completed, it became apparent that the 1990 flood event was the more critical boundary condition and the 2007 storm was not run for the remaining sections. Likewise, boundary conditions using peak flood levels that correspond to 50 and 75% of the levee height were not run for all sections as the storms scaled to 100% of the levee height represented the critical condition.

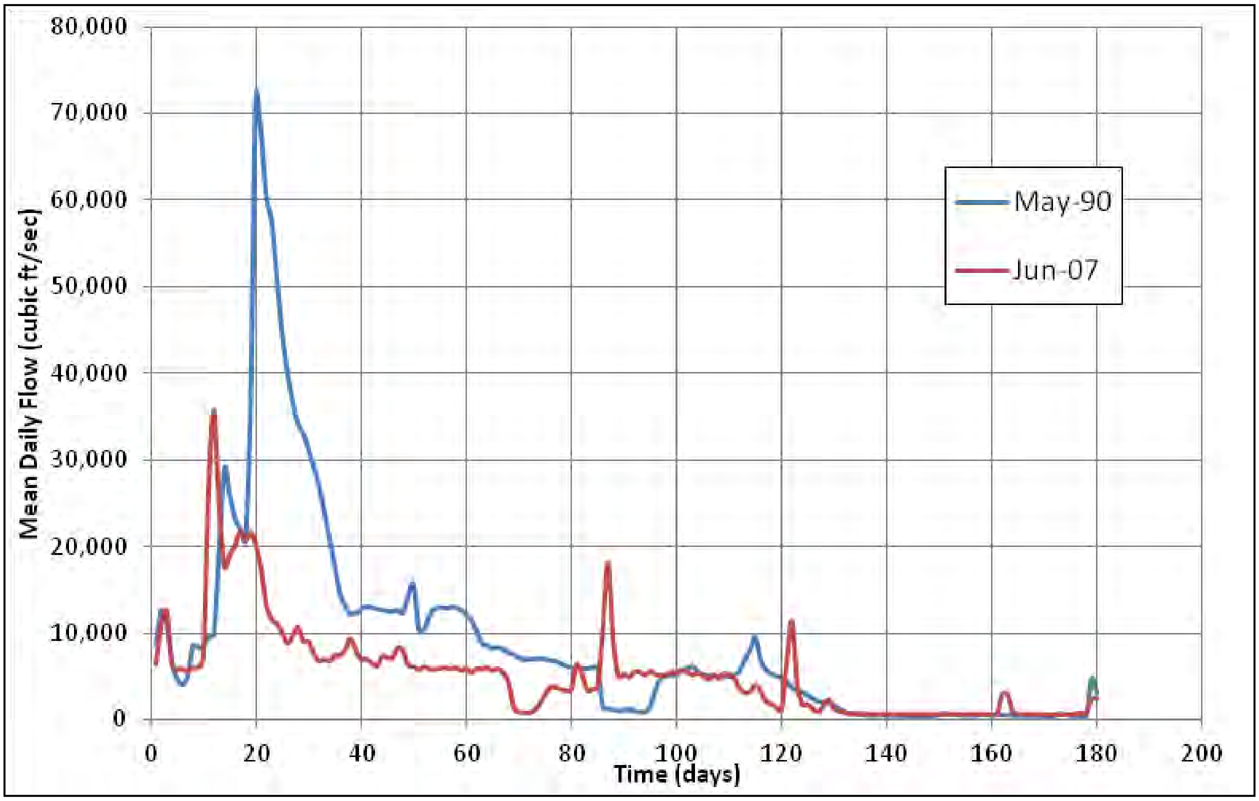


Figure 34 - Graphs of the base hydrographs used for the investigation.

In order to glean more information out of the seepage model, the dry side of the levee and the free field surface boundary conditions were set as review nodes. In addition, the end boundary conditions were set up to be infinite regions with material properties equivalent to adjacent regions. Infinite regions allow seepage to flow into the edges of the cross section as if the seepage model was infinitely long. This eliminates the potential for seepage pressures to back up during the analysis and produce artificially inflated pore pressures due to the limited extents the model boundaries.

Appendix C – Stability Analysis

A stability analyses were run using the results of every seepage analyses as a parent analysis in SLOPE/W. The stability analyses provide the metric that describes how robust the levee system is under the changing seepage conditions. All stability analyses carried out for this investigation used the optimization feature in SLOPE/W to determine the most critical failure surface. Stability analyses were performed using the step in the seepage analysis that corresponds to the peak flood stage of the flood event. Some additional stability analyses were also done on time steps beyond the peak time to account for the possibility that later stages could produce more critical pore pressures. None of these sensitivity cases were found to be more critical to the model. Also, because the hydrographs were scaled without compensating for volume effects, the duration is likely conservative.

As discussed in the preceding Seepage Analysis section, it is anticipated that steady state conditions will not have an opportunity to develop due to the brief nature of flood events and low permeabilities of the fine-grained soils in the levees and foundations of the Dallas Floodway Levee System. Therefore, drained shear strength parameters were used for the stability analyses.

The stability parameters used in the investigation were based on laboratory strength data that was reported in the Geotechnical Appendix of HNTB's 408 Application Report. Similar to how the strength estimates were prepared in the 408 Report, laboratory strength testing data was grouped by soil type and plotted on a graph. The 408 report applied a linear best-fit line to the coarse-grained soil data and non-linear envelopes for the fine-grained soils. Those estimates of strength were used for subsequent evaluations of the levee system. For the seepage and stability investigation developed for the risk assessment, the same data was plotted in the same fashion but a curvilinear best-fit line was used to define the behavior of each soil in order to account for the change in the test results with changing confining stresses. In addition to the curvilinear best-fit line, three linear best-fit lines were also used to define the upper, lower, and best estimate of shear strengths suggested by the data.

Table 13

Material	Parameter	Min	Best	Max
CH Fill	Phi	15.5	18.4	30
	c (psf)	100	300	500
CH	Phi	16.7	19.3	26.6
	c (psf)	200	250	300
CL Fill	Phi	21.3	23.5	31
	c (psf)	100	300	500
CL	Phi	18.4	24	26.5
	c (psf)	150	300	500
CH FSS	Phi	14	18	27
	c (psf)	100	180	250

Material	Parameter	Min	Best	Max
Basal Sands	Phi	29	32	34
	c (psf)	-	-	-
Clean Basal Gravel	Phi	32	35	38
	c (psf)	-	-	-
Clayey Sand	Phi	27	30	32
	c (psf)	-	-	-
Shale	Phi	15	24	36
	c (psf)	200	1950	3000

The shear strength data was used in two ways in the stability analyses. First, the curvilinear strength envelope that represents the best estimate of the strength of each soil was used with the best estimates of permeability. This method was used at the start of the analysis to provide the overall best estimate of levee performance. The second method used the straight line estimates that represent the upper, lower, and best estimate of the shear strength of each material in a probabilistic stability analysis to determine the threshold at which the levee system would cease to perform as it was designed. This method made up the bulk of the investigation. The maximum and minimum anticipated shear strength limits for each material was defined by the upper and lower bound estimates, respectively, according to a triangular distribution pattern with the best estimate of shear strength as its peak. Probabilistic stability analyses were then ran on each seepage analysis result to provide an indication of how levee performance changed due to whether upper, lower, or best estimates of material permeabilities were used. Each probabilistic stability analysis uses Monte Carlo simulation to run 10,000 individual stability analyses that randomly varied the shear strength for each material. The results provide a graph depicting the distribution of the results (normal), the mean factor of safety, a minimum factor of safety, and the probability of failure (the chance that the factor of safety is less than unity). What is shown on the output of the stability analysis, however, is the factor of safety determined using the best estimates of shear strength (see Figure 38 below).

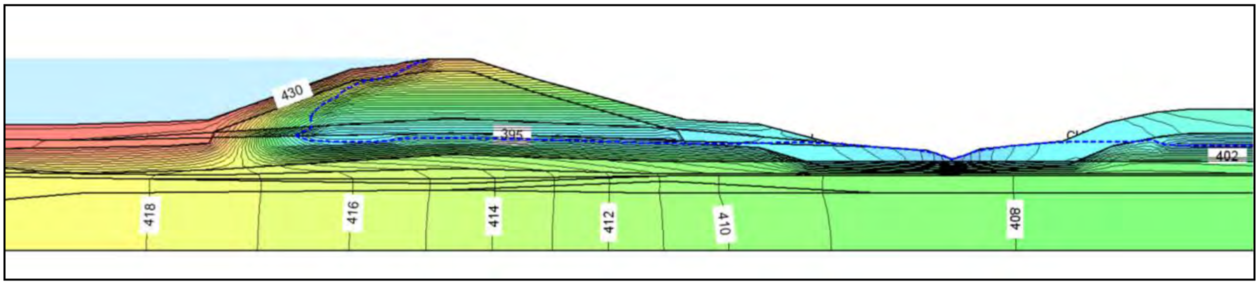


Figure 35 – Example Seepage Analysis Results-Best Permeability Estimates at STA 220+00 East Levee, Seepage Results Showing Total Head

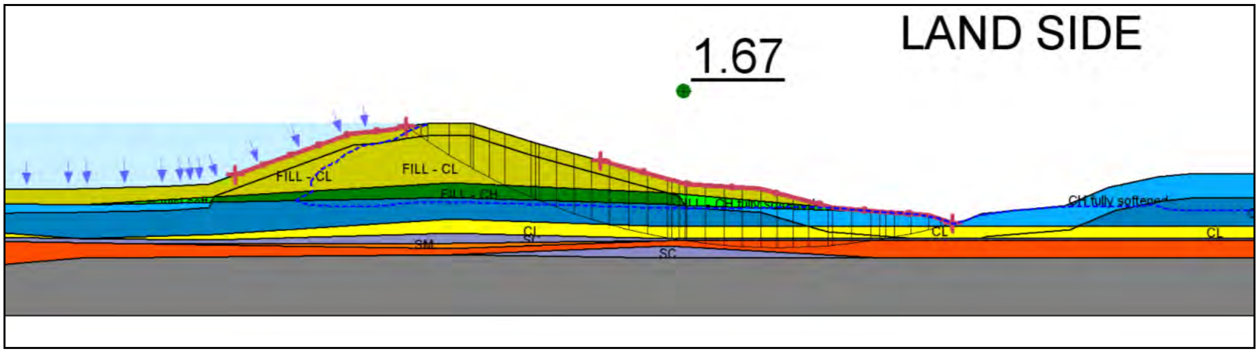
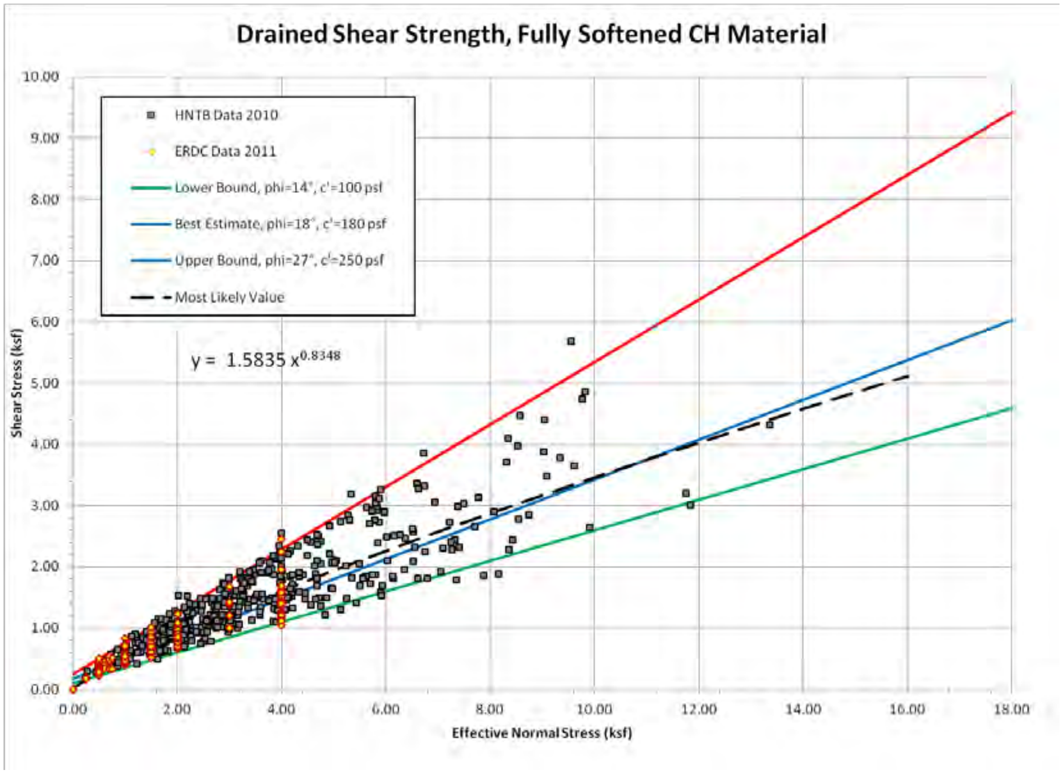


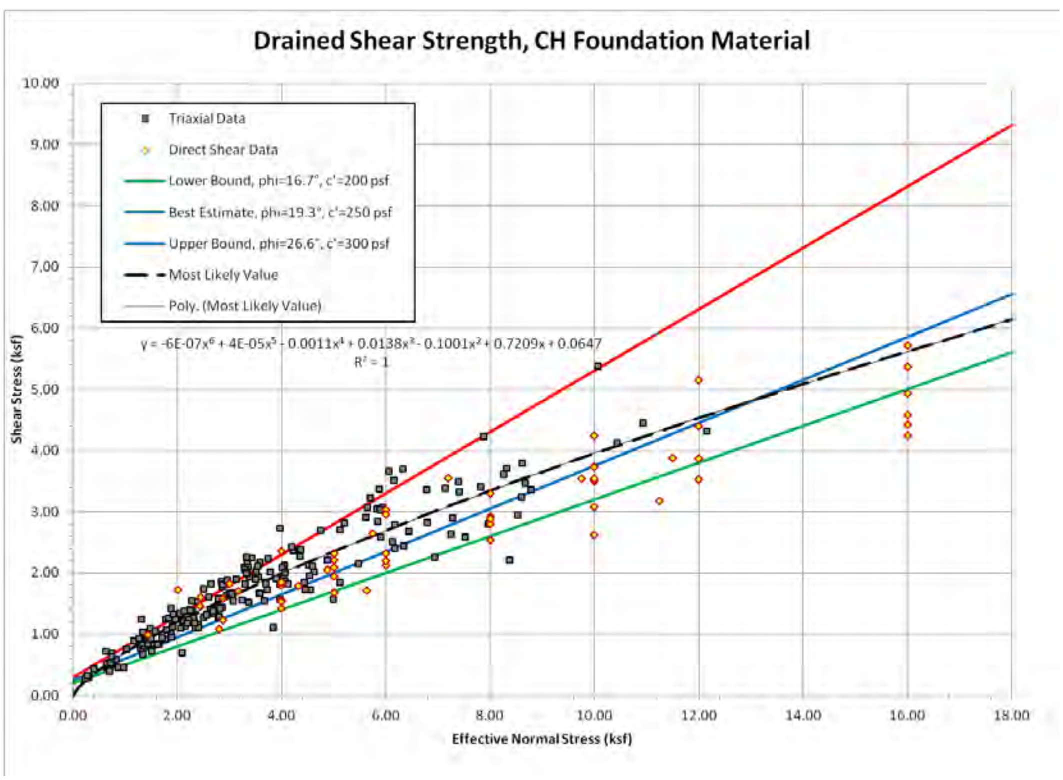
Figure 36 – Example Stability Analysis Results-Best Permeability Estimates at STA 220+00 East Levee

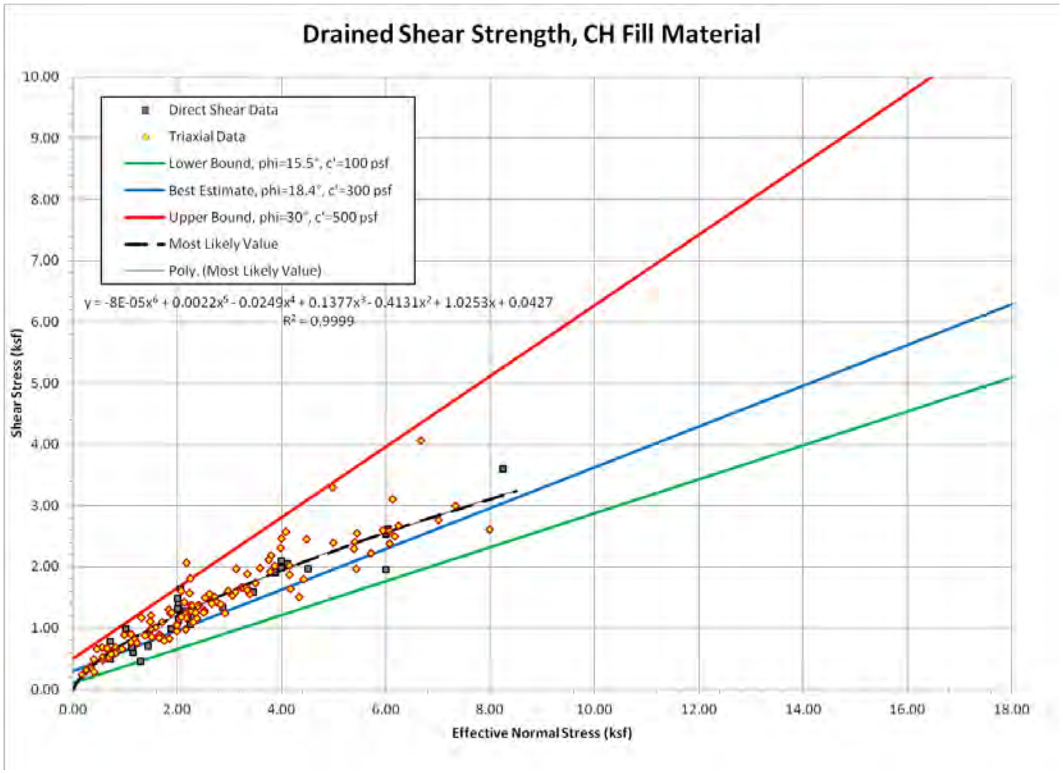
The numerical definitions of the upper, lower, and best estimates of drained shear strength can be seen in Table 15. The unit weights made for the material are shown in the table below. A total moist unit weight of 125 lbs/ft3 was used for the basal sands (except for SC).

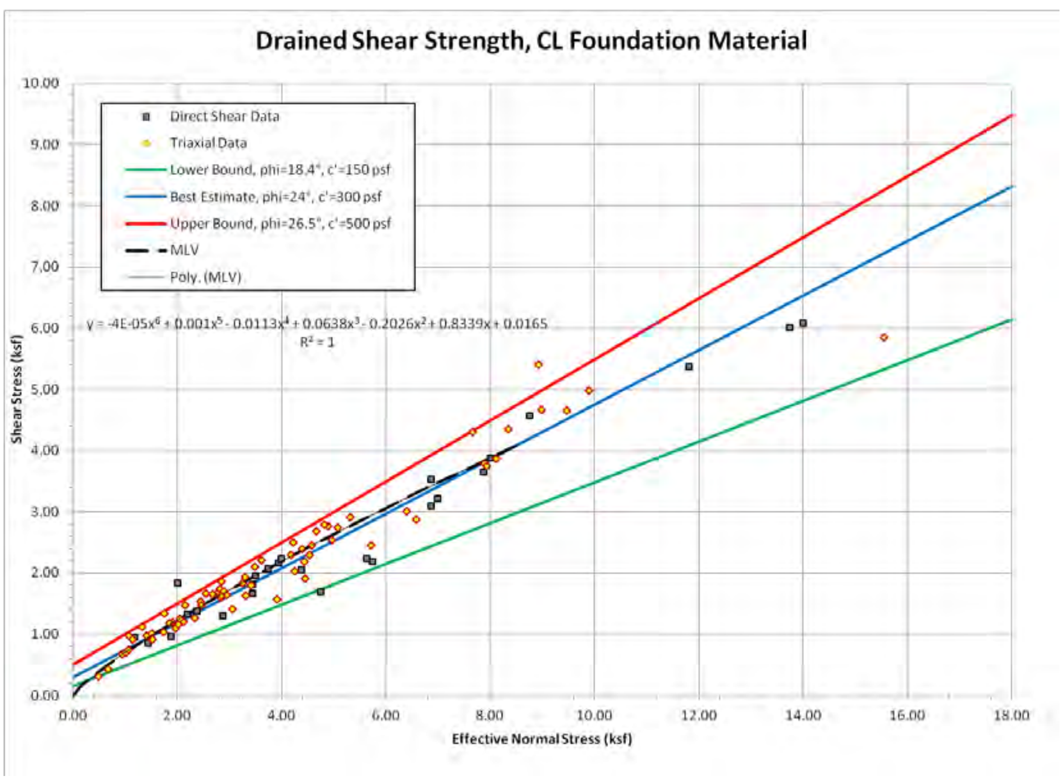
West Levee Moist Unit Weight (pcf)						
	CH - Fill	CH	CL - Fill	CL	SC	Shale
Mean	122.3	123.2	126.9	127.2	127	132.3
Min	105.8	102.3	96.9	113.6	116.3	112.5
Max	136..1	142	149.5	162.1	139.5	141.2
Samples	176	330	46	130	19	121

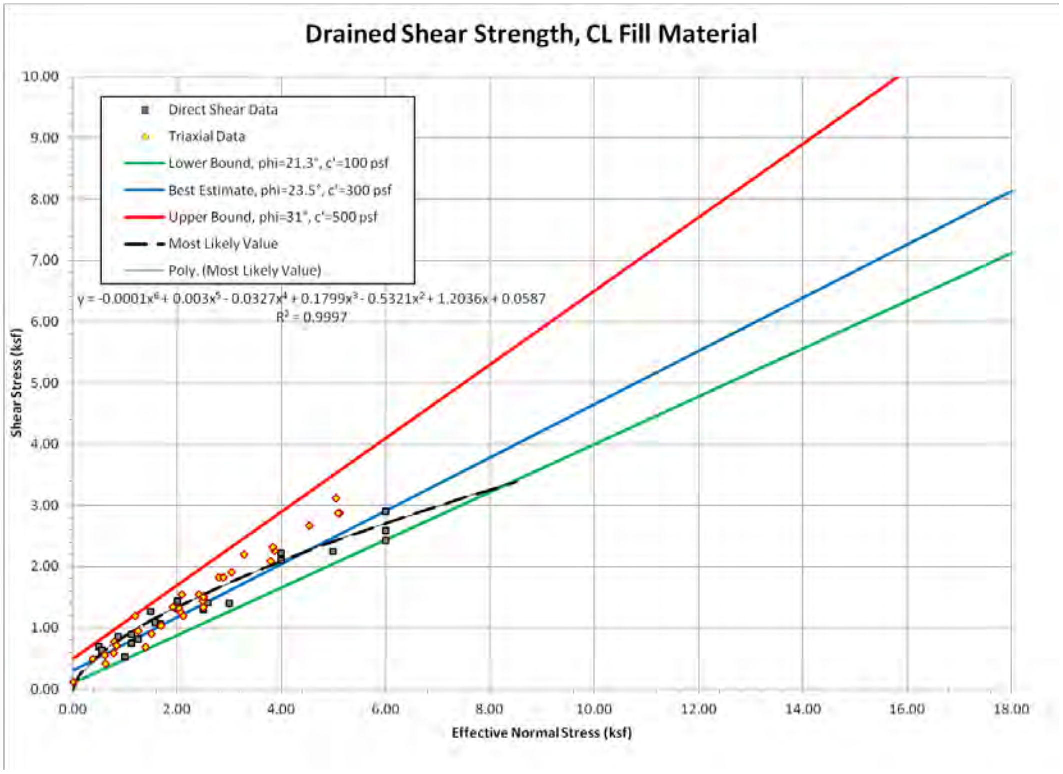
East Levee Moist Unit Weight (pcf)						
	CH - Fill	CH	CL - Fill	CL	SC	Shale
Mean	120.2	122.1	123.1	125.6	126.5	133.2
Min	103.3	101.6	102.7	108.4	108.1	125.5
Max	137.9	139.6	136.9	138.1	137.2	137.4
Samples	121	408	72	181	27	11

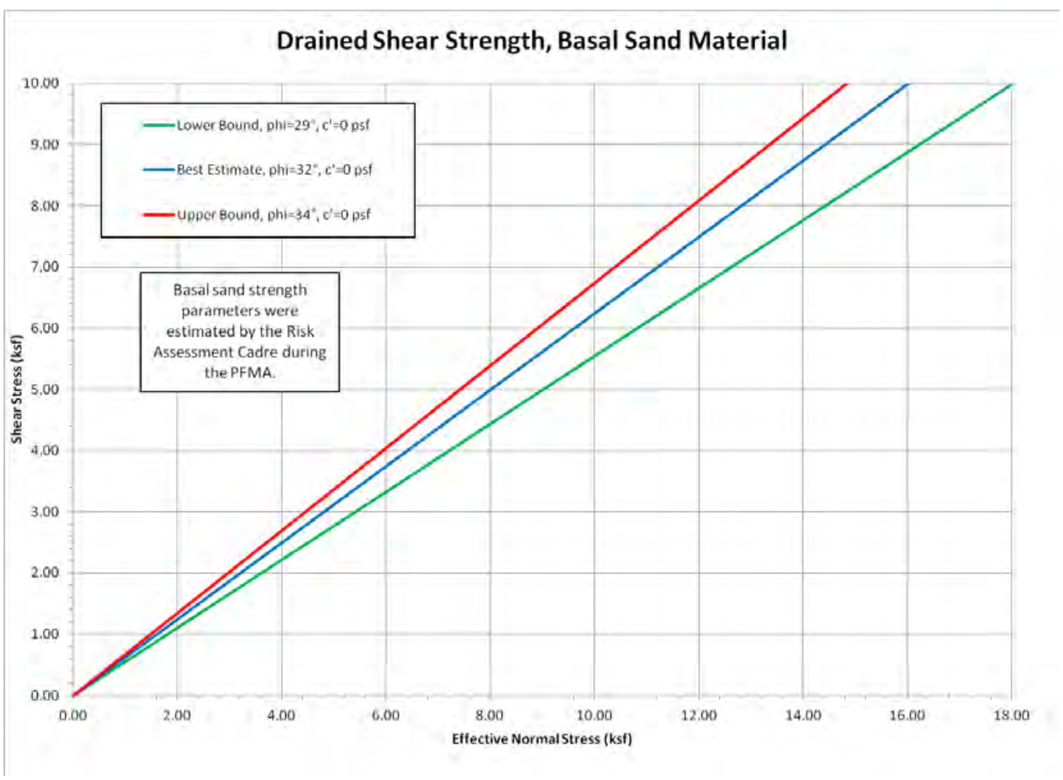

Drained Shear Strength, Fully Softened CH Material
HNTB Data 2010
ERDC Data 2011
Lower Bound, phi=14°, c'=100 psf
Best Estimate, phi=18°, c'=180 psf
Upper Bound, phi=27°, c'=250 psf
Most Likely Value
y = 1.5835 x^0.8348
Shear Stress (ksf)
Effective Normal Stress (ksf)

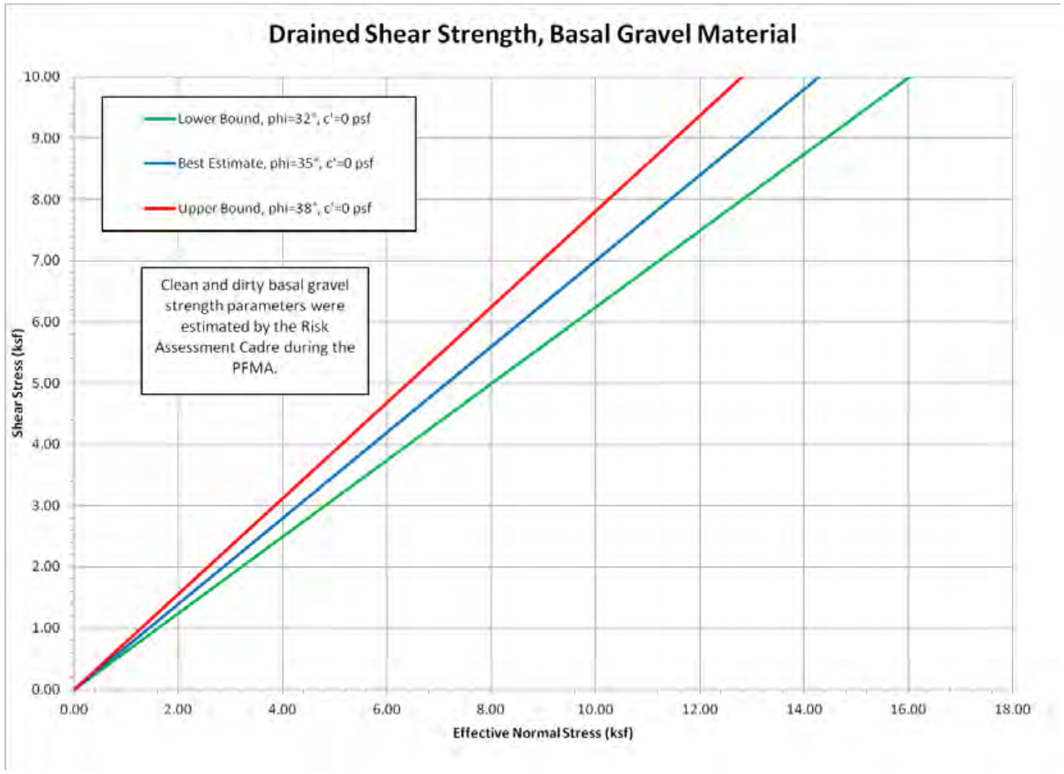

Drained Shear Strength, CH Foundation Material
Triaxial Data
Direct Shear Data
Lower Bound, phi=16.7°, c'=200 psf
Best Estimate, phi=19.3°, c'=250 psf
Upper Bound, phi=26.6°, c'=300 psf
Most Likely Value
Poly. (Most Likely Value)
y = -6E-07x^6 + 4E-05x^5 - 0.0011x^4 + 0.0138x^3 - 0.1001x^2 + 0.7209x + 0.0647
R^2 = 1
Shear Stress (ksf)
Effective Normal Stress (ksf)

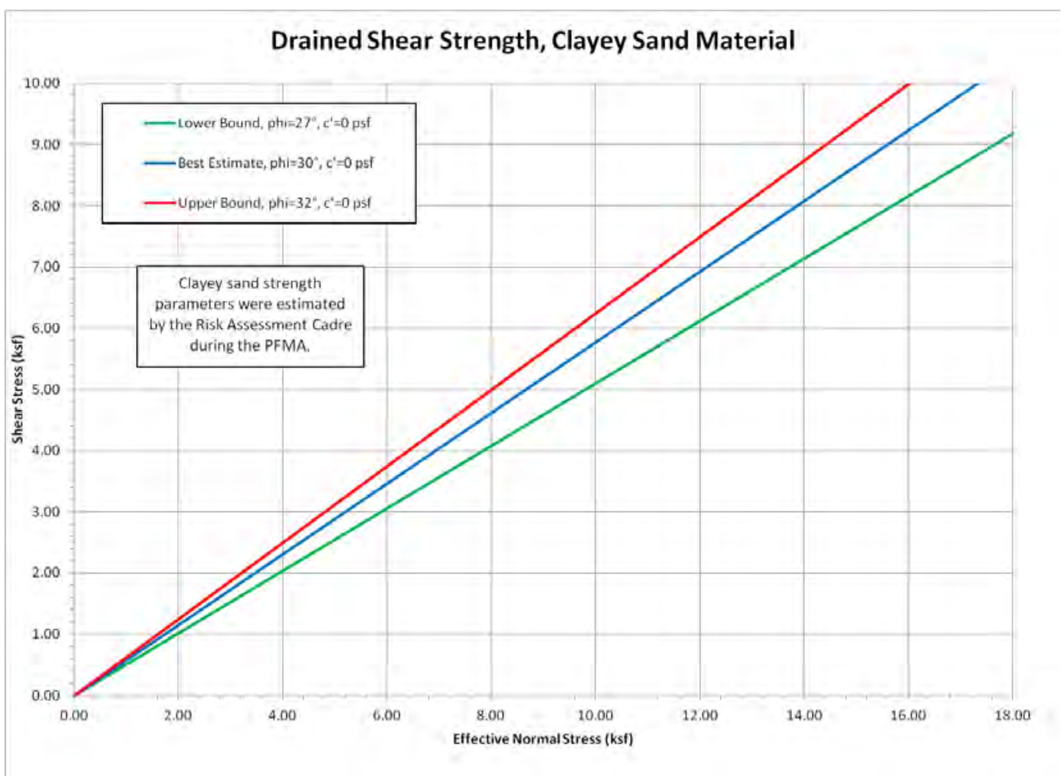

Drained Shear Strength, CH Fill Material
Direct Shear Data
Triaxial Data
Lower Bound, phi=15.5°, c'=100 psf
Best Estimate, phi=18.4°, c'=300 psf
Upper Bound, phi=30°, c'=500 psf
Most Likely Value
Poly. (Most Likely Value)
y = -8E-05x⁶ + 0.0022x⁵ - 0.0249x⁴ + 0.1377x³ - 0.4131x² + 1.0253x + 0.0427
R² = 0.9999
Shear Stress (ksf)
Effective Normal Stress (ksf)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
12.00
14.00
16.00
18.00

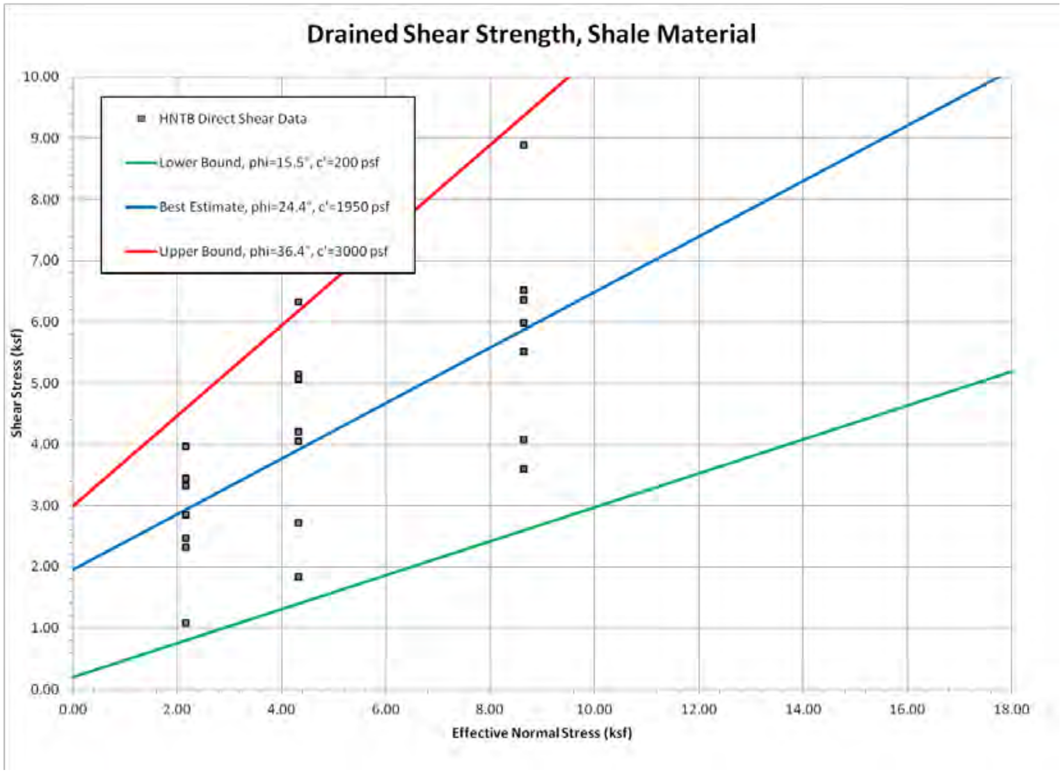

Drained Shear Strength, CL Foundation Material
Direct Shear Data
Triaxial Data
Lower Bound, phi=18.4°, c'=150 psf
Best Estimate, phi=24°, c'=300 psf
Upper Bound, phi=26.5°, c'=500 psf
MLV
Poly. (MLV)
y = -4E-05x⁶ + 0.001x⁵ - 0.0113x⁴ + 0.0638x³ - 0.2026x² + 0.8339x + 0.0165
R² = 1
Shear Stress (ksf)
Effective Normal Stress (ksf)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
12.00
14.00
16.00
18.00


Drained Shear Strength, CL Fill Material
Direct Shear Data
Triaxial Data
Lower Bound, phi=21.3°, c'=100 psf
Best Estimate, phi=23.5°, c'=300 psf
Upper Bound, phi=31°, c'=500 psf
Most Likely Value
Poly. (Most Likely Value)
y = -0.0001x6 + 0.003x5 - 0.0327x4 + 0.1799x3 - 0.5321x2 + 1.2036x + 0.0587
R2 = 0.9997
Shear Stress (ksf)
Effective Normal Stress (ksf)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
12.00
14.00
16.00
18.00


Drained Shear Strength, Basal Sand Material
Lower Bound, phi=29°, c'=0 psf
Best Estimate, phi=32°, c'=0 psf
Upper Bound, phi=34°, c'=0 psf
Basal sand strength parameters were estimated by the Risk Assessment Cadre during the PFMA.
Shear Stress (ksf)
Effective Normal Stress (ksf)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
12.00
14.00
16.00
18.00


Drained Shear Strength, Basal Gravel Material
Lower Bound, phi=32°, c'=0 psf
Best Estimate, phi=35°, c'=0 psf
Upper Bound, phi=38°, c'=0 psf
Clean and dirty basal gravel strength parameters were estimated by the Risk Assessment Cadre during the PFMA.
Shear Stress (ksf)
Effective Normal Stress (ksf)


Drained Shear Strength, Clayey Sand Material
Lower Bound, phi=27°, c'=0 psf
Best Estimate, phi=30°, c'=0 psf
Upper Bound, phi=32°, c'=0 psf
Clayey sand strength parameters were estimated by the Risk Assessment Cadre during the PFMA.
Shear Stress (ksf)
Effective Normal Stress (ksf)



Seepage and Stability Results

The seepage and stability investigation done for the risk assessment of the Dallas Floodway Levee System was carried out to identify the threshold at which the levee system ceases to perform as it should. This was done by systematically varying individual parameters and geometries and investigating the levees' response to assorted detrimental effects. Upon establishing which sections were going to be analyzed, an initial determination was made before the risk assessment to vary the material permeabilities, the shear strengths of the materials, and the loading conditions. The results of these initial analyses were presented to the cadre at the start of the risk assessment in order to provide some insight to how robust the levee system is. As the risk assessment continued into its second week shear strengths, permeability parameters, levee and subsurface geometries, and detrimental phenomenon were being compared and contrasted based on issues that were brought up by the cadre during the risk assessment discussion. What resulted was a sensitivity analysis with the goal of providing those in attendance a greater understanding of how robust and resilient the levees are so a more informed risk assessment could be made.

Permeability

The first parameters of the investigation to be varied were the material permeabilities. As discussed in the preceding Seepage Parameters subsection, seepage analyses were performed using all three estimates of hydraulic permeability for each material: the low, high, and best estimate. The results of the subsequent stability analyses indicate that the single largest factor that affects the stability of the levee system is the permeability of the foundation and levee materials. Table 16, Table 17 , and

Table 18 show the seepage and stability results of the section at Station 220+00 using the best, low and high estimates of permeability, respectively. The most stable levee models were those using the low permeability estimates having a mean factor of safety of 1.84. The mean factor of safety is the mean of the 10,000 Monte Carlo stability analysis runs. Because of the relatively brief nature of the flood event and the fact that the seepage analyses used the low estimate of permeability for the soils, seepage was largely unable to penetrate into the levee. Therefore, there were, in most cases, no excess pore pressures induced in the levee and foundation which would serve to reduce the effective stress and subsequently reduce the stability of the section. The least stable models were those using the high estimate of permeability having a mean factor of safety of 1.24. The greater permeability of the material now allowed seepage to generate higher pore pressures which reduces the effective stress. As one would expect, models that used the best estimates of permeability returned results that lie between the low and high estimates at a mean factor of safety of 1.77.

Table 14 - 1990 Hydrograph, River Elevation at Crest of Levee

	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**1.89**	**1.66**	**2.86**	**2.35**	**1.20**	**2.37**	**1.86**	**2.50**
Low k, Prob Str	Mean	**2.19**	**1.84**	**2.87**	**2.06**	**2.71**	**2.71**	**2.04**	**2.38**
	P(f)	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min	1.65	1.43	1.89	1.69	2.00	2.03	1.47	1.77
Best k, Prob Str	Mean	**2.18**	**1.77**	**2.63**	**2.30**	**1.38**	**2.38**	**1.7**	**2.38**
	P(f)	0.00%	0.00%	0.00%	0.00%	13.00%	0.00%	0.00%	0.00%
	Min	1.58	1.31	1.84	1.65	0.95	1.98	1.21	1.77
High k, Prob Str	Mean	**1.91**	**1.24**	**2.35**	**1.88**	**1.89**	**2.24**	**1.56**	**2.15**
	P(f)	0.00%	1.67%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
	Min	1.48	0.82	1.58	1.53	1.41	1.87	1.12	1.61

Table 15 - 1990 Hydrograph, River Elevation at 75% Height of Levee

	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**2.22**	**1.84**		**2.35**	**1.46**	**2.43**		**2.50**
Low k, Prob Str	Mean		**2.05**		**2.09**	**2.71**	**2.71**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.71		1.72	2.00	2.03		1.77
Best k, Prob Str	Mean		**1.89**		**2.30**	**1.60**	**2.42**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.56		1.66	1.15	2.02		1.77
High k, Prob Str	Mean		**1.56**		**1.95**	**2.05**			**2.31**
	P(f)		0.00%		0.00%	0.00%			0.00%
	Min		1.15		1.60	1.54			1.77

Table 16 - 1990 Hydrograph, River Elevation at 50% Height of Levee

	FoS	**74+00 E**	**220+00 E**	**311+00 E**	**410+00 E**	**10+00 W**	**250+00 W**	**335+00 W**	**188+00 W**
Best k, Best Str	FoS	**2.11**	**1.95**		**2.36**	**1.76**	**2.49**		**2.50**
Low k, Prob Str	Mean		**2.08**		**2.30**	**2.71**	**2.71**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.71		1.66	2.00	2.03		1.77
Best k, Prob Str	Mean		**2.01**		**2.30**	**1.86**	**2.5**		**2.38**
	P(f)		0.00%		0.00%	0.00%	0.00%		0.00%
	Min		1.56		1.6568	1.38	2.06		1.77
High k, Prob Str	Mean		**1.82**		**2.01**	**2.09**			**2.38**
	P(f)		0.00%		0.00%	0.00%			0.00%
	Min		1.15		1.65	1.70			1.77

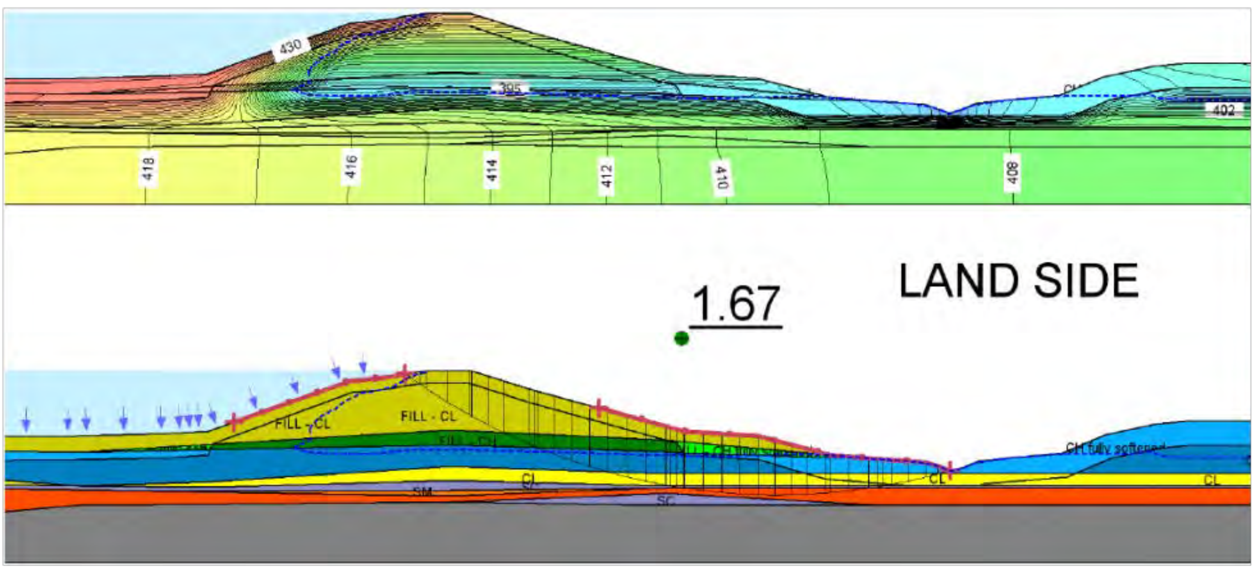


Figure 37 - Seepage and stability analysis results using the best estimates of material permeabilities. The top figure shows the results of the seepage analysis depicting total head isolines. The bottom figure shows the stability analysis. The dotted blue line represents the piezometric surface

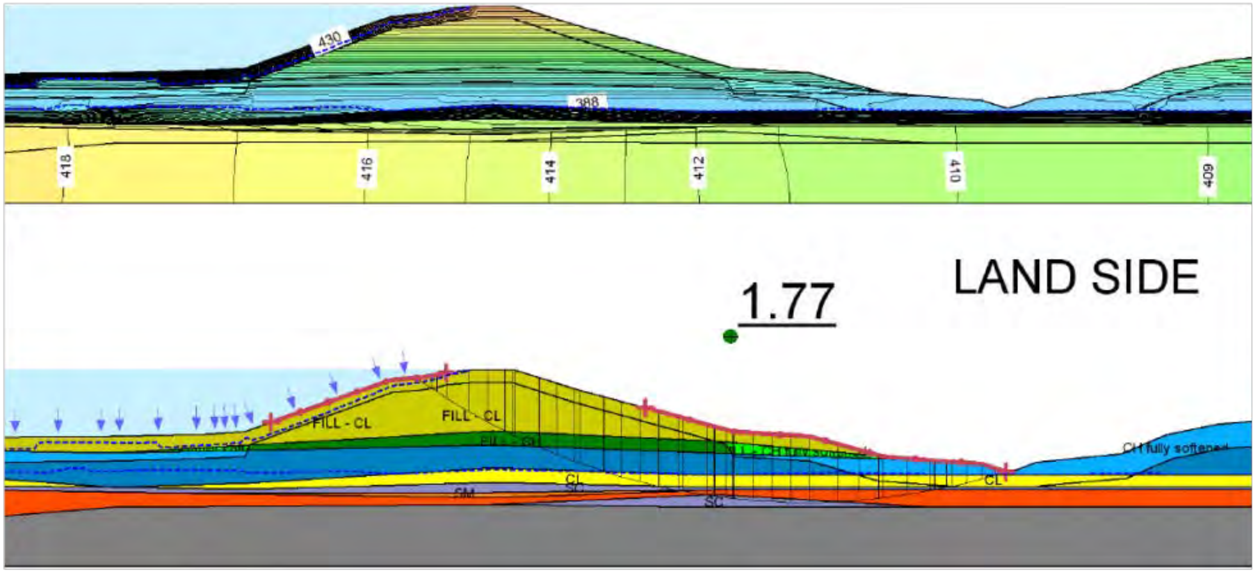


Figure 38 - Seepage and stability analysis results using the low estimates of material permeabilities. The top figure shows the results of the seepage analysis depicting total head isolines. The bottom figure shows the stability analysis. The dotted blue line represents the piezometric surface

Desiccation

Perhaps one of the least understood phenomena present on the Dallas Floodway is how pervasive desiccation is in the levee system. Much of the foundation soil and many of the levee sections are made up of high plasticity clays which are subject to desiccation. The extent of the desiccation into the levee and foundation soils, however, is a subject of some debate. The construction history of the original levee system isn't completely clear so it's possible that some of the foundation materials beneath the existing levee had been subject to desiccation prior to construction and were not reworked before construction began. In addition, when the system was rebuilt in the 1950's and 1960's it's not clear how much the surface of the pre-existing levee sections were reworked before the additional material was placed.

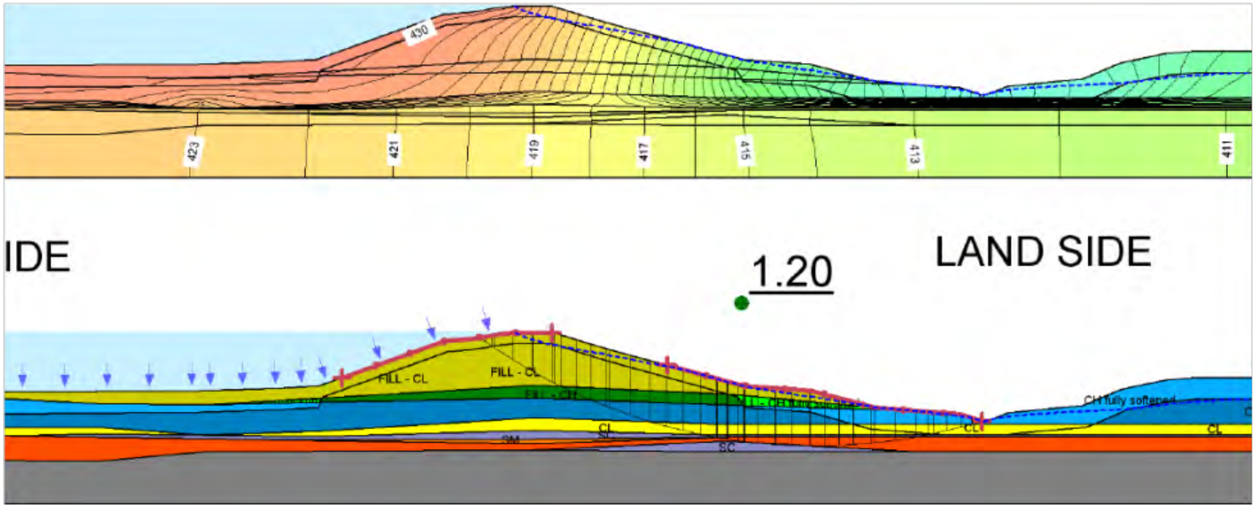


Figure 39 - Seepage and stability analysis results using the high estimates of material permeabilities. The top figure shows the results of the seepage analysis depicting total head isolines. The bottom figure shows the stability analysis. The dotted blue line represents the piezometric surface.

Desiccation was modeled in the seepage and stability analysis in two ways. First, the permeability of potentially desiccated soils was increased over that of soils considered to be intact. It was assumed that as cracks and fissures open up in high plasticity clays due to prolonged stages of drying, the cracks either remain open during a flood event or more permeable materials such as sand or gravel in-fill the desiccation cracks and hold them open during an event. This results in a higher overall permeability for the soil mass. Second, desiccated soils were considered to have fully softened soil strengths in the stability analyses. Approximately 50 near-surface slope failures (also referred to as infinite slope failures or skin slides) have been observed since the levees were modified in the 1960's. This could indicate that the intact strength of the high plasticity levee clays could degraded over time. The previous geotechnical investigations at the site have provided strength data of what is considered to be fully intact clay specimens as well as specimens considered to have fully softened soil strengths. This testing has indicated that the strengths of intact clays and fully softened clays are quite close. In order to capture all potential behavior in the analyses, however, fully softened strengths were used for clays considered to be desiccated.

In order to investigate the question of how the depth of desiccation affects the stability of a section, the levee sections that are made up of high plasticity clay were modeled with three depths of desiccation; 5, 10, and 15 ft into the levee surface. 5 ft was considered a typical depth of desiccation and 15 ft was considered the upper bound that the desiccation cracks could penetrate into the levee surface. Desiccation was assumed to only penetrate 10 ft into the foundation soils outside the levee footprint. As an additional measure to the sensitivity analysis some cross sections were modeled with fully desiccated levee sections.

Seepage analyses paired with subsequent stability analyses indicate that the depth of desiccation in the surface of the levees had little impact on the stability of the levee. Figure 42 shows the results of three stability analyses on section 74+00, east alignment, using depths of desiccation at

5, 10, and 15 ft. They have mean factors of safety of 2.18, 2.16, and 2.13, respectively. All the analyses used the best estimates of permeability and flood loads corresponding to 100% of the levee height. Due to the relatively low permeability of the intact high plasticity clay, the transient seepage analyses indicate that the piezometric surface will penetrate the desiccated soils completely but won't penetrate into the intact clay core of the levee. This does little to impact stability because the effective overburden stress is only reduced near the surface of the levee where the transient seepage analysis indicates excess pore pressures are present and where overburden stress was already low. This only serves to further reduce the shear strength in an area that offered little contribution to the overall stability to the levee anyway. Any failure surface that would result in a loss of the levee crest (a critical condition as it would not be able to retain a flood) passes through the core of the levee. Desiccation does not penetrate into the intact clay of the levee core so neither does the piezometric surface during a flood event. Since the pore pressures did not significantly increase, the shear strength of the levee material in this area did not change and the stability of the section remains largely unaffected.

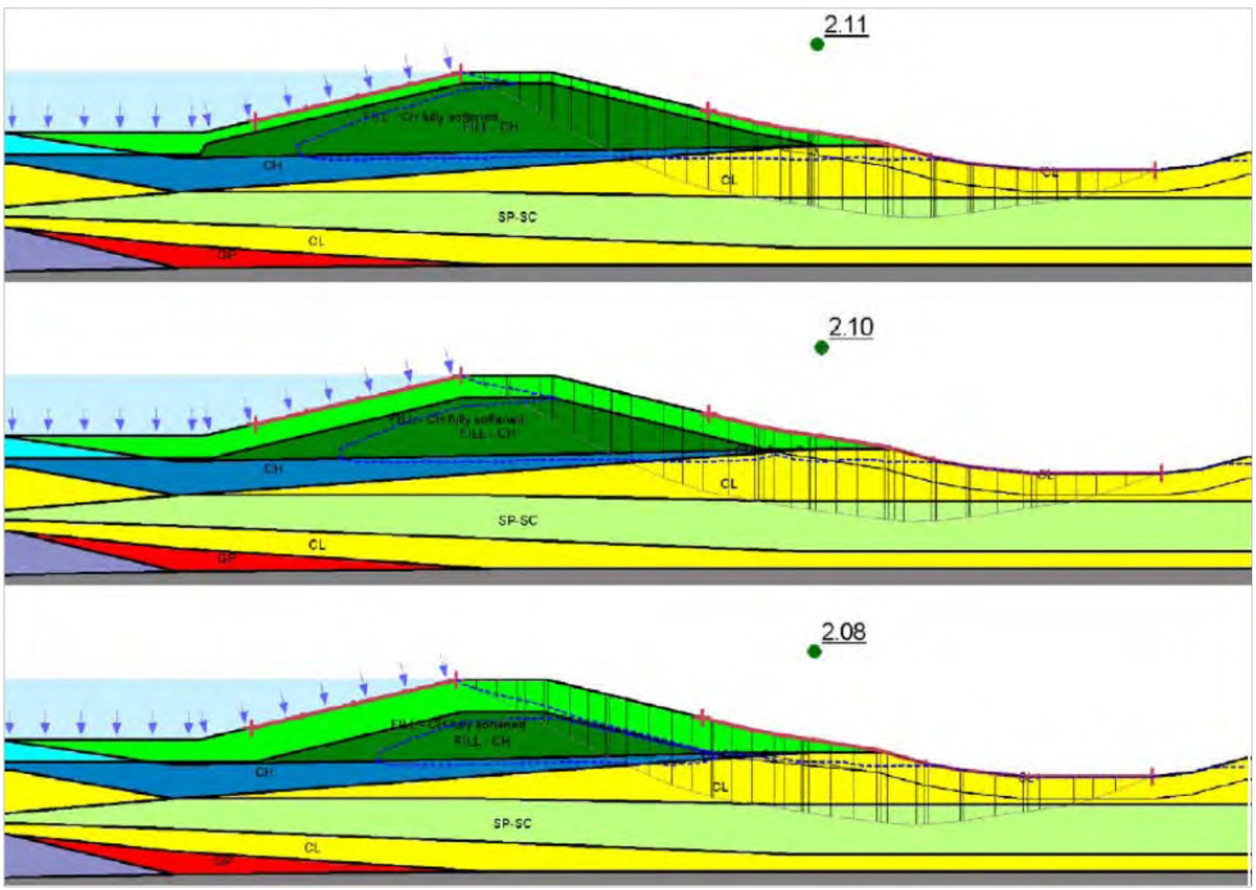


Figure 40 - Stability analysis results showing a 5-ft depth of desiccation on the top figure, 10 ft on middle figure, and 15 ft on the bottom figure. The bright green layer on the top of the levee section and the light blue at the left of the figures represent the desiccated soil.

In order to account for the possibility that the surface of the foundation soils had desiccated prior to placement of the original levees, wherever a high plasticity clay layer was present below the bottom of the levee section it was modeled as fully desiccated. For simplicity, the entire high plasticity clay layer was modeled as desiccated instead of sticking to the maximum desiccation crack penetration depth of 10 ft that was used for free field soils. Figure 43 shows the results of two stability analyses on section 74+00, east alignment, using an intact foundation material and a desiccated foundation material. The results of the seepage analyses indicate that this allowed flood waters to more easily penetrate under the levee, increasing the pore pressure and driving the effective stress down in a soil layer through which the critical failure surface passes. This is demonstrated by a decrease in the mean factor of safety from 2.18 to 1.98. The addition of a desiccated foundation layer was shown to have more of an effect on stability than varying the depth of desiccation on the levee surface, though not enough of an effect to indicate that there was a stability problem under the load of a flood height to the crest of the levee.

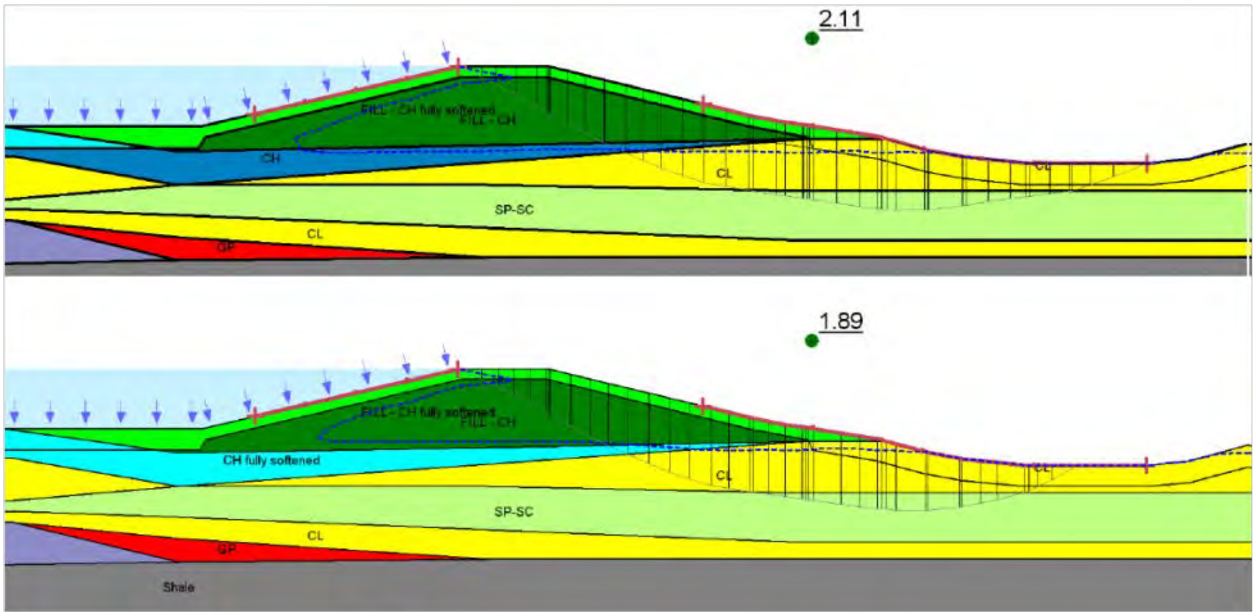


Figure 41 - Stability analysis results using a desiccated foundation layer. The desiccated material is represented by the light blue color.

Defining Pore Pressures with Ru

Near-surface stability failures, as discussed in the previous subsection, are considered to be a result of fully softened soils and desiccation. Based on the historical record, these types of failures typically occur following a heavy rain event that was immediately preceded by a prolonged dry period. It's thought that the surfaces of the levees dry out due to the lack of moisture and subsequently desiccate. Then a large rain event occurs and water infiltrates the surface of the levees relatively quickly through the desiccation cracks and saturates the surface. This reduces the effective stress and subsequently drives down the shear strength to induce a near-surface failure slide.

The risk assessment cadre wanted to ensure that this type of slide is incorporated in a potential failure mode because it's so abundant in the historical record. Therefore, it was postulated that a catastrophic failure could occur if a near-surface failure occurred immediately before or during a large flood event and there would be no opportunity to repair the slope. At that point, seepage from the flood event would more quickly infiltrate the post-slide (decreased) levee section. This would in turn result in higher pore pressures in the levee at the peak of the flood which would lead to an unstable levee section.

Since rainfall cannot specifically be replicated in SEEP/W, the pore water pressures due to a rainfall event were induced using an Ru factor in SLOPE/W. The Ru factor forces the pore water pressure to a value that is based on the overburden pressure of the soil according to the following relationship.

$$R_U = \frac{u}{\gamma_t H_S}$$

Pore water pressure is defined as u, the total unit weight of the soil is γ_t,, and the height of the soil column is H_S.

In the Dallas Floodway stability investigation, the factor was initially set to 0.5 in both the CH Fill material and the Fully Softened CH Fill material in order to model the groundwater level at or near the surface of the levee. It was assumed during the risk assessment that the factor of safety had to be close to unity for the near-surface failure to occur so further stability runs were executed while the Ru factor was incremented up or down in order to achieve a final factor of safety of one. Ultimately it was determined that an Ru of 0.46 results in a factor of safety of one. The resulting near-surface failure penetrated approximately 5 ft into the levee. Seepage analyses were then performed on the levee section accounting for the lost material due the movement of the failure mass. The best estimates of permeability for all materials were used for these analyses. Figure 44 shows the results of the two stability analyses on the section located at station 410+00 on the east alignment: the top figure shows the results of the fully intact levee at the peak of the May 1990 flood event scaled to the crest of the levee and bottom figure depicts the results of an analysis assuming a near surface slide has occurred immediately before the same flood load was applied. The figure shows that the piezometric grade line on each picture changes very little indicating there was little change in the pore water pressure induced by the flood load in the levee. This was due to the fact that near-surface failure mass did not penetrate past the desiccated zone into the low permeability core of the levee so the primary portion of the levee remained intact. As a result, the factor of safety against stability failure remained at 2.19.

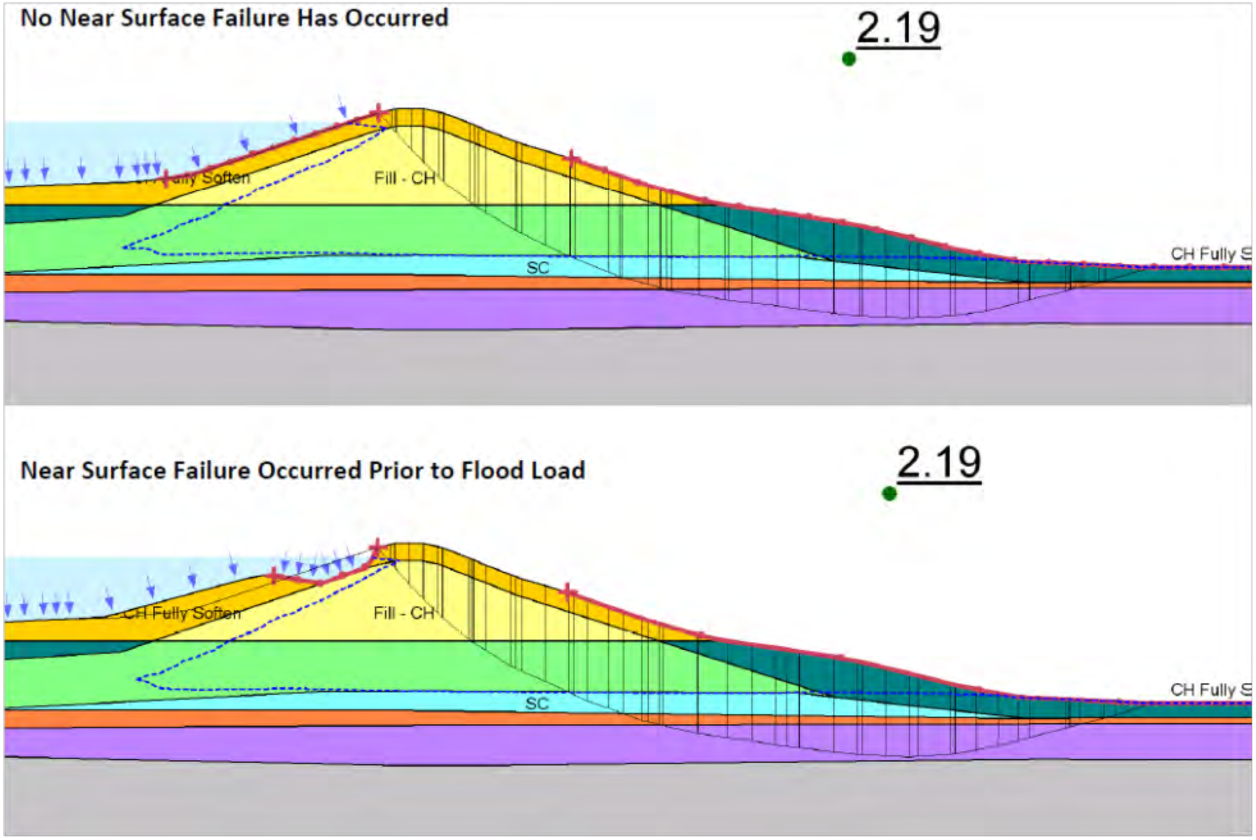


Figure 42 - Stability analyses results showing the effect of a near surface failure. Both stability runs are based on independent seepage analyses; the top is performed on the fully intact levee section, the bottom seepage analysis was performed on the levee section missing the near surface failure mass.

Tension Cracks

Tension cracks were included in the sensitivity analysis as part of investigation into the effects of desiccation. While the increased permeability of desiccated soils was captured in the modeling as described above, the actual cracks that manifest themselves on the soil surface was accounted for by introducing tension cracks in the analyses. Tension cracks were modeled to a depth of 5 and 10 ft in the surface of the levee in sections 220+00 and 410+00 on the east alignment and at 335+00 on the west alignment. Tension cracks were also used in conjunction with Ru defined pore water pressures and near-surface stability failures.

Stability analyses results indicate that the presence of desiccation or tension cracks at the surface of the section have little influence on the overall stability of the section. Figure 45 shows the results of stability analyses on the section at station 220+00 on the east alignment. The analyses resulted in mean factors of safety of 1.77, 1.74, and 1.71 for sections without tension cracks, with a 5-ft tension crack, and with a 10-ft tension crack, respectively. Due to the drained nature of the stability analysis, the shear strength of the soil relies on the effective overburden stress to develop resistance. Since the tension cracks are near the levee surface and overburden stress is low, the loss of shear resistance along a tension crack compared to the strength of the fully intact soil is nominal. This results in a minimal effect on factor of safety against stability failure as compared to a section with no tension cracks.

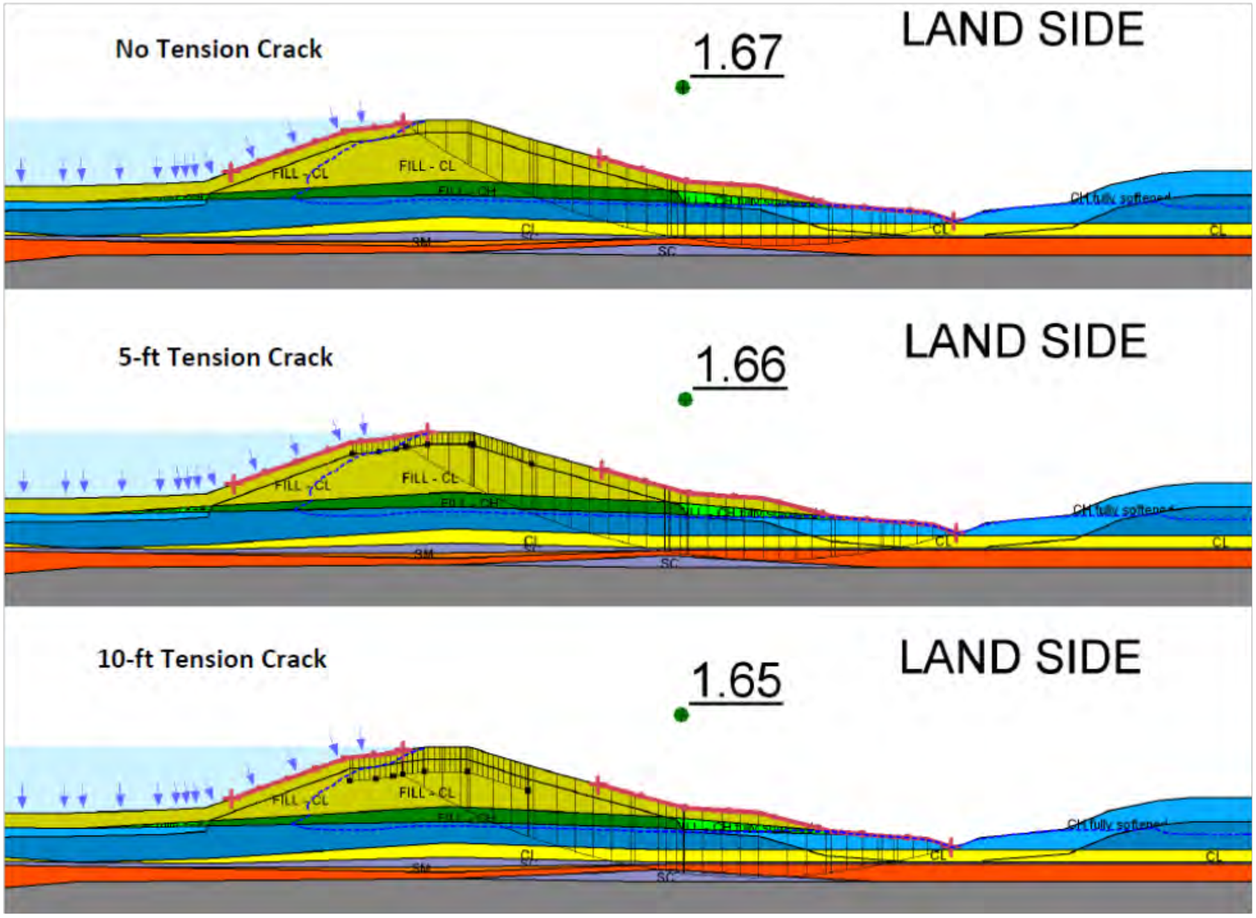


Figure 43 - Stability analysis results showing varying tension crack depths.

Low CL Permeability

Some additional sensitivity analysis of the Dallas Floodway levee system was requested by the risk assessment cadre to determine if any potential failure modes were being missed. After much of the seepage and stability analysis had been done and it had become evident that the material permeabilities had the most influence on the stability, it was suggested that a levee section with a permeable flood-side paired with an impermeable protected-side could result in an unstable condition. A situation such as this would give flood waters a path to seep into the levee but not provide a pathway for the seepage to flow out on the protected side. This would back the seepage up and cause an increase in pore water pressures in the soils below the levee, decreasing the effective stress and causing a decrease in the overall stability of the levee.

Section 74+00 was selected for this analysis because it had high plasticity clay (CH) on the flood-side that is susceptible to desiccation and low plasticity clay (CL) on the protected-side that is not susceptible to desiccation. Figure 46 and Figure 47 depict the analysis section. In order to model the worst case scenario of the problem, all of the material permeabilities in the seepage model were set at the best estimate except for the low plasticity clay (CL) which was modeled at the low estimate of permeability. The flood-side high plasticity clay was assumed to be desiccated to a depth of 37 ft. This resulted in a difference in permeability between the flood and protected sides of 4 orders of magnitude (3.6e-2 ft/hr for the desiccated CH and 8.6e-6 ft/hr for the CL). This unique levee geometry resulted in an unstable levee. In order to develop a sense of how pervasive this situation was, some further modeling was done to determine what the threshold for failure was. Additional modeling revealed that if the CL were modeled using a permeability one order of magnitude higher (8.6e-5 ft/s was used instead of 8.6e-6 ft/s), the mean factor of safety changed from 0.69 to 1.34 and the section became stable (Figure 46). Increasing the permeability an additional order of magnitude to 8.6e-4 ft/s further increased the mean factor of safety to 1.87. Furthermore, the section was modeled under a flood load that only reached 75% of the levee height at its peak elevation (instead of 100%) and the mean factor of safety changed from 0.69 to 1.3 and the section became stable (Figure 47). Decreasing the flood load to 50% of the levee crest elevation further increased the mean factor of safety to 1.99.

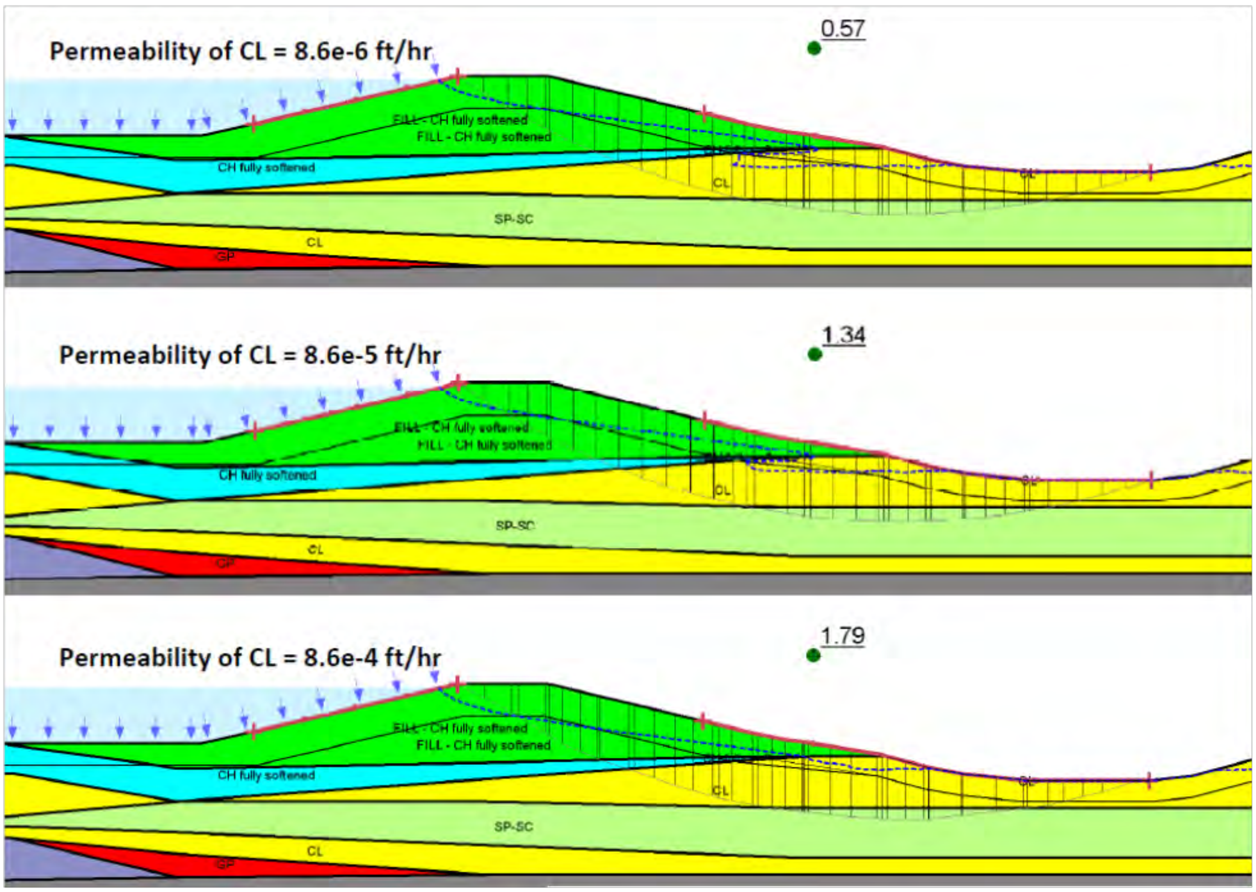


Figure 44 - Stability analysis results showing varying values of permeability for the low plasticity clay (CL) layer.

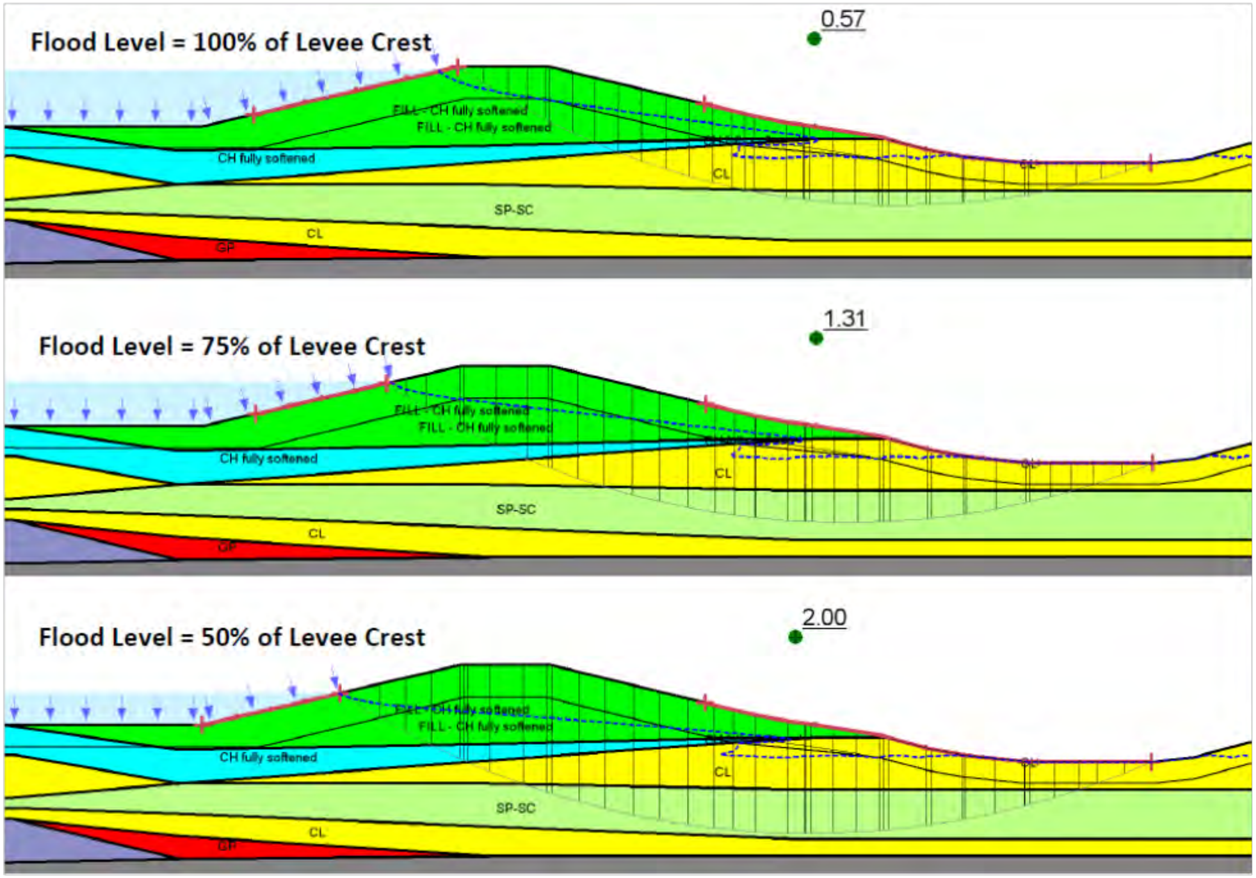


Figure 45 - Stability analysis results performed on the section at STA 74+00 using a low estimate of permeability for the CL material layer (8.6e-6 ft/hr) under the load of varying flood heights equal to 100, 75, and 50% of the levee crest elevation.

The seepage and stability sensitivity modeling indicate that several outlying events would need to occur at the same time for this particular failure mode to occur. (1) The high plasticity clay on the flood-side of the levee would need to be desiccated to 37 ft despite the fact that a significant portion of the desiccated clay would reside below the normal water table, (2) permeability of the low plasticity clay (CL) would have to correspond to the low estimate in the analysis, despite the fact that the estimate is based on outlying laboratory test results that are considered to be outside of the typical permeability of a low plasticity clay, and (3) a flood would need to reach the full height of the levee, an event for which the frequency is very low. Based on the assessment of the risk of each of these factors actually occurring, it was decided that the chance of this unique levee geometry actually existing was extremely remote.

Blocking the seepage entrance and modeling SP as CH

As an additional means of understanding how the Dallas Floodway Levee System behaves under varying conditions, the risk assessment cadre wanted to see how the stability of a marginally stable levee section with a desiccated foundation would change if few parts of the foundation geometry were modified. Figure 48 depicts the results of these analyses. Section 220+00 on the east alignment was selected as the analysis section because it is the critical section for several failure modes and the levee is made from low plasticity clay which is not subject to desiccation. This has the benefit of seeing how much a fully-desiccated foundation layer influences the stability of a section if there are no other high permeability zones (such as a continuous sand layer) to conduct seepage through.

Section 220+00 modeled with the high estimate of material permeabilities and a fully desiccated foundation layer was selected as the base case of the problem. This section has a mean factor of safety of 1.16 against stability failure. In order to give greater influence to the desiccated layer, first the primary seepage entrance to the foundation sand was blocked. This was done by replacing the first 180 ft of the 5-ft thick poorly graded sand (SP) with a fully-intact high plasticity clay (CH), eliminating the open pathway to the primary river channel. This marginally increased the mean factor of safety from 1.16 to 1.18. It appears that the increase was only slight because seepage could still penetrate through the desiccated foundation layer that overlies the sand layer in the free-field area between the river thalweg and the flood-side toe of the levee. To further modify the problem, the subsurface sand layers were made less permeable. This was done by modeling all the sand layers in the section as fully-intact high plasticity clay. This significantly improved the stability of the section to a mean factor of safety of 1.66. The primary difference between the first two iterations (base case and blocked entrance) and the last was that seepage pressures could still stack up under the levee in the sand layers in the first two runs, reducing the effective stress and subsequently reducing the available shear resistance. However, when the sand layers were replaced with CH material, seepage pressures could not build up to the same extent (total head in the sand layers decreased by 10 ft) and more shear resistance became available. This further supports the finding that material permeability has the largest influence on stability.

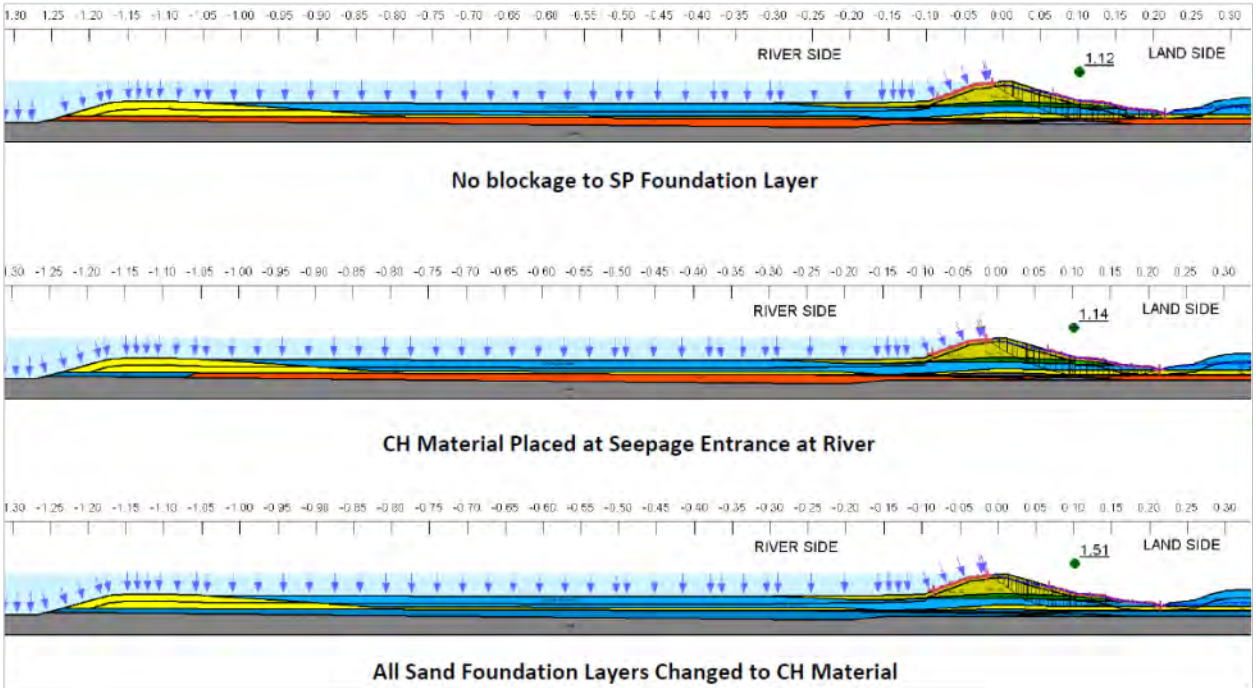


Figure 46 - Stability analysis results showing varying blockages to seepage in the foundation of the section.

1990 Hydrograph, Full Levee

Case	Parameter	74+00 E			220+00 E			220+00 E Desiccated			311+00 E			250+00 W			335+00 W			410+00 E			410+00 E Desiccated			188+00 W		
Best k, Best Str	Mean FoS		1.89			1.66			1.43			2.86			2.37			1.86			2.35			1.57			2.50	
	Uplift FoS, PS Toe; Elev	1.00		367	0.465		382	0.465		382	N/A		396.5	N/A		400	1.377		368	1.85		387.31	0.7152		387.83	-		-
	Gradient across levee		0.0195			0.0323			0.0381			0.1154			0.0919			0.0096			0.0011			0.0507			0.0321	
Low k, Prob Str	Mean FoS; P(failure); Min FoS	2.19	0.00%	1.65	1.84	0.00%	1.43	1.87	0.00%	1.44	2.87	0.00%	1.89	2.71	0.00%	2.03	2.04	0.00%	1.47	2.0555	-	1.6853	1.798	-	1.4019	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev	1.157		367	0.468		382	0.493		382	N/A		396.5	N/A		400	1.291		368	1.9305		387.82	1.8884		387.82	-		-
	Gradient across levee		0.0039			0.02			0.0142			0.1182			0.0085			0.0219			0.0145			0.0269			0.0087	
Best k, Prob Str	Mean FoS; P(failure); Min FoS	2.18	0.00%	1.58	1.77	0.00%	1.31	1.59	0.00%	1.16	2.63	0.00%	1.84	2.38	0.00%	1.98	1.7	0.00%	1.21	2.3006	-	1.6524	1.5837	-	1.2016	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev	1.063		367	0.511		382	0.484		382	N/A		396.5	N/A		400	1.323		368	1.3977		387.82	0.678		387.82	-		-
	Gradient across levee		0.0057			0.0404			0.0412			0.1151			0.0923			0.0114			0.0082			0.0499			0.0309	
High k, Prob Str	Mean FoS; P(failure); Min FoS	1.91	0.00%	1.48	1.24	1.67%	0.82	1.16	11.25%	0.7	2.35	0.00%	1.58	2.24	0.00%	1.87	1.56	0.00%	1.12	1.879	-	1.528	1.6142	-	1.2348	2.1495	-	1.6145
	Uplift FoS, PS Toe; Elev	1.01		367	0.434		382	0.411		382	N/A		396.5	N/A		400	1.57		368	1.0051		387.83	0.8441		387.83	3.069		396.64
	Gradient across levee		0.0348			0.0381			0.0358			0.1209			0.1201			0.0684			0.0424			0.0665			0.0952	

1990 Hydrograph, 75% Full Levee

Case	Parameter	74+00 E			220+00 E			220+00 E Desiccated			311+00 E			250+00 W			335+00 W			410+00 E						188+00 W		
Best k, Best Str	Mean FoS		2.22			1.84			1.71						2.43						2.35			1.67			2.50	
	Uplift FoS, PS Toe; Elev	1.302		367	0.521		382	0.521		382				N/A		400				1.8615		387.31	0.7699		387.83	-		-
	Gradient across levee		0.0195			0.03			0.0331						0.0791						0.0010			0.0440			0.0283	
Low k, Prob Str	Mean FoS; P(failure); Min FoS				2.05	0.00%	1.71							2.71	0.00%	2.03				2.0907	-	1.7163	1.8948	-	1.512	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev				0.543		382							N/A		400				1.9374		387.82	1.8993		387.82	-		-
	Gradient across levee					0.0215									0.0085						0.0124			0.0228			0.0072	
Best k, Prob Str	Mean FoS; P(failure); Min FoS				1.89	0.00%	1.56							2.42	0	2.02				2.3049	-	1.6568	1.6786	-	1.2876	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev				0.566		382							N/A		400				1.4556		387.82	0.7322		387.82	-		-
	Gradient across levee					0.0346									0.0735						0.0070			0.0433			0.0274	
High k, Prob Str	Mean FoS; P(failure); Min FoS				1.56	0.00%	1.15													1.9517	-	1.6023	1.7257	-	1.2763	2.3129	-	1.7683
	Uplift FoS, PS Toe; Elev				0.468		382													1.058		387.83	0.8964		387.83	4.4162		396.64
	Gradient across levee					0.0315															0.0368			0.0577			0.0834	

1990 Hydrograph, 50% Full Levee

Case	Parameter	74+00 E			220+00 E			220+00 E Desiccated			311+00 E			250+00 W			335+00 W			410+00 E						188+00 W		
Best k, Best Str	Mean FoS		2.11			1.95			1.85						2.49						2.36			1.79			2.50	
	Uplift FoS, PS Toe; Elev	1.132		367	0.592		382	0.6		382				N/A		400				1.8771		387.31	0.8527		387.83	-		-
	Gradient across levee		0.0148			0.023			0.025						0.062						0.0008			0.0358			0.0198	
Low k, Prob Str	Mean FoS; P(failure); Min FoS				2.08	0	1.76							2.71	0.00%	2.03				2.3049	-	1.6568	2.128	-	1.336	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev				0.62		382							N/A		400				1.9474		387.82	1.915		387.82	-		-
	Gradient across levee					0.0162									0.0085						0.0098			0.0171			0.0043	
Best k, Prob Str	Mean FoS; P(failure); Min FoS				2.01	0	1.68							2.5	0	2.06				2.3049	-	1.6568	1.8254	-	1.3944	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev				0.651		382							N/A		400				1.4633		387.82	0.8149		387.82	-		-
	Gradient across levee					0.0268									0.0585						0.0054			0.0353			0.0199	
High k, Prob Str	Mean FoS; P(failure); Min FoS				1.82	0	1.52													2.0109	-	1.6508	1.8021	-	1.3804	2.3839	-	1.7707
	Uplift FoS, PS Toe; Elev				0.534		382													1.1334		387.83	0.9742		387.83	-		-
	Gradient across levee					0.0238															0.0301			0.0472			0.0474	

Appendix D – Hydrology and Hydraulics

Objectives

The objective of the study was to provide hydrologic information relative to the risk assessment for the Dallas Floodway project. The data includes estimates of flood duration, volume-frequency, discharge-frequency, and unsteady flow modeling for estimating overtopping and consequences. As the data is being used for a risk assessment, estimates of best values were calculated.

Background

The drainage area of the Trinity River, from its headwaters to the confluence of Five Mile Creek, near the Interstate Highway 20 bridge in south Dallas, was evaluated during this study. This area, which is commonly referred to as the "Upper Trinity" watershed, covers about 6,275 square miles. It includes the majority of the Dallas-Fort Worth (DFW) Metropolis. Terrain in this watershed varies in elevation from about 1,200 feet National Geodetic Vertical Datum (NGVD) at the headwaters of the West Fork of the Trinity River just northeast of Olney, Texas, to about 380 feet NGVD at the confluence of Five Mile Creek. A general watershed map is included as Figure 49 - Trinity River Sub-Basins and Reservoirs.

Of the five US Army Corps of Engineers (USACE) flood control reservoirs in the study area, three (Lakes Benbrook, Lewisville, and Grapevine) were impounded in the early 1950's. Impoundments in the other two USACE reservoirs (Lakes Joe Pool and Ray Roberts) were initiated in January 1986 and June 1987, respectively. Additional major USACE flood control projects in the study area include the Fort Worth Floodway and Dallas Floodway levee/channel improvement systems.

The two largest non-Federal lakes in the study area, both of which are situated on the West Fork of the Trinity River, are Lake Bridgeport and Eagle Mountain Lake. Lake Bridgeport is located just west of Bridgeport in Wise County. Eagle Mountain Lake is located in northwestern Tarrant County, just upstream from the much smaller Lake Worth, which is owned by the City of Fort Worth. Eagle Mountain Lake has two sets of outlet gates and an emergency spillway, but since it has no dedicated flood control storage, large releases are required during flooding periods. Smaller lakes within the Upper Trinity watershed include: Lake Amon Carter, located on Big Sandy Creek south of Bowie in southwestern Montague County; Lake Weatherford, located on the Clear Fork of the Trinity River northeast of Weatherford in Parker County; Lake Arlington, located on Village Creek in western Arlington in Tarrant County; and Mountain Creek Lake, located on its namesake in Grand Prairie in western Dallas County. Table 19 contains the completion dates, normal storage and NLD ID for the reservoirs in the basin.

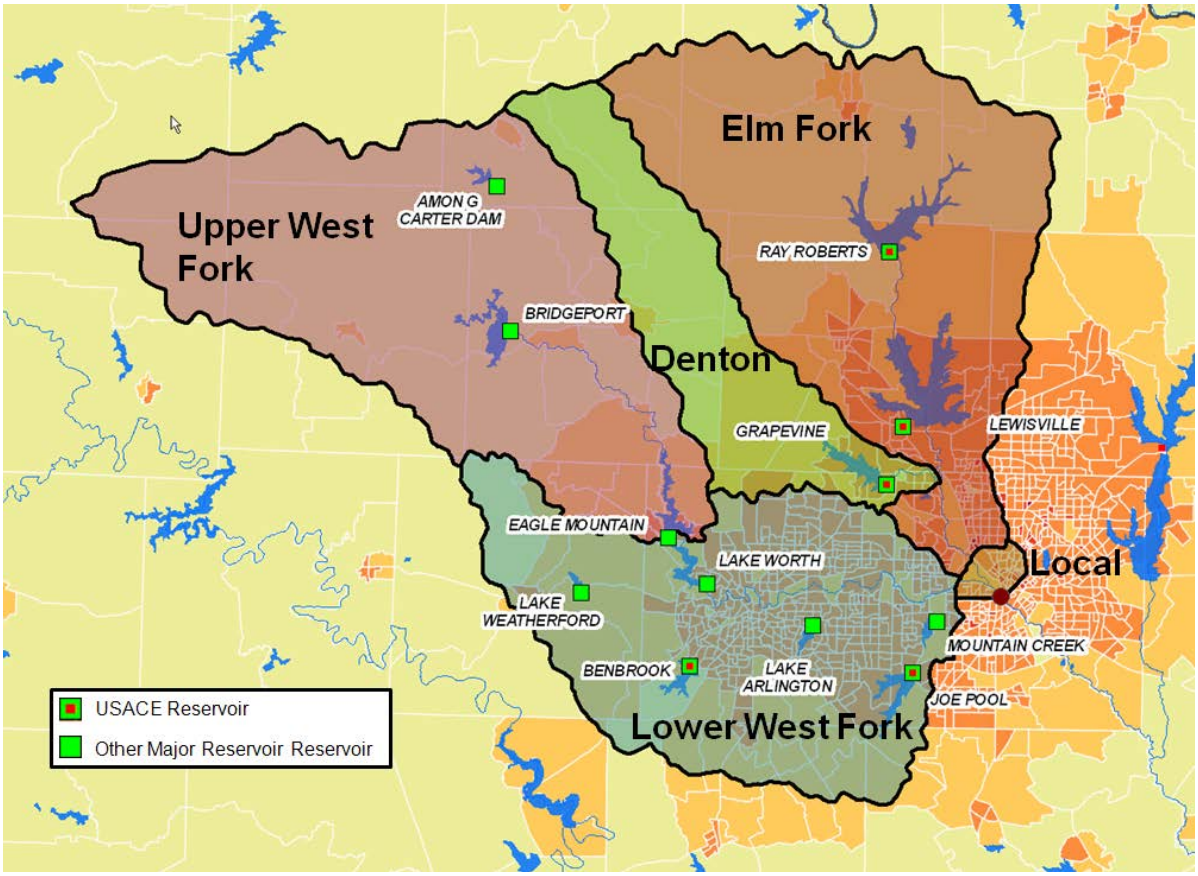


Figure 47 - Trinity River Sub-Basins and Reservoirs

Table 17 - Information on Dam Construction Completion

Reservoir	Year Completed	Normal Storage (Acre-ft)	NLD ID
Lake Worth	1914	38130	TX00785
Bridgeport	1931	386539	TX01496
Eagle mountain	1932	190460	TX00779
Mountain Creek	1937	40000	TX00827
Benbrook	1951	88250	TX00003
Grapevine	1952	188550	TX00005
Lake Arlington	1955	38785	TX00776
Lewisville	1955	618400	TX00008
Lake Amon Carter	1956	20050	TX00699
Lake Weatherford	1957	19866	TX01222
Joe Pool	1986	176900	TX08009
Ray Roberts	1987	799600	TX08010

Climatology

The climate in the Upper Trinity watershed is humid subtropical with hot summers and mild winters. Snowfall and subfreezing temperatures are experienced occasionally during the winter season. Generally, the winter temperatures are mild with occasional cold periods of short duration resulting from the rapid movement of cold pressure air masses from the Northwestern polar regions and the continental western highlands.

Recorded temperatures at the DFW International Airport have ranged from a high of 113°F in June 1980 to a low of -1°F in December 1989. The average annual temperature over the watershed varies from 64°F at Bridgeport in the northwestern extremity of the watershed to 66°F at DFW International Airport. The mean annual relative humidity for the DFW Metropolis is about 65 percent. The average annual precipitation over the watershed varies from about 30 inches at Jacksboro, in the northwestern extremity of the watershed, to about 32 inches in the DFW Metropolis. The extreme annual precipitation amounts since 1887 include a maximum of 53.54 inches in 1991 at the DFW International Airport and a minimum of 17.91 inches in 1921 at Fort Worth. The maximum recorded precipitation in a 24 hour period was 9.57 inches, at Fort Worth on the 4th and 5th of September 1932. A large part of the annual precipitation results from thunderstorm activity, with occasional very heavy rainfall over brief periods of time. Thunderstorms occur throughout the year, but are more frequent in the late spring and early summer.

The average length of the warm season (freeze-free period) in the DFW Metropolis is about 249 days, extending from mid-March to mid-November.

Hydrologic Frequency Curve Development

Available Data

Data for this analysis were obtained from the USGS Trinity River at Dallas (Commerce Street Gage) USGS Streamflow gage #08057000 which has been recording discharge data from 1 Oct 1903 until present. Data used were the daily discharge and the peak annual discharge which with a few exceptions provide a full record from the gage installation date. The daily discharge value is an average of the flow during the entire day, which is always less than the peak daily discharge. Since the 2002 water year (begins 01 Oct 2001), USGS has recorded the daily mean, minimum and maximum discharge. Peak annual discharge is the highest discharge recorded during the water year (01 Oct through 30 September). It is important to note that the USGS method for peak discharges can produce 2 peak discharges in one calendar year since this time period spans two water years.

Data from the gage web site was retrieved via a routine in the program HEC-SSP (referred to as SSP) which is the Hydrologic Engineering Center Statistical Software Package dated October 2010. After a review of the raw data obtained by SSP, it was found that daily discharge values are absent from the following dates found below in Table 20. The summation of missing data results in 1067 days without record. Based on this, the daily discharge data from before 1920 were excluded in the analysis (i.e. only 1 Jan 1920 to present data was used). Annual peak data was complete, but on initial import contained peak flows from the “At Dallas” gage (correct gage) prior to 1957, and the “Below Dallas” (USGS gage # 08057410) for years 1957 to present. Once found, the errant record from the “Below Dallas” gage was removed and replaced with the data shown on the USGS website for the “At Dallas” gage prior to analysis.

Table 18 - Missing Daily Discharge

Start Date	End Date	Days Missing	Notes
3/2/1919	6/1/1919	91	
3/2/1918	11/11/1918	254	Missed peak event
4/2/1917	5/1/1917	29	
8/2/1916	1/1/1917	152	
8/2/1914	10/1/1914	60	
5/2/1913	7/1/1913	60	Missed peak event
12/2/1912	1/1/1913	30	
8/2/1912	11/1/1912	91	
6/2/1912	7/1/1912	29	
8/2/1911	12/1/1911	121	Missed peak event
10/2/1910	3/1/1911	150	

Previous Frequency Analysis

A previous frequency analysis was completed by the Ft. Worth District using 40 years of gage data for the period of record 1953 to 1992. This data was supplemented with design rainfall estimates modeled with a calibrated HEC-1 mode. More information on the HEC-1 model, its calibration, and the discharge frequency curve can be found in the General Re-evaluation Report (GRR) Appendix A. The final curve recommended in the report combined the analytical data with the HEC-1 model results. A comparison of the final analytical curve with the HEC-1 modeling points is shown in Figure 50. The final adopted skew for the analytical curve was 0.0982. Table 21 shows the discharge values for specific frequency points.

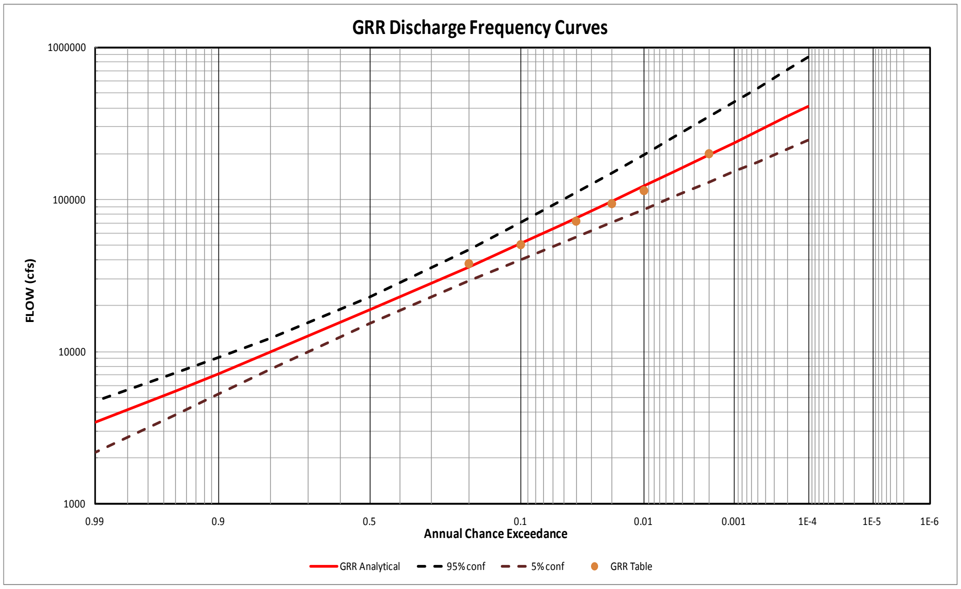


Figure 48. GRR Discharge Frequency Curves

Table 19. GRR Discharge Frequency Values are shown highlighted

Location along the Trinity River	Computed Probability Peak Discharges (cfs) for:								
	Recurrence Interval (years)								SPF
	1	2	5	10	25	50	100	500	Event
	Annual Exceedance Probability (percent)								
	NA	50	20	10	4	2	1	0.2	NA
Below the confluence of the West and Elm Forks	18300	24500	38700	51500	73400	95100	115800	202700	270100
At the "Dallas" Streamflow Gage	18000	24100	38100	50800	72500	94600	115200	201400	269200
Above the confluence of White Rock Creek	14100	20900	35200	48400	69100	90200	111800	188500	251100

Regulated and Unregulated Conditions.

Based on Table 19, the majority of dams in the Trinity River system were completed by the late 1950's. While a few dams were completed prior to the early 1950s, it was assumed their location and storage volume would have negligible impact on the study area hydrographs. From Figure 49, these earlier reservoirs are located on the upper West Fork of the Trinity River, and in the case of Bridgeport, only control a small portion of the Trinity River basin drainage area. Therefore, the decision was made to use the 1904-1951 record of peak discharges as an analog "unregulated" period and discharges from 1956-2011 as "regulated" discharges. Years between 1951 and 1956 were excluded due to the completion of two major projects (Grapevine and Lewisville) and three smaller projects (Benbrook, Amon Carter and Arlington).

It should be noted that within the "regulated" period, there were changes to project standard operating procedures, as well as new reservoirs coming online. Most notably, Joe Pool and Ray Roberts were completed in 1896 and 1987 respectively. While Ray Roberts is considerably larger, it is upstream of Lewisville, and regulated flows from Lewisville were assumed to have a greater impact on flood hydrographs than impacts from Ray Roberts. Joe Pool is much closer to the USGS Trinity River at Dallas gage, but is modestly sized at 177,000 acre-ft of storage. Had the "regulated" period of record been truncated to 1988-present, only 23 years of data would be available for the analysis. For this report, frequency data based on "unregulated" flows are from 1904-1951, and "regulated" flows are from 1956-2011. Volume frequency data "unregulated" flows are from 1920-1951 due to the aforementioned data gaps, and "regulated" flows are 1956-2011.

Volume-Frequency

Volume frequency analysis takes the maximum mean daily flows over the analysis period, and relates those discharge values to the frequency of their occurrence. Separate analyses were completed for calendar year and water year. Calendar year analysis was chosen to reduce the number of occurrences which spanned one period to the next. While volume frequency analysis for the Period of Record (1920-2011), 1920-1951 and 1956-2011 were computed, only the "regulated" period of 1956-2011 was used in the risk assessment. In each of the plots, the events are plotted using Weibull plotting positions, and trendlines are created using calculated statistics.

Statistical smoothing of the volume-frequency curves was done to prevent crossing of volume-frequency curves. Smoothing was done by plotting the calculated standard deviation for each of the duration periods against the logarithmic mean of the flow values. A trend equation of all data points is then taken, and a smoothed standard deviation is created based on the trend relationship. In addition the computed skews were replaced with an adopted skew for the data. The smoothed standard deviation and adopted skew was then input into HEC-SSP, and revised plots calculated as shown below in Figure 51. The final statistics for the frequency curves is shown in Table 22.

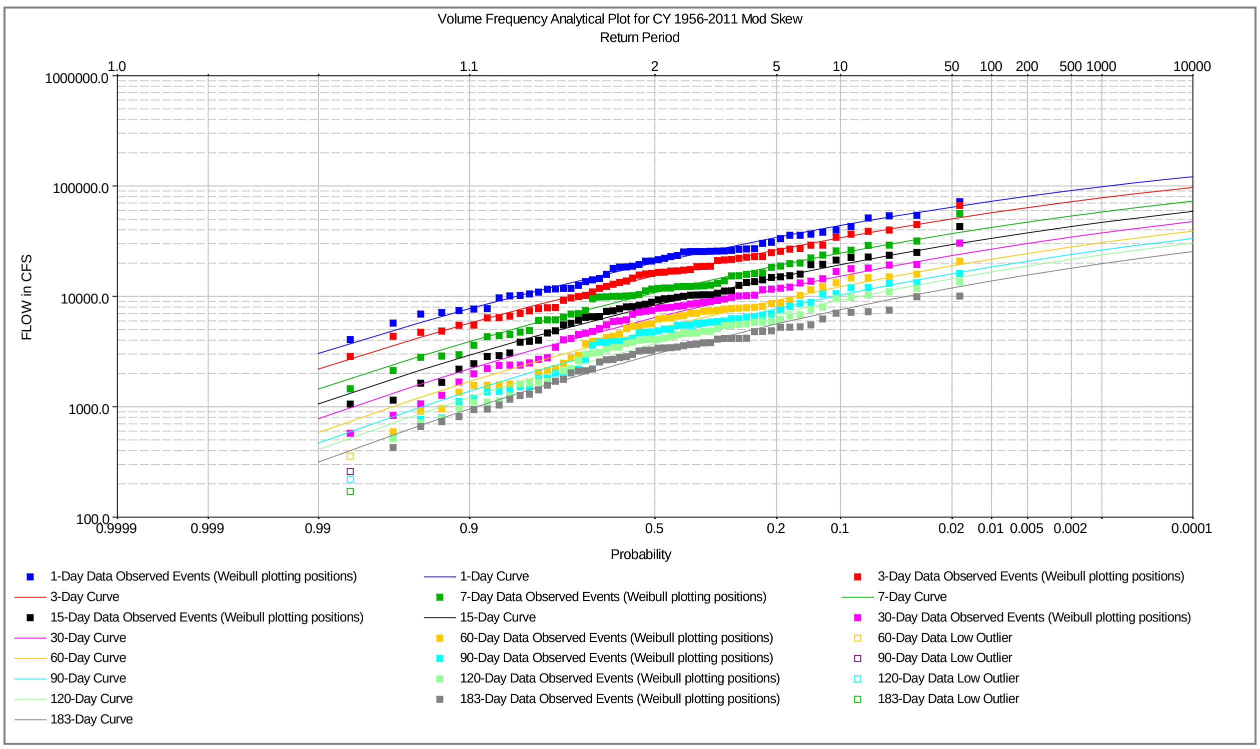


Figure 49. Smoothed Volume Frequency Analytical plot, 1956-2011

Table 20. Computed Volume Frequency Statistics

Percent Chance Exceedance	1	3	7	15	30	60	90	120	183
0.5	80,659	63,663	47,105	37,643	30,131	24,451	20,853	18,823	15,614
1	72,623	57,155	42,135	33,578	26,805	21,696	18,469	16,649	13,779
2	64,321	50,453	37,036	29,422	23,414	18,895	16,049	14,447	11,925
5	52,893	41,267	30,086	23,779	18,829	15,122	12,798	11,493	9,447
10	43,833	34,022	24,641	19,380	15,272	12,209	10,297	9,227	7,553
20	34,279	26,427	18,976	14,829	11,612	9,228	7,748	6,922	5,636

Percent Chance Exceedance	1	3	7	15	30	60	90	120	183
50	20,201	15,349	10,819	8,339	6,442	5,053	4,202	3,731	3,003
80	10,984	8,207	5,663	4,296	3,267	2,525	2,076	1,830	1,453
90	7,725	5,717	3,896	2,929	2,207	1,691	1,382	1,213	956
95	5,675	4,164	2,807	2,093	1,565	1,190	967	846	662
99	3,043	2,195	1,448	1,062	782	585	470	408	315
Mean	4.281	4.161	4.008	3.894	3.782	3.675	3.595	3.543	3.448
Standard Dev.	0.271	0.29	0.317	0.346	0.368	0.357	0.35	0.332	0.31
Station Skew	-0.26	-0.211	-0.418	-0.592	-0.749	-0.563	-0.561	-0.592	-0.595
Adopted Skew	-0.26	-0.211	-0.418	-0.592	-0.749	-0.563	-0.561	-0.592	-0.595
# Years	56	56	56	56	56	56	56	56	56
User Statistics	1	3	7	15	30	60	90	120	183
Adj. Mean	4.281	4.161	4.008	3.894	3.782	3.675	3.595	3.543	3.448
Adj. Std. Dev.	0.297	0.305	0.315	0.323	0.331	0.338	0.344	0.347	0.354
Adj. Skew	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Adj. # Years	56	56	56	56	56	56	56	56	56

Discharge-Frequency

As described in the section Regulated and Unregulated Conditions, the discharge-frequency calculation primarily used the regulated period between 1956 and 2011. When the regulated and unregulated data are shown on the same plot there is a separation between the data. In theory, the upstream dams and their regulation should decrease as the dam's storage fills and spillways begin to discharge flood flows. This would lead to the curve approaching unregulated points at infrequent events. A comparison of frequency curves for the various periods using computed statistics is shown in Figure 52. Note these curves were shown only for information purposes and were not used in the study.

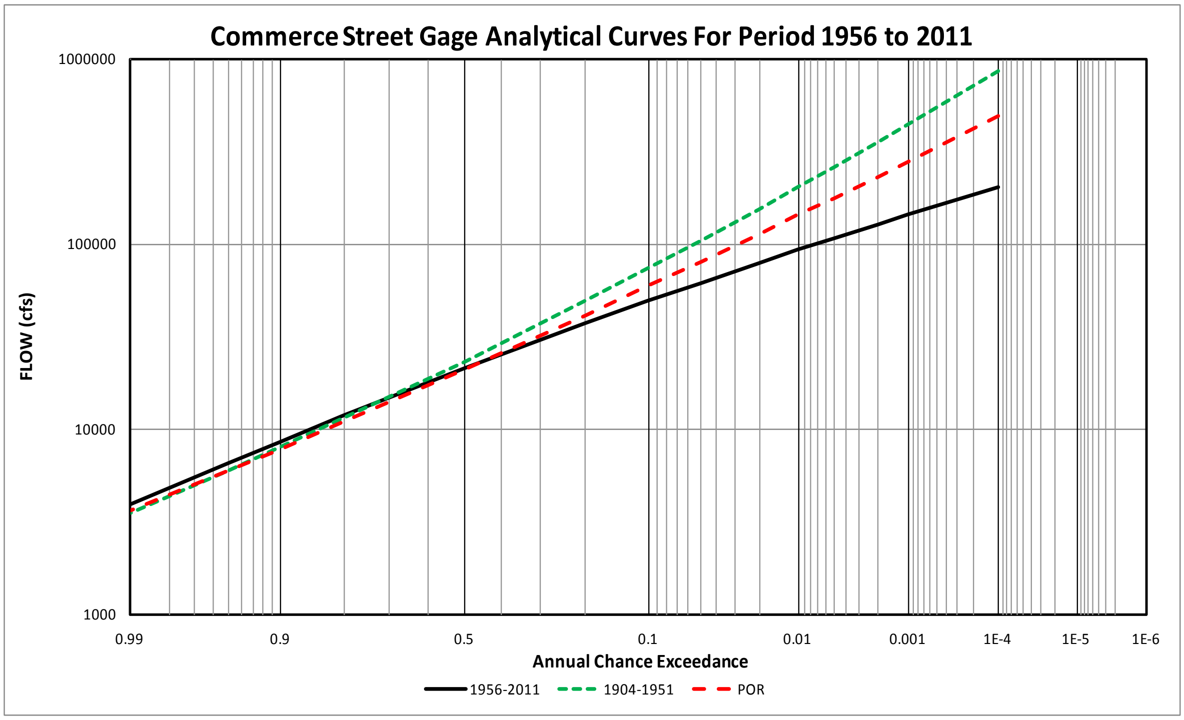


Figure 50. Discharge Frequency For Various Periods (Period of Record is POR)

When the observed events between 1904-1951 and 1956-2011 are plotted on the same chart, there is clear separation between the magnitude, and also with the computed skew in the analytical curves. (While the unregulated events have a positive skew of 0.18, and curves upwards, the regulated events have a skew of -0.18, and a corresponding downward curve.

From this information and the GRR frequency curve, a best estimate of the discharge frequency curve was derived. Using the regulated points from 1956-2011 and a user adjusted skew of 0.2 to match an expected skew of the unregulated values, a best estimate frequency curve was calculated. This curve plots slightly to the right of the GRR frequency curve but well within its uncertainty range. Values computed from this method are presented below in Figure 53 as “1956-2011 with 0.2 skew”. Table 23 contains the calculated flows from the best estimate frequency curve as compared to SWF’s General Reinvestigation Report (GRR).

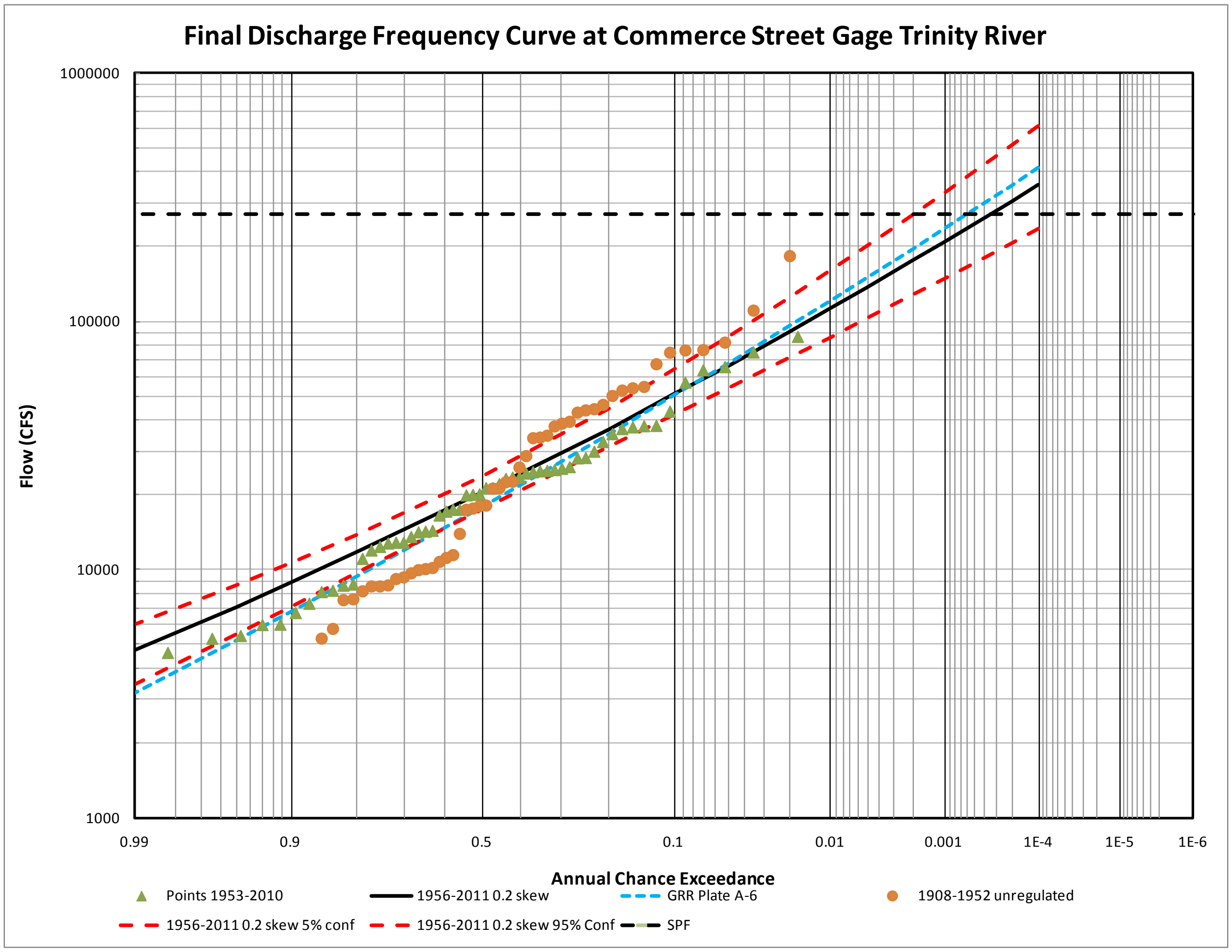


Figure 51. Unregulated, Regulated, and final graphical curves

Table 21. Best Estimate Frequency Curve Flows & Statistics compared to GRR

Annual Chance Exceedance	Recurrence	Final Graphical Flows (cfs)	GRR Flows (cfs)
0.0001	(1/10000)	353,365	
0.0002	(1/5000)	303,761	
0.0005	(1/2000)	245,987	
0.001	(1/1000)	208,314	
0.002	(1/500)	175,065	201,400
0.005	(1/200)	137,143	
0.01	(1/100)	112,492	115200
0.02	(1/50)	90,902	94,600
0.04	(1/25)	72,025	72,500
0.1	(1/10)	50,666	50,800
0.2	(1/5)	36,773	38,100
0.5	(1/2)	20,421	24,100
Mean	4.32		
Standard Deviation	0.296		
Station Skew	-0.179		
Adopted Skew	0.2		

Table 23 clearly illustrates that the GRR and the Final Graphical curves are very comparable for frequencies up to the 1% annual chance exceedance (ACE). However, since the GRR values were calculated in the mid 1990s, an additional 15 years of gage record has occurred, and has been utilized in the discharge frequency calculations. This extends the 40 years of record that the GRR results used to 55 years of record, roughly 33% longer. A longer record period provides a better estimation of the overall frequency curve but there is still a high degree of uncertainty in events larger than the 0.01 ACE. The GRR values are well within the 95% and 5% confidence bands calculated by SSP.

It should be noted this frequency curve contains significant judgment as there is a lack of unregulated points in the gage record to represent infrequent events. As it stands, the 1908 flood of record plots as the highest calculated unregulated frequency point at 0.02 ACE. Values for a flood less frequent would be needed to better estimate the upper end of the frequency curve.

Several methods could be utilized to estimate these values but were beyond the time limitations of this study. A model using infrequent rainfall estimates to calculate the unregulated and regulated points would be needed to better define the upper end of the frequency curve.

Hydrograph Shape Analysis & Rainfall – Runoff Modeling

Purpose and Scope of Rainfall – Runoff Modeling

The risk assessment of the levee system required an estimate of the probability of a failure for a variety of potential failure modes including several geotechnical/geologic failure modes, which was done through an expert elicitation process. In many cases, the expert elicitation was informed by transient seepage analysis described in Appendix B – Seepage Analysis in this report. The transient seepage analysis is sensitive to the duration of flood loading, i.e. the shape of the flood hydrographs. Generally, for the Dallas Floodway System, seepage conditions may become a concern high flood loads were to persist for several weeks.

The flood hydrograph shapes are a primarily a function of the upstream watershed conditions (shape, soils, land use, reservoir regulation) and regional weather patterns. Examination of the historic hydrograph shapes from the gage data did not show any examples of large floods that persisted near the peak for more than a week. However, there is the possibility that the historic record does not include an extreme storm that would produce a hydrograph shape that differs from the historical record.

To gain insight into the variability of the hydrograph shapes in the event of an extreme rainfall event, the isohyetal rainfall patterns for two very large regional storm events were applied (i.e. transposed) to the existing (HEC-1) rainfall runoff model of the upstream basin and resulting hydrographs at Dallas were produced. The runoff model has been previously calibrated by the Fort Worth District (References 1 and 2) and all model parameters used for the computation of the Standard Project Flood were used for the current analysis.

Observed Hydrographs

As stated in the Hydrologic Frequency Curve Development section above, discharges at the Dallas Gage since 1903. The hydrograph shapes for large observed floods give a strong indication of the hydrograph shape that would be expected during future/hypothetical major floods on the Trinity River at Dallas. A plot of some of the largest observed floods at the gage is shown below in Figure 54.

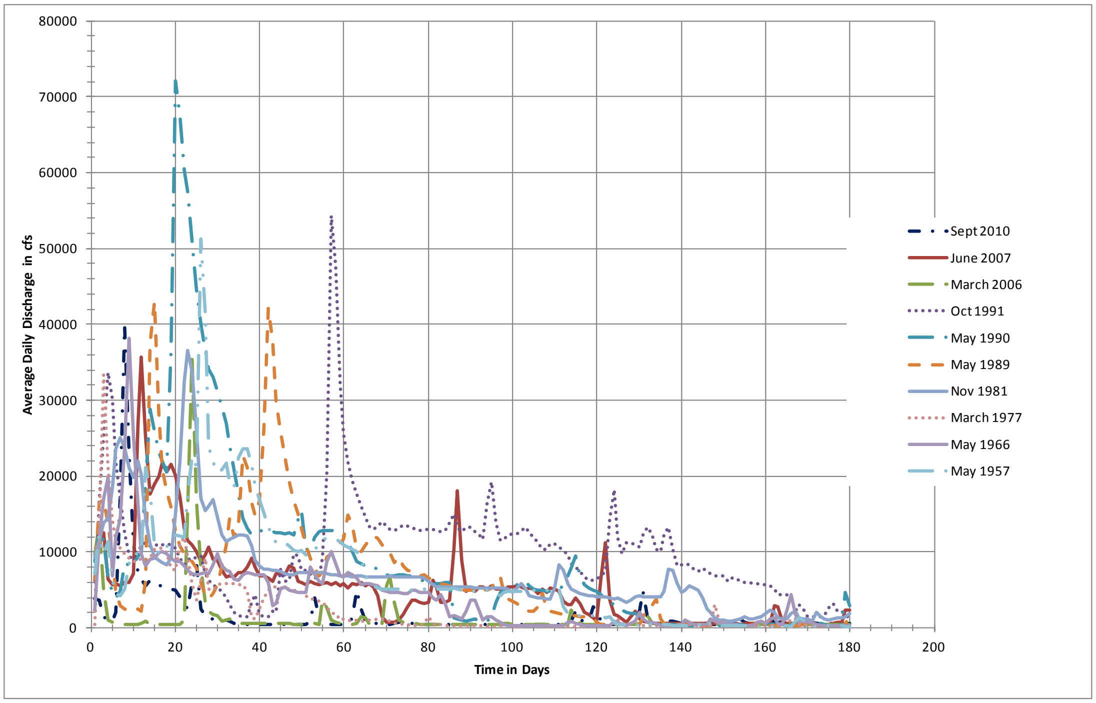


Figure 52. Observed Flood hydrographs at the Dallas Gage.

Modeled Hydrographs (HEC-1)

Previous Frequency Rainfall and Standard Project Flood Analysis

As described in References 1 and 2, an HEC-1 model was calibrated and verified for existing basin conditions and has been used for the determination of flood frequencies throughout the basin as well as for the determination of the Standard Project Flood (SPF). This model gives an indication if possible shapes of hydrographs for these standardized, hypothetical extreme storms (0.01 ACE, 0.002 ACE, and the SPF). The shape of these hydrograph is shown in Figure 57 below.

Historic Storms

The isohyetal rainfall patterns for two very large regional storm events were applied (i.e. transposed) to the existing (HEC-1) rainfall runoff model of the upstream basin and resulting hydrographs at Dallas were produced. The runoff model has been previously calibrated by the Fort Worth District (References 1 and 2) and all model parameters used for the computation of the Standard Project Flood were used for the current analysis. In regards of starting reservoir elevations, the following is an excerpt from Reference 2:

> *Each reservoir having flood control storage was assumed to be at conservation pool level at the start of the hypothetical, frequency related storms/floods and at a level corresponding to that at which one-third of the full flood control pool (except at Lewisville Lake which was started at 89 percent full) would already be occupied at the start of the USACE' Standard Project Flood (SPF). All reservoirs without flood control storage were assumed to be at normal (conservation pool) levels at the start of all storm/flood events. Lake Bridgeport, Eagle Mountain Lake, Lake Worth, and Lake*

> *Arlington were assumed to reside at a level corresponding to 2, 3, 2, and 3, feet, respectively, above normal (conservation pool) level at the start of the SPF event.*

The isohyetal patterns for the 1997 and 2006 storms are shown in Figure 55 and Figure 56. Both events were large, having maximum point rainfall approaching 15 inches. The 1997 event had a larger areal extent and occurred over two days. As there was a lack of information, the temporal pattern of the modeled rainfall was simplified to a NRCS Type II pattern, which is 24 hour storm duration. The resulting hydrographs at Dallas are shown in Figure 57 below.

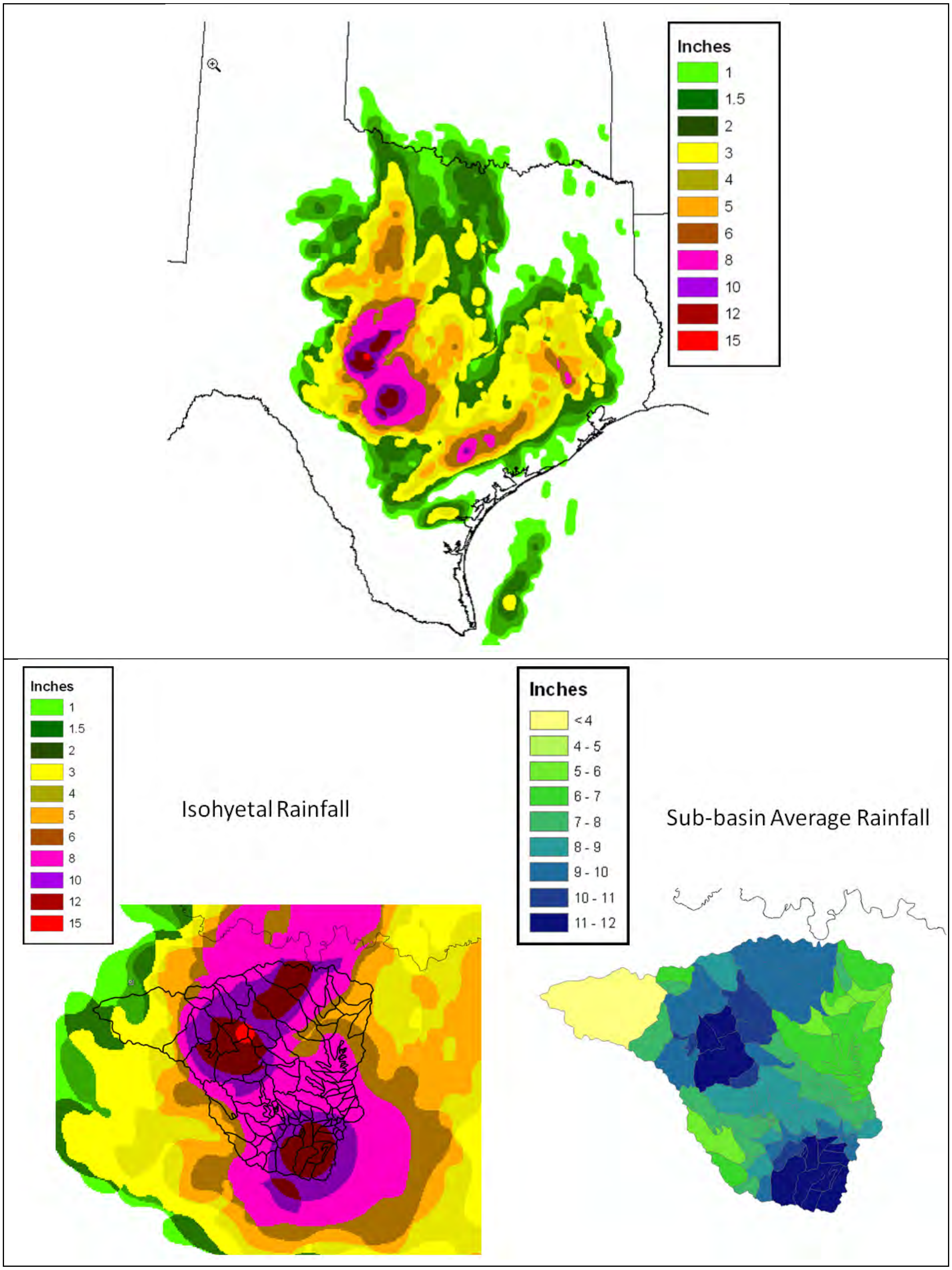


Figure 53. 1997 Regional Storm Event, Actual Location (Upper) and Transposed over Dallas Basing (Lower)

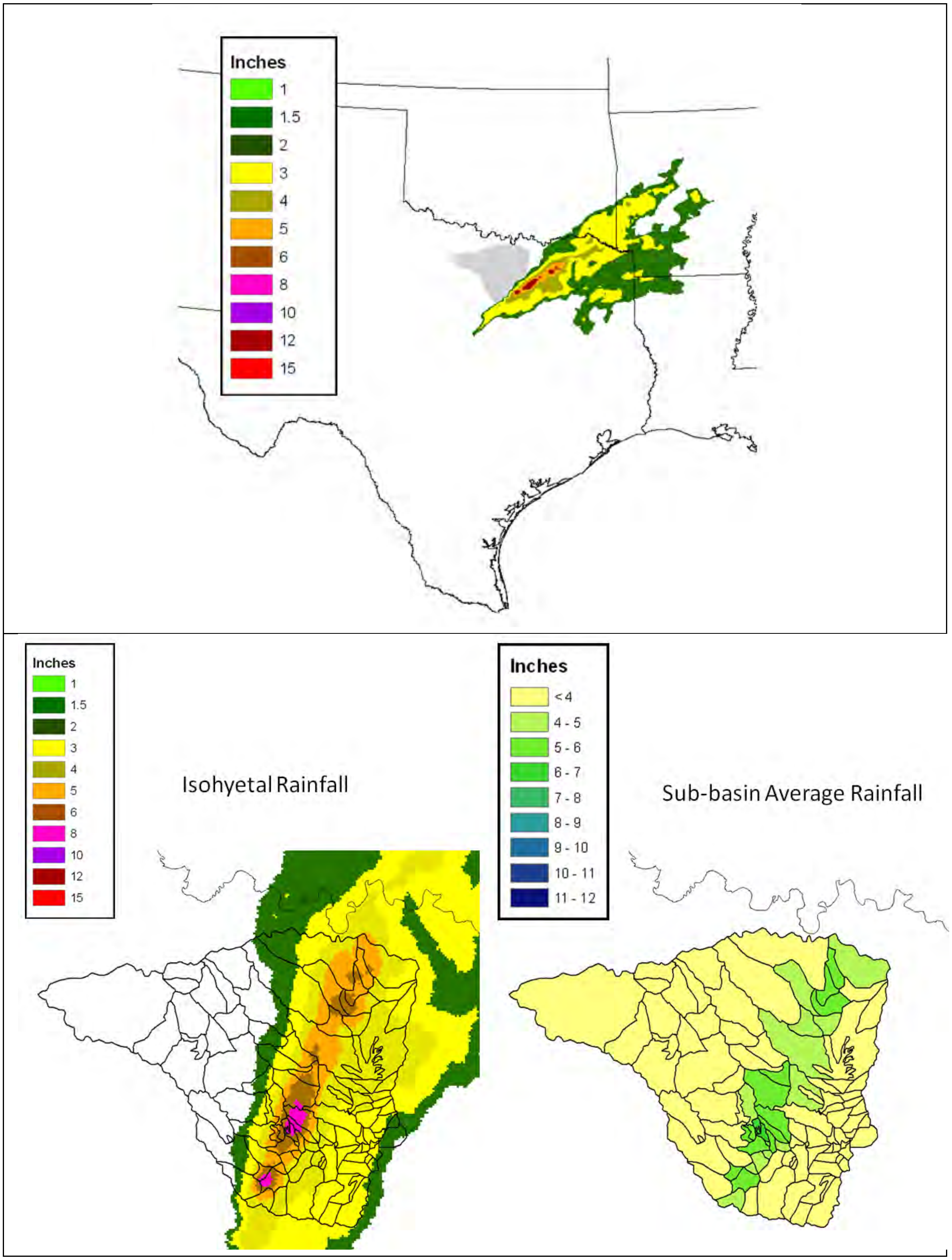


Figure 54. 1997 Regional Storm Event, Actual Location (Upper) and Transposed over Dallas Basing (Lower)

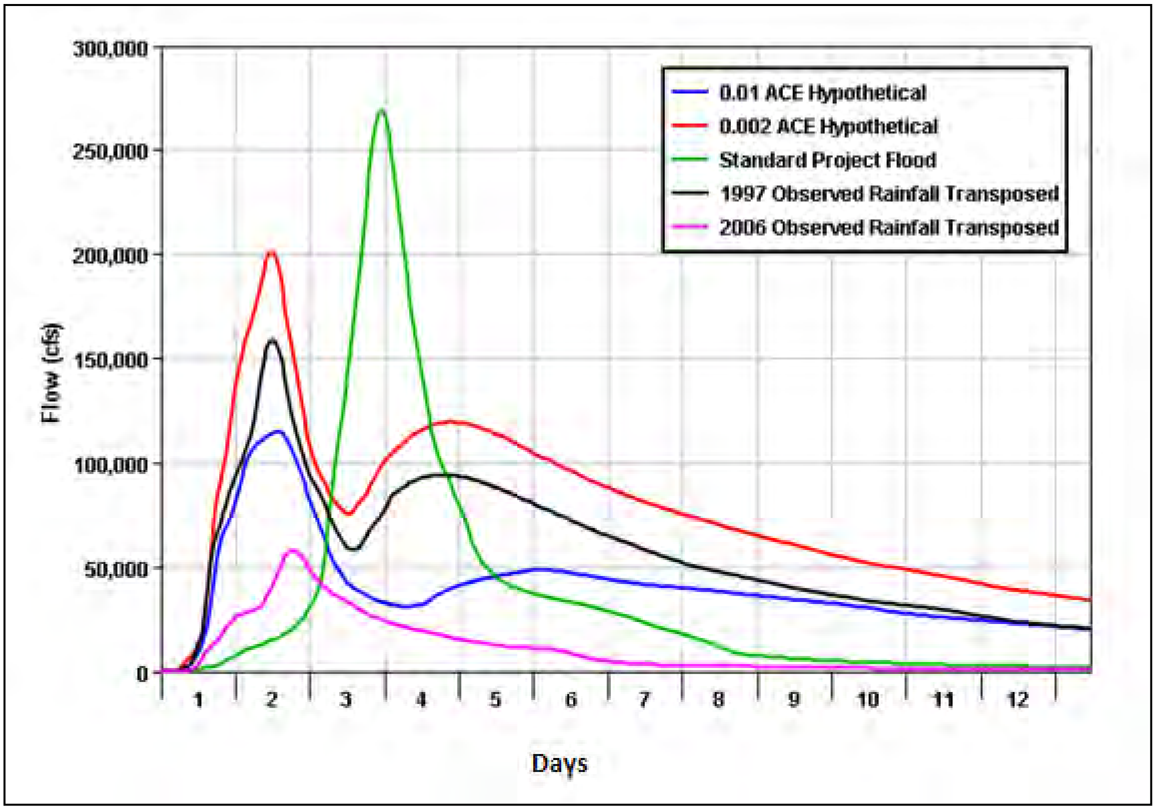


Figure 55. Hydrographs produced by the HEC-1 Rainfall-Runoff Model

Timeline for Extreme Event (SPF)

The routing of extreme events also gives insight into how much time would be available to forecast and react to an extreme flood. It is likely that during an extreme rainfall the Trinity River at Dallas will have a shorter forecasted lead time will rise faster compared to what has previously been observed. The estimated rainfall hyetograph, stage hydrograph, and flow hydrograph shown in Figure 58 was used to inform the expert elicitation in regards to available warning times for the public during an emergency.

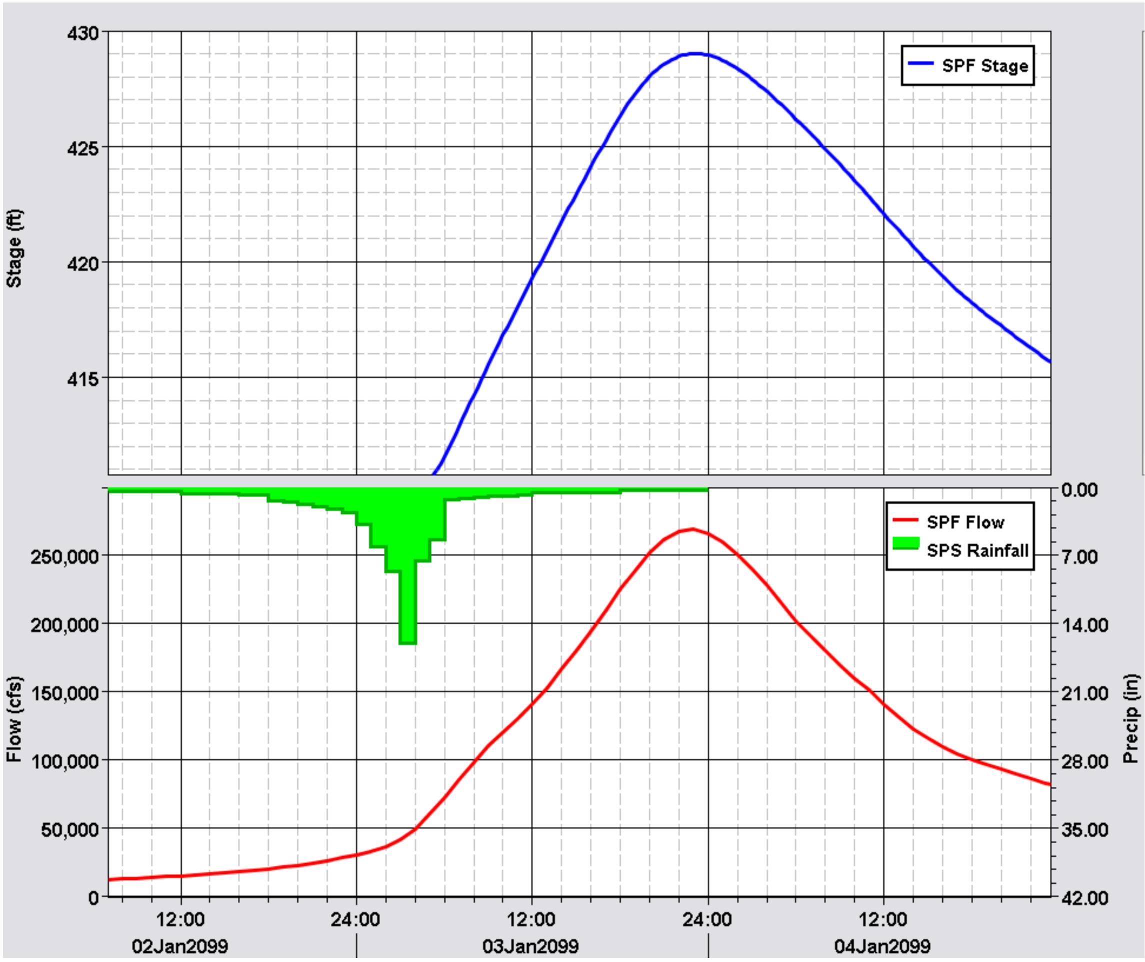


Figure 56. Timeline of Standard Project Flood - Type event

Results & Recommendations

Considering the observed hydrographs on the Trinity River at Dallas, the modeled routing of hypothetical rainfall events, and the modeled routing of 2 very large regional storms transposed over the Dallas basin; it was concluded that very high flood stages persist for several days in Dallas, but they do not stay high for weeks or months. Furthermore, the volume-frequency analysis described above indicates that a long duration period of high flow would be an extremely rare event in terms of total runoff volume. This conclusion was carried forward to the transient seepage analysis and the expert elicitation to determine the probability of failure for the seepage, piping, and stability related failure modes described in the geotechnical section of this report.

Typical flood hydrographs were provided for a range of hydraulic loading conditions for several locations along the levee system were provided as input for the transient seepage analysis (described in detail below). For this input, the hydrograph shape from the 1990 and 2007 event were used as patterns with the idea that the 1990 event had a more gradual recession, while the 2007 event had quicker peak but stayed at a moderately high level for a period of time during its recession.

If future work were to be considered in terms of analysis of hydrograph shapes on the Trinity River at Dallas, one possible and more objective approach would be to use a "balanced" hydrograph to represent a typical flood hydrograph for the system. A balanced hydrograph matches the peak flow, and the full range of the peak volumes for a given frequency based event. In this way, a "best estimate" of the representative hydrograph shape could be evaluated.

Unsteady Flow Modeling

Scope of Hydraulic Modeling and Mapping Effort

Unsteady flow hydraulic modeling of the system was performed in order to inform several aspects of the risk assessment. The results from the hydraulic modeling produced stage hydrographs for a variety of inflow scenarios at several locations of interest throughout the levee system, which were used by the risk assessment team to analyze the seepage and stability conditions at those sections. The model was also used to predict the timing and depths of inundation of the protected areas for a variety of levee breach and overtopping scenarios, which was used as input for the consequence assessment in terms of loss of life.

The hydraulic modeling effort considered only the "base condition" and made no attempt to consider proposed future projects such as the Dallas Floodway Extension Project (DFE). The modeling effort focused on the East and West levee reaches and did not consider other related nearby projects such as the Rochester Levee and the Waste Water Treatment Plant Levee.

The current version of the detailed steady flow model, known as the Corridor Development Certificate (CDC) model, was used as the basis for the construction of the unsteady flow model for the risk assessment. The model extends from near Hutchins, TX at the downstream up to near the Interstate 35E crossing on the Elm Fork River and Grand Prairie, TX on the West Fork. The downstream boundary condition, which is 10-15 river miles downstream of the study area was defined with a rating curve taken from the CDC model.

The most significant edit to the geometry of the CDC model and the Risk Assessment model is the direct incorporation of the levee profile as lateral structures and the addition of a series of interconnected storage areas to handle the spreading of flow throughout the floodplain. Several other, relatively minor edits to the geometry were made in order to convert the model to unsteady flow and calibrate the model. Although not an all-inclusive list, a general summary of those edits are shown below.

- Defined the HTAB parameters for all cross sections and structures
- Adjusted some unsteady flow expansion and contraction coefficients (note that these are separate from the steady flow C&E coefficients, which are not used for unsteady RAS)
- Some adjustments to roughness parameters to better match the steady flow model results
- Addition of one cross section near the confluence of the Elm Fork and West Fork to allow for placement of the lateral structure representing the levee in this area
- Removal of one low bridge due to unsteady constraint regarding number of cross sections between bridges

The RAS model was assembled using the Texas North Central State Plane (feet) coordinate system, to stay consistent with the CDC model. As a national standard for the Corps of Engineers Modeling, Mapping, and Consequence (MMC) center; an Albers Equal Area Projection is typically applied. However the extra step to convert the model to the standard coordinate system was not needed or warranted for the current Risk Analysis.

Vertical Datum

The CDC hydraulic model used as basis for the risk assessment modeling effort used the NGVD vertical datum. The 5-meter DEM model received from the Fort Worth District did not explicitly state the vertical datum, but was assumed to be provided in the NGVD 29 datum. The 2003 crest survey was provided in the NGVD 29 datum. Further investigation into the DEM datum was deemed unnecessary considering the vertical adjustment for the entire mapped area, as shown in the table below is less and 1-inch. More detailed future work should check for consistency among vertical datums. All elevation references in this report are reported in the NGVD 1929 project datum.

Location	**Vertical Datum Adjustment** **NAVD 88 = NGVD 29 + Adjustment**
Upstream Model Limit (West Fork)	-0.07’
Upstream Model Limit (Elm Fork)	-0.04’
Downstream Model Limit (Trinity River)	-0.05’

Data Sources

The levee profile entered into the unsteady flow model is based solely on the 2003 crest survey, which included a crest elevation roughly every 100 feet. Although a more recent 2010 survey that included crest information was taken, it was not readily available at the time of model construction. The 2003 survey was later verified against the 2010 information as the differences were minimal. The original design grade of the levee profile is of interest from a future project planning perspective, but was not necessary for the base-condition risk assessment and therefore is not included in the risk assessment hydraulic model.

For the purpose of inundation mapping, a Digital Elevation Model (DEM) was obtained from the Fort Worth District (Point of Contact: David Wilson). The DEM was based on a relatively recent LiDAR data collection, and was assumed to have a vertical datum of NAVD 29 (see Vertical Datum section for discussion on correction factors). The horizontal resolution of the original DEM provided was 5 meters

Calibration

The modeling effort did not intend to produce a detailed hydraulic model for all applications. The purpose of this effort was to create a model that could be used to obtain a reasonable estimate of consequences associated with levee failure and non- failure conditions. For this level of risk assessment, modeling procedures can often be simplified.

The unsteady RAS model was calibrated to match the results from the CDC (steady flow) model. Specifically the RAS cross sections 148,136; 135,899; 109,035; and 103,533 were used as index locations. The unsteady model generally matched the CDC model within ±0.5’ or closer.

Hydraulic Modeling of Non-Failure Scenarios

Representative Hydrographs for Transient Seepage Analysis

The unsteady hydraulic model was used to provide stage hydrographs at eight locations throughout the system as direct input to the transient seepage analysis. Three inflow magnitudes for 2 different hydrograph shapes were considered in addition to the standard project flood (SPF). Table 24 below summarizes the stage hydrographs that were provided to the geotechnical risk analysis team. The result was 56 different stage hydrographs (along with stage frequency curves), which are shown in Figure 59 to Figure 74.

The 1990 and 2007 pattern hydrographs were factored to produce stages that were approximately ½ , ¾ , and full levee loading for a typical levee section. Note that the scenarios do not produce exactly the ½ ¾ or full levee load at any of the levee sections if the levee crest is compared to the levee toe. The inflow was split between the Elm Creek (24%) and the West Fork of the Trinity River (76%). The table of inflow factors for the 1990 and 2007 patterned events is shown below in Table 25.

Table 22. Summary of Hydrographs Provided.

	1990 Pattern Hydrograph			2007 Pattern Hydrograph			
	½ Load	¾ Load	Full Load**	½ Load	¾ Load	Full Load**	SPF
Location							
East 410	X	X	X	X	X	X	X
East 310	X	X	X	X	X	X	X
East 220	X	X	X	X	X	X	X
East 74	X	X	X	X	X	X	X
West 335	X	X	X	X	X	X	X
West 250	X	X	X	X	X	X	X
West 188	X	X	X	X	X	X	X
West 10	X	X	X	X	X	X	X

***The "Full Load" inflow did produce some of overtopping of the levee in the lowest areas. This inflow scenario was used also for the consequence assessment, but named "Overtopping A"*

Table 23. Flow Factors used for the Development of Hydrographs

Pattern	Nominal Loading	Factor
1990	½ Levee	1.7
	¾ Levee	2.75
	Full Levee/Overtopping A	3.75
2007	½ Levee	3.3
	¾ Levee	5.5
	Threshold**	6.7
	Full Levee/Overtopping A	7.9
	Overtopping B**	9.0

***The "threshold" and "overtopping B" scenarios were additional runs used for the consequence modeling but were not used for the transient seepage analysis*

To carry the results from the expert elicitation regarding the probability of failure from stability and internal erosion failure modes forward to the consequence and risk assessment, it is necessary to assign a flood frequency for each of the loading conditions. For a observed/actual flood event, the frequency of the peak flow not typically match the frequency of the volume of the flood hydrograph. The risk analysis team found that longer duration flood hydrographs that are reasonable for the Trinity River at Dallas (see Hydrograph Shape Analysis above) do not generally induce a significantly worse condition in terms of stability and internal erosion compared to floods that have a shorter duration and higher peak. For this reason, the team adopted the frequencies shown in Table 26 for the flow scenarios considered.

Table 24. Peak Flow Frequencies and Confidence Limits.

Location	**Discharge (cfs)**	**ACE+**	**5% Conf Limit**	**95% Conf Limit**
June 2007 Flood	35,700	0.213306	0.289829	0.149791
		(1/5)	(1/3)	(1/7)
May 1990 Flood	72,100	0.040015	0.077197	0.019325
		(1/25)	(1/13)	(1/52)
½ Levee	117,810	0.008629	0.023682	0.002821
		(1/116)	(1/42)	(1/354)
¾ Levee	196,350	0.001307	0.005506	0.000263
		(1/765)	(1/182)	(1/3799)
Threshold*	232,050	0.000659	0.003257	0.000110
		(1/1517)	(1/307)	(1/9089)
Full Levee /Overtopping B**	282,030	0.000289	0.001738	0.000040
		(1/3461)	(1/575)	(1/25,274)
Overtopping B	321,300	0.000160	0.001113	0.000020
		(1/6251)	(1/899)	(1/51,018)
***Threshold is the discharge before levee begins to overtop**				
****Full Levee scenario overtops some low points in levee**				

Table 25. 60 Day Volume Frequencies

Location	60 Day flow (cfs)	ACE	5% Conf Limit	95% Conf Limit
June 2007 Flood	10,050	0.165195	0.256207	0.095783
		(1/6)	(1/4)	(1/10)
May 1990 Flood	20,914	0.012270	0.043767	0.002112
		(1/81)	(1/23)	(1/474)
½ Levee	33,164	0.000544	0.006660	0.000011
		(1/1837)	(1/150)	(1/88,401)
¾ Levee	55,274	0.000001	0.000240	0.000000
		(1/>100,000)	(1/4173)	(1/>100,000)
Full Levee /Overtopping B*	79,394	0.000000	0.000006	0.000000
		(1/>100,000)	(1/>100,000)	(1/>100,000)
***Full Levee (Top of Levee) does overtop low points in levee**				

Table 26. 90 Day volume Frequencies

Location	90 Day flow (cfs)	ACE	5% Conf Limit	95% Conf Limit
June 2007 Flood	6,963	0.248669	0.347277	0.165246
		(1/4)	(1/3)	(1/6)
May 1990 Flood	15,534	0.023554	0.067065	0.005779
		(1/42)	(1/15)	(1/173)
½ Levee	22,978	0.002820	0.017660	0.000199
		(1/355)	(1/57)	(1/5037)
¾ Levee	38,297	0.000026	0.001260	0.000000
		(1/38,610)	(1/794)	(1/>100,000)
Full Levee /Overtopping B*	55,008	0.000000	0.000072	0.000000
		(1/>100,000)	(1/13,901)	(1/>100,000)
***Full Levee (Top of Levee) does overtop low points in levee**				

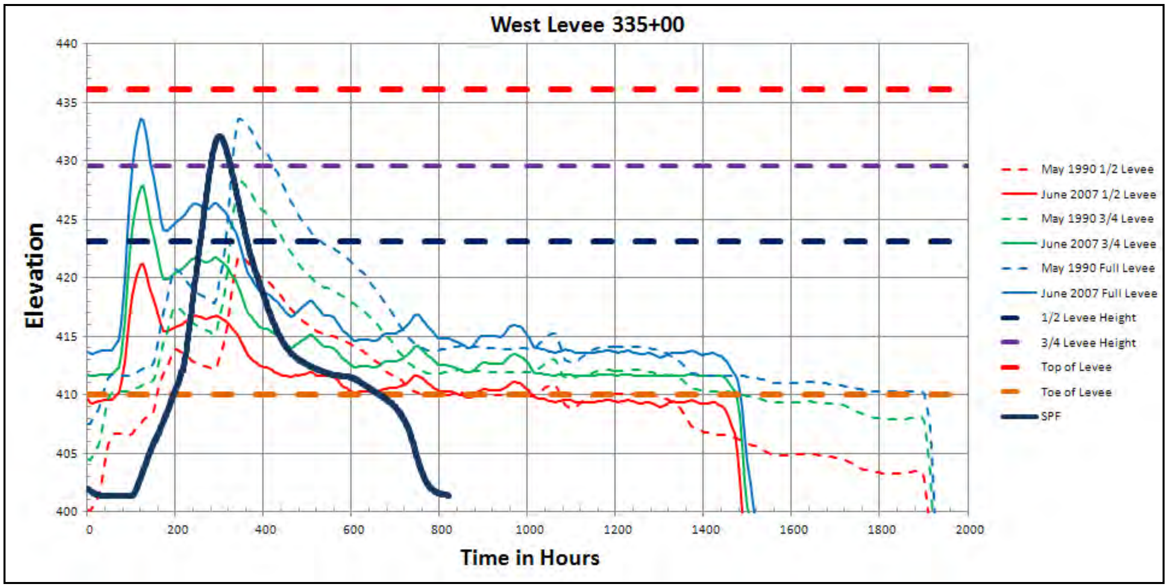


Figure 57. Levee Section W 335+00 non-failure flood durations

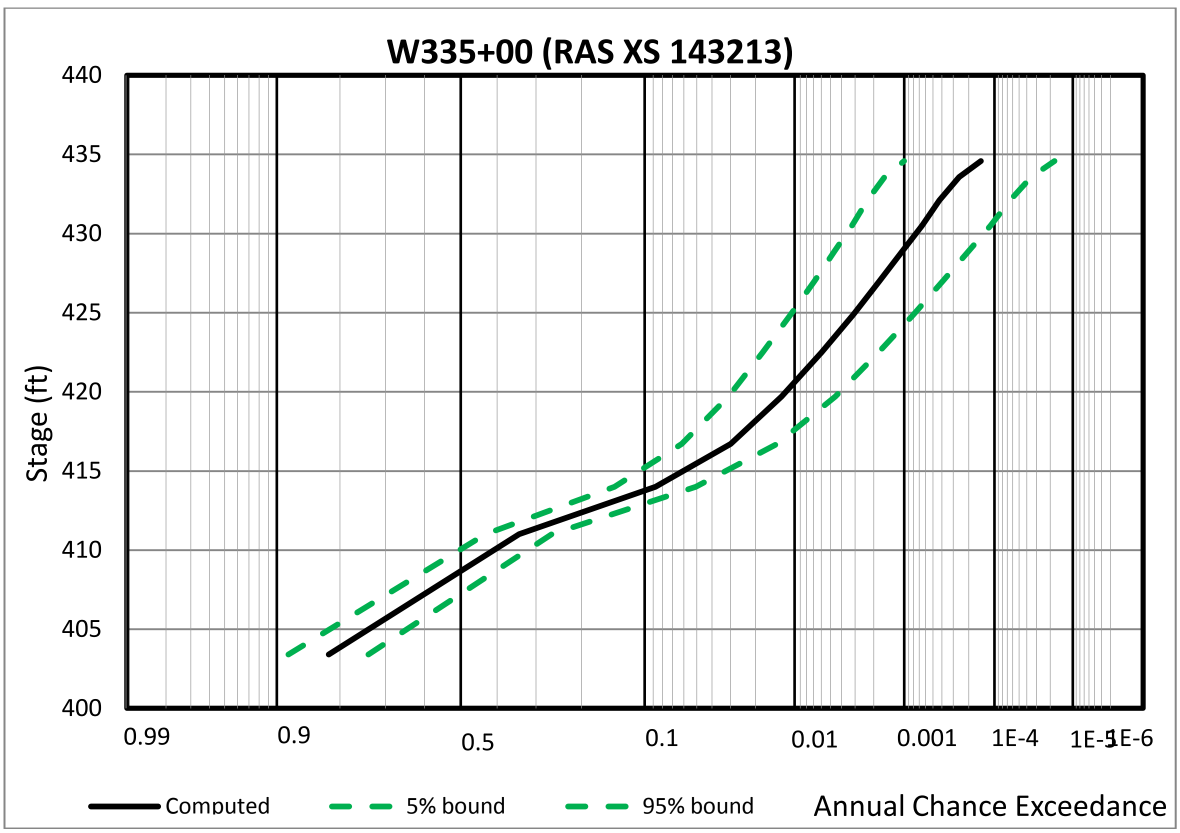


Figure 58. Levee Section W 335+00 Stage frequency Curve

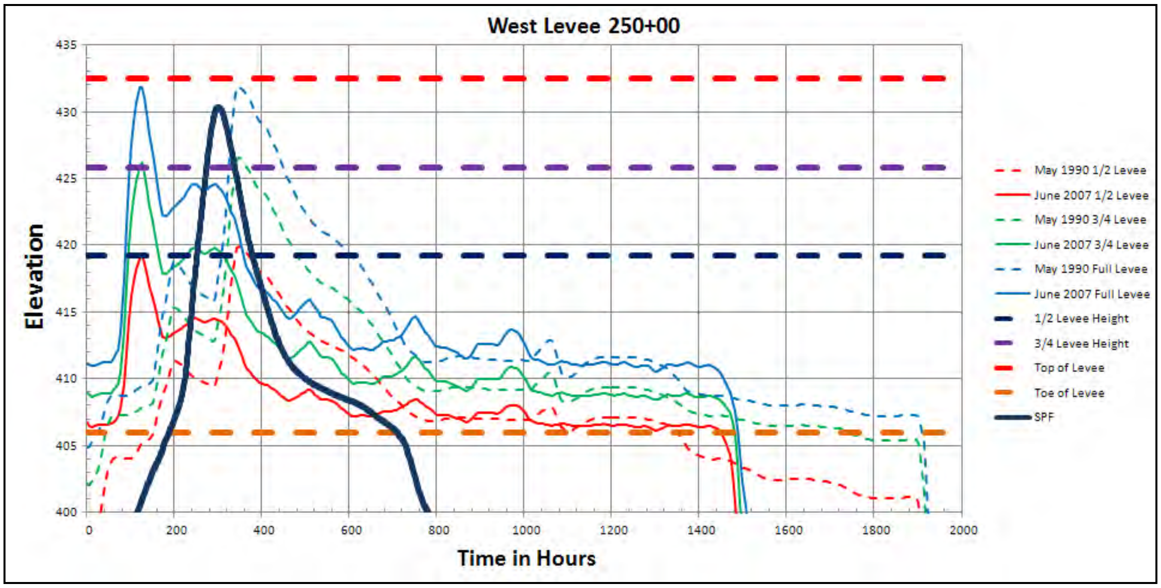


Figure 59. Levee Section W 225+00 non-failure flood durations

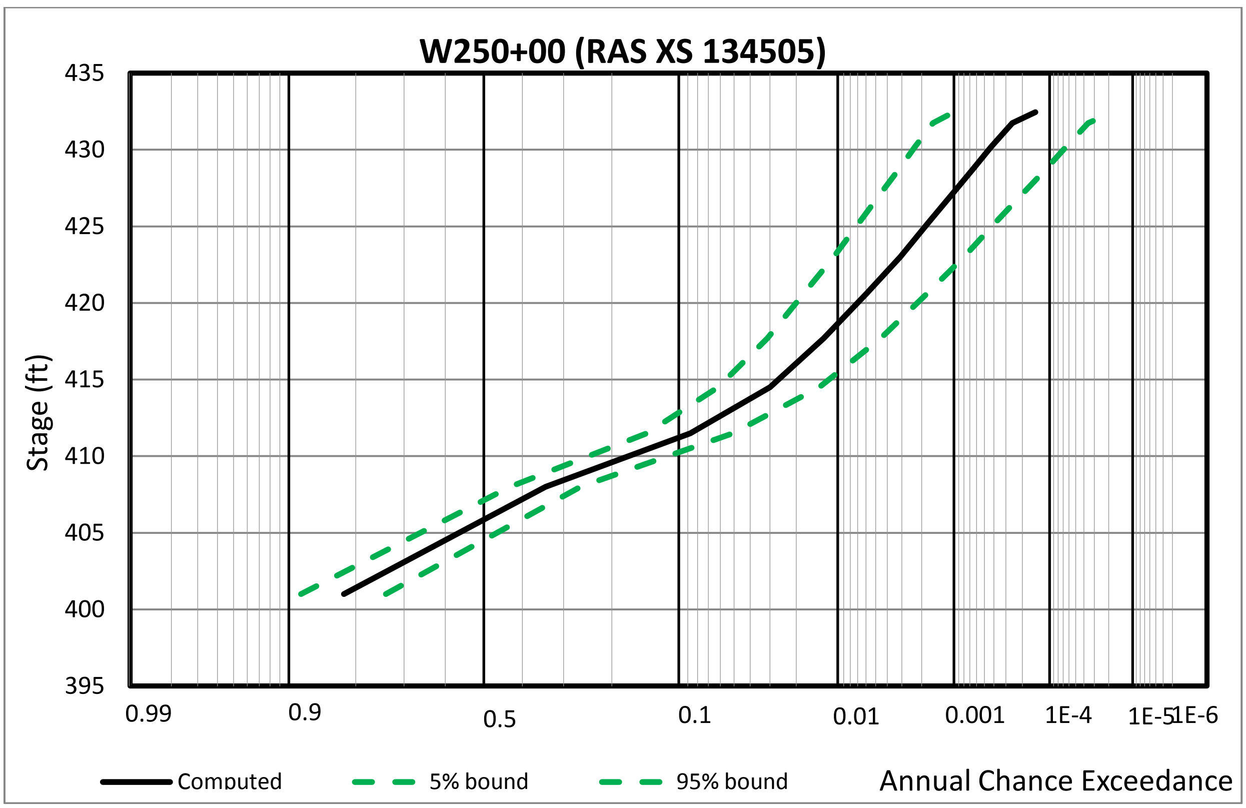


Figure 60. Levee Section W 250+00 Stage Frequency Curve

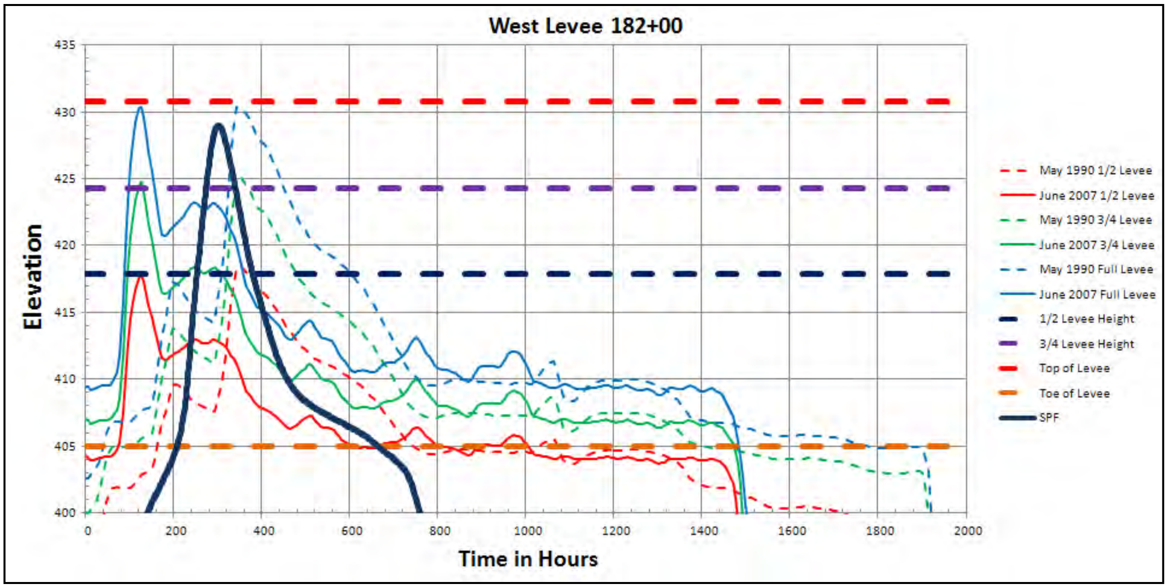


Figure 61. Levee Section W 182+00 non-failure flood durations

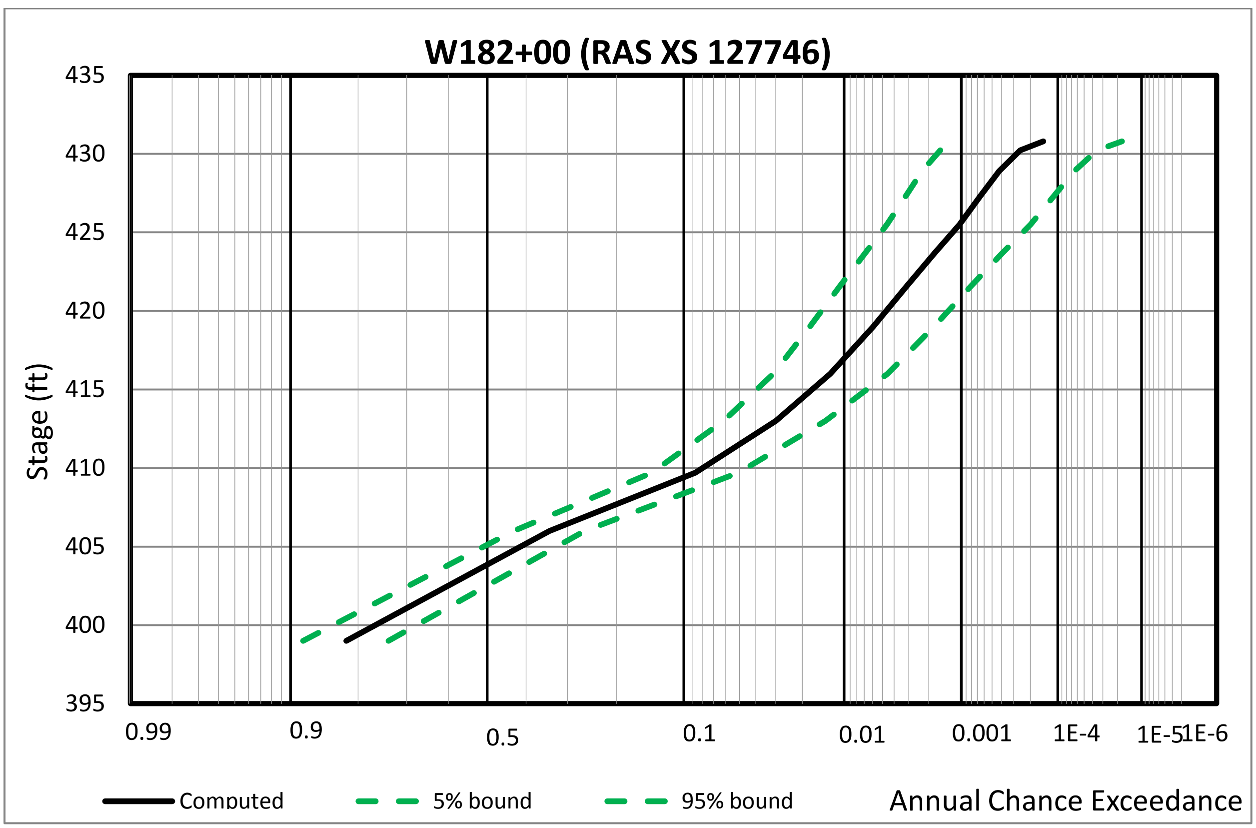


Figure 62. Levee Section W 182+00 Stage Frequency Curve

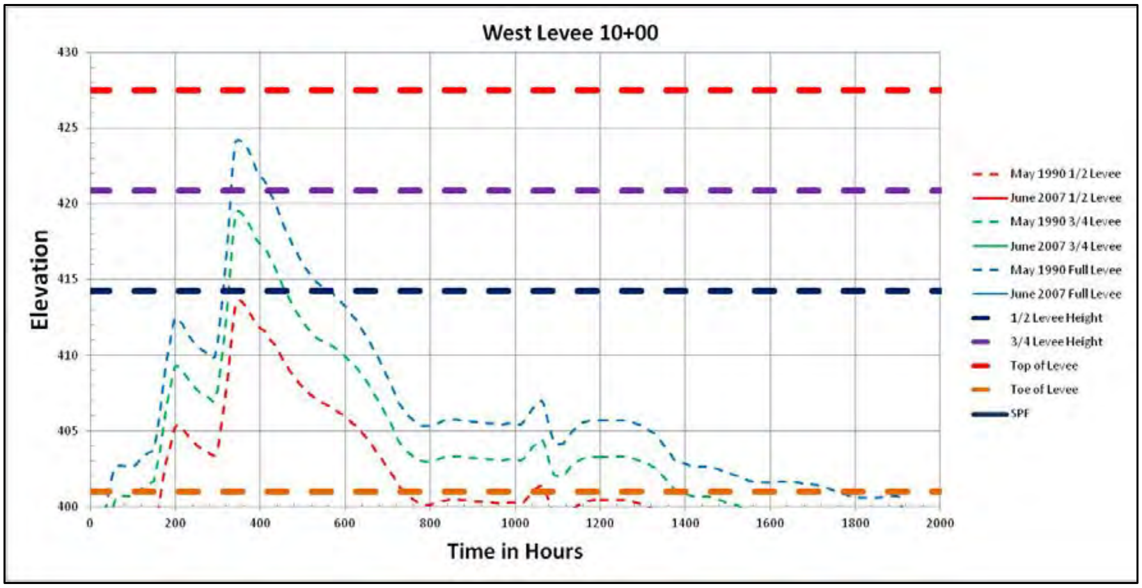


Figure 63. Levee Section W 10+00 non-failure flood durations

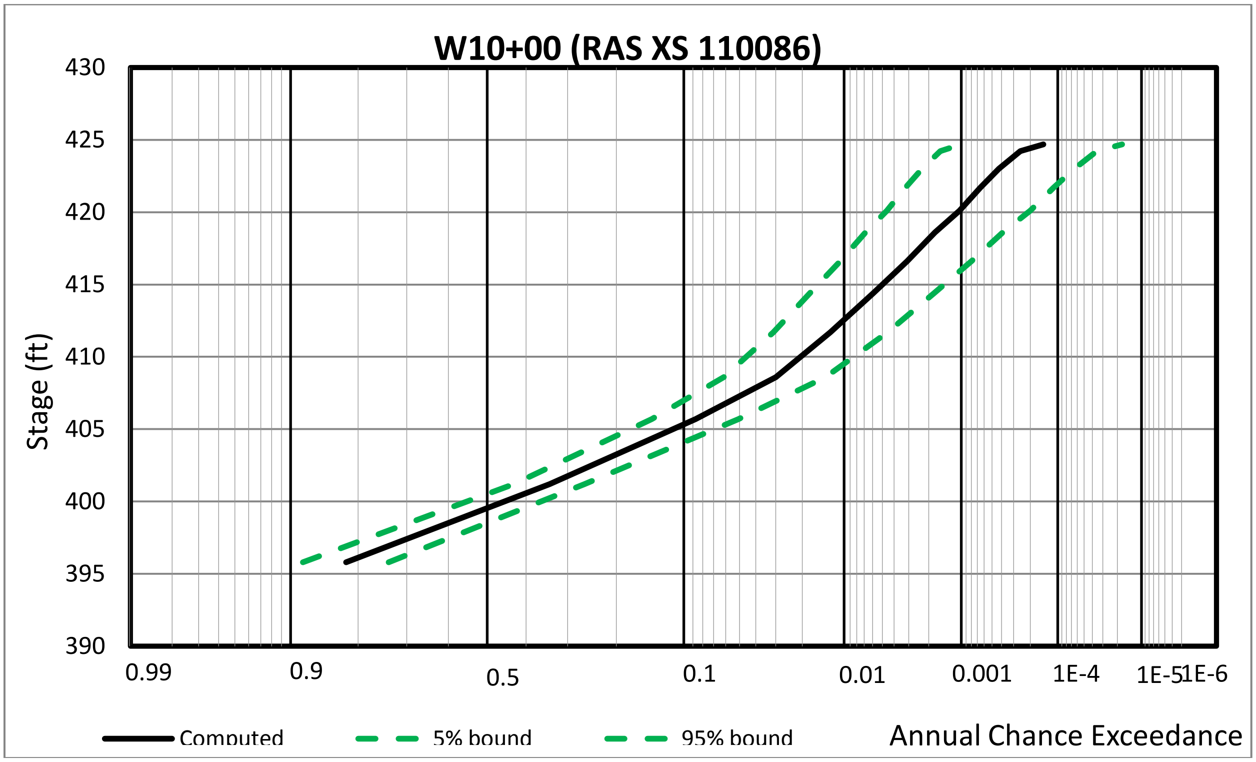


Figure 64. Levee Section W 10+00 Stage Frequency Curve

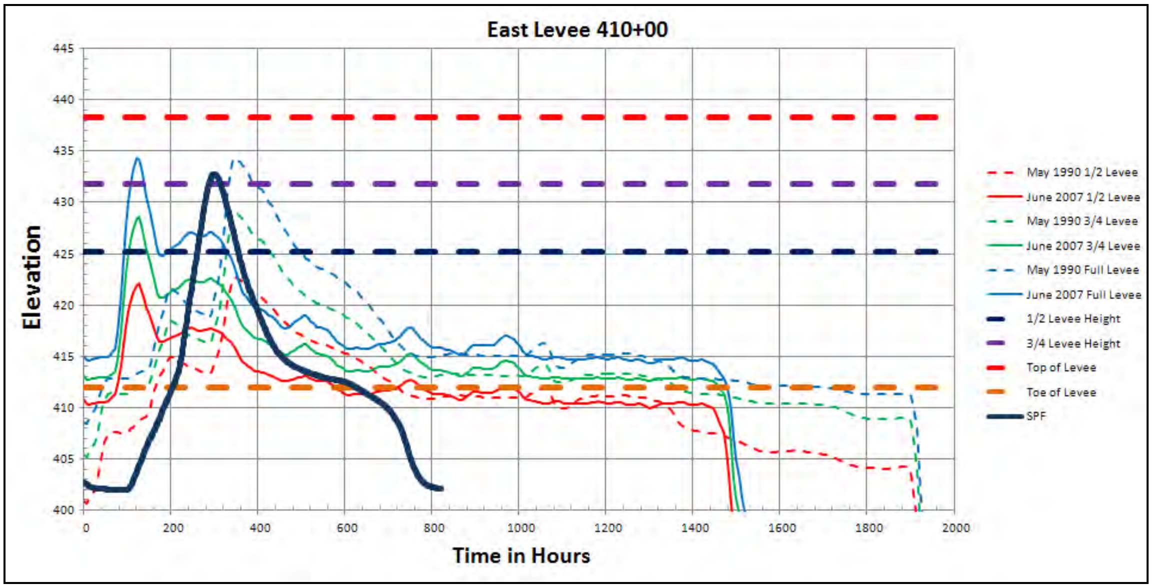


Figure 65. Levee Section E 410+00 non-failure flood durations

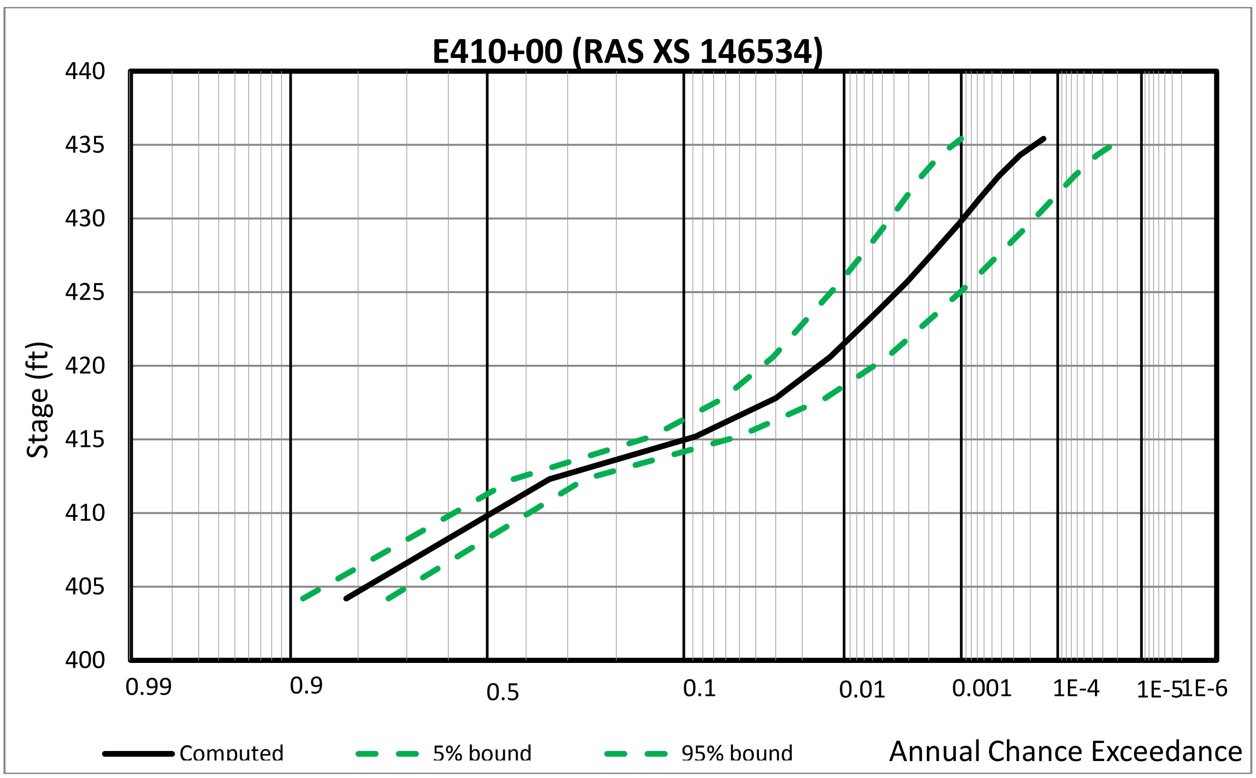


Figure 66. Levee Section E 410+00 Stage Frequency Curve

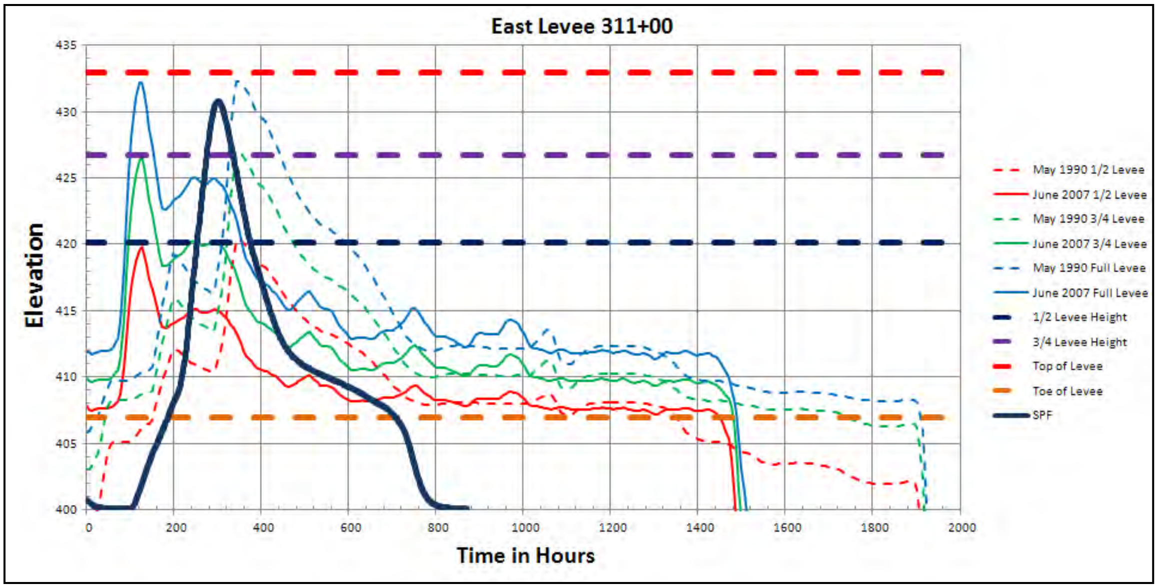


Figure 67. Levee Section E 311+00 non-failure flood durations

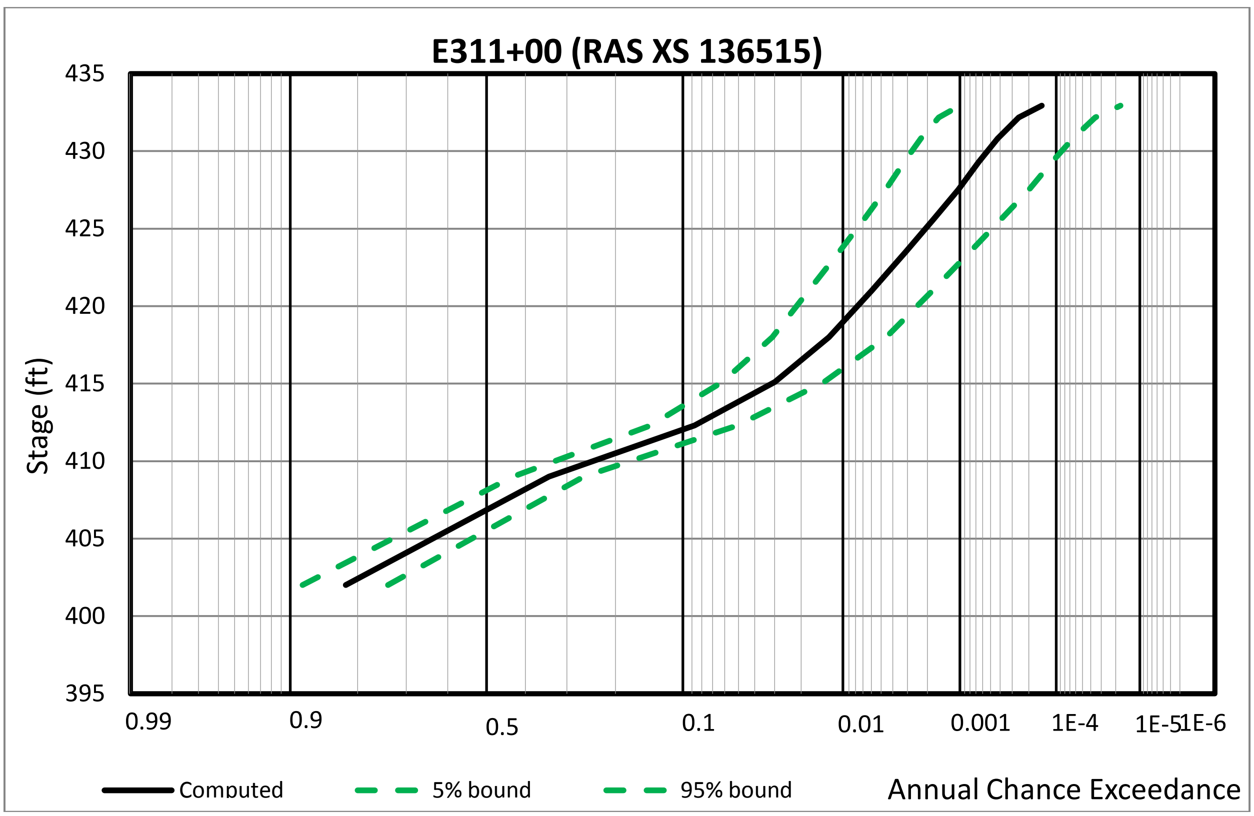


Figure 68. Levee Section E 311+00 Stage Frequency Curve

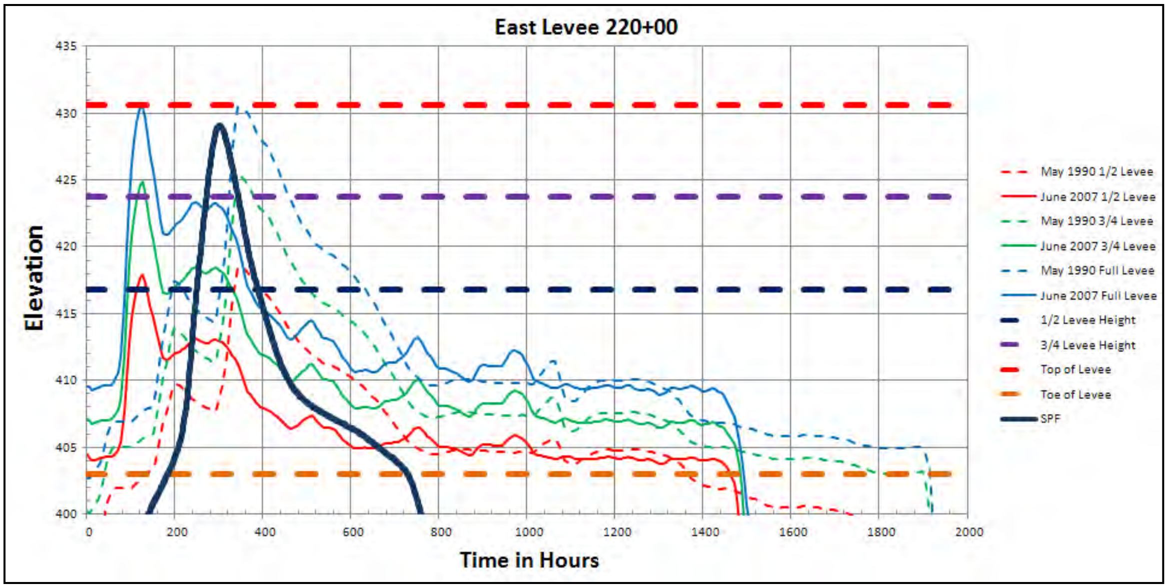


Figure 69. Levee Section E 220+00 non-failure flood durations

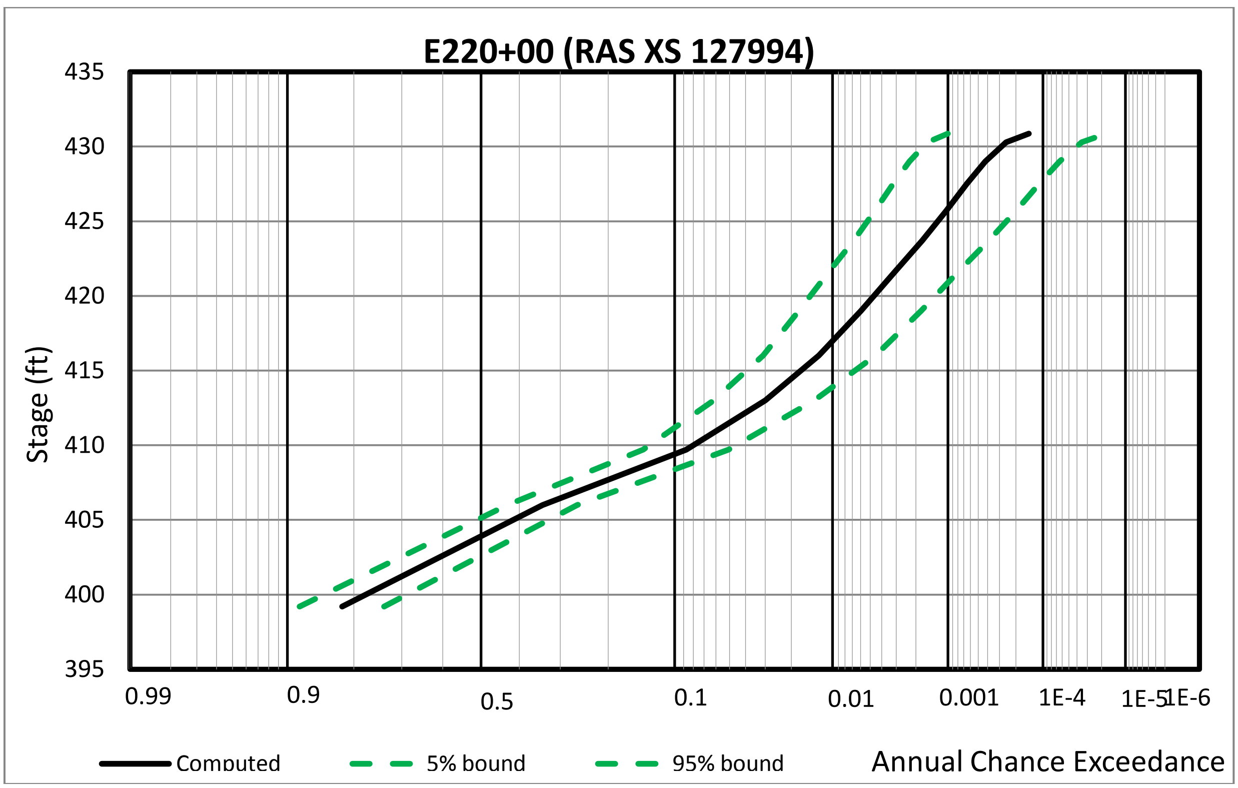


Figure 70. Levee Section E 220+00 Stage Frequency Curve

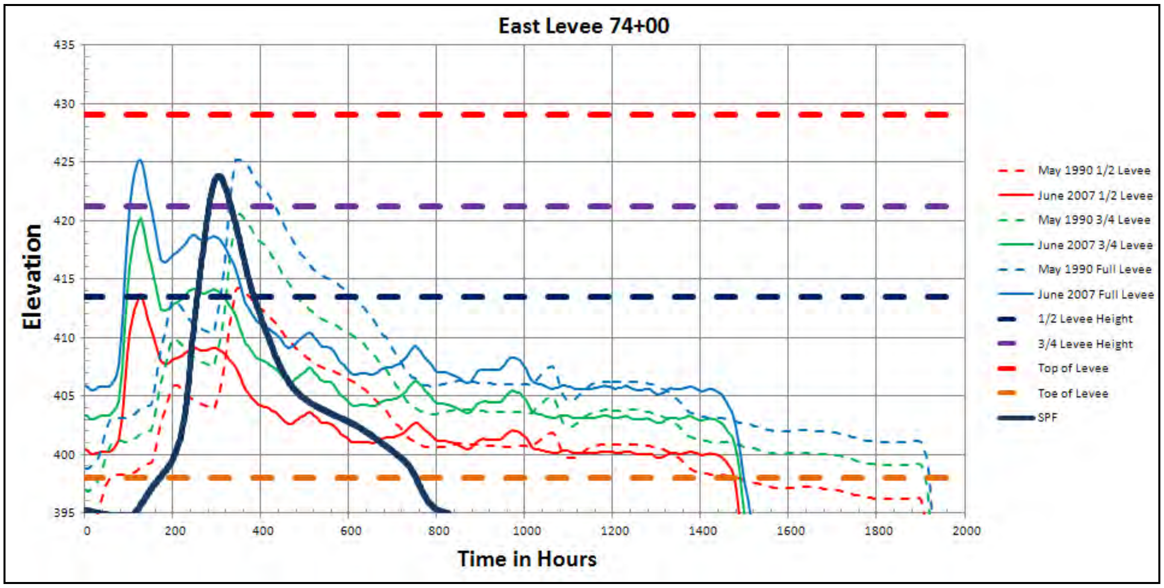


Figure 71. Levee Section E 74+00 non-failure flood durations

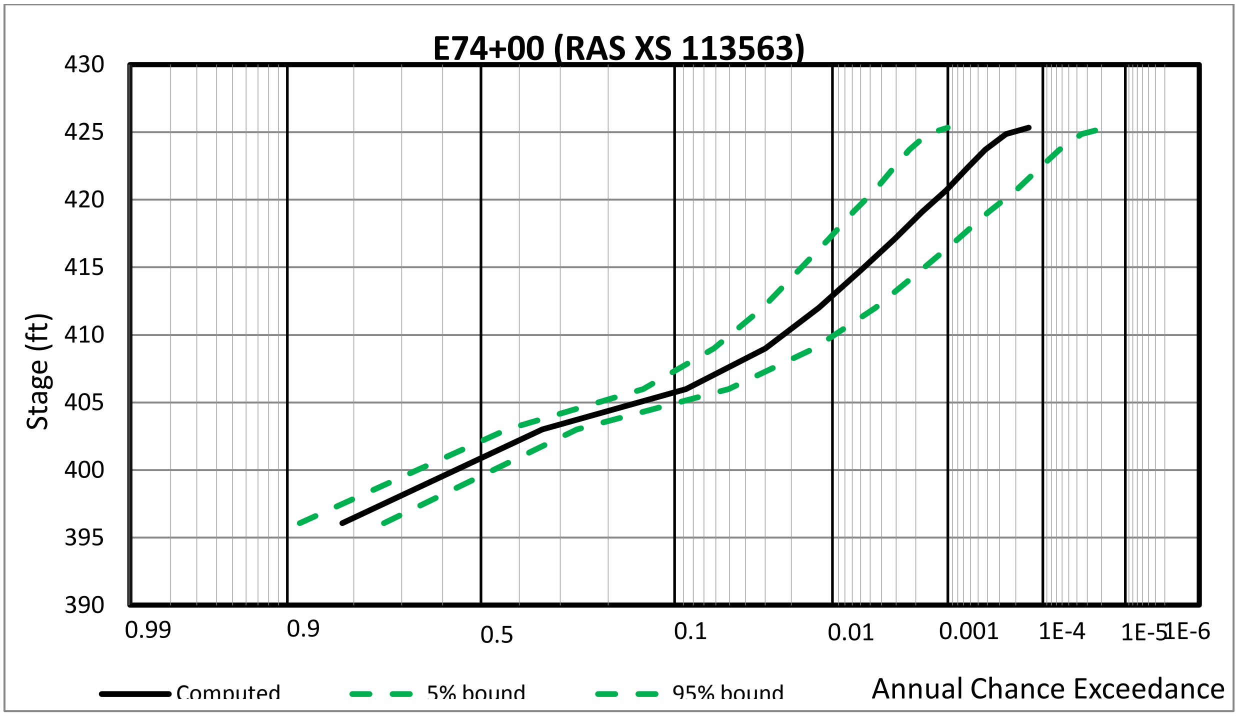


Figure 72. Levee Section E 74+00 Stage Frequency Curve

Table 27. Geotechnical Sections of Interest

	Location	Elevation	Frequency	5% bound	95% bound
Section W335 / RAS XS 143213	Toe Elev	410.0	0.406	0.492	0.326
			2yr	2yr	3yr
	Hyd 1/2 levee	421.2	0.009	0.024	0.003
			(1/111)	(1/42)	(1/329)
	Hyd 3/4 levee	427.9	0.001	0.006	0.000
			(1/700)	(1/172)	(1/3299)
	Hyd Full levee	433.6	0.0003	0.0016	0.0000
			(1/3888)	(1/633)	(1/28216)
Section W250 / RAS XS 134505	Toe Elev	406.0	0.479	0.561	0.399
			2yr	2yr	3yr
	Hyd 1/2 levee	419.3	0.009	0.024	0.003
			(1/111)	(1/42)	(1/331)
	Hyd 3/4 levee	426.2	0.001	0.006	0.000
			(1/719)	(1/175)	(1/3412)
	Hyd Full levee	431.8	0.0002	0.0015	0.0000
			(1/4086)	(1/660)	(1/29890)
Section W188 / RAS XS 127746	Toe Elev	405.0	0.411	0.497	0.331
			2yr	2yr	3yr
	Hyd 1/2 levee	417.9	0.009	0.023	0.003
			(1/117)	(1/44)	(1/352)
	Hyd 3/4 levee	424.8	0.001	0.005	0.000
			(1/754)	(1/182)	(1/3624)
	Hyd Full levee	430.3	0.0002	0.0015	0.0000
			(1/4239)	(1/680)	(1/31199)
Section W10 / RAS XS 110086	Toe Elev	401.0	0.360	0.449	0.281
			3yr	2yr	4yr
	Hyd 1/2 levee	413.1	0.009	0.024	0.003
			(1/108)	(1/41)	(1/319)
	Hyd 3/4 levee	419.3	0.001	0.006	0.000
			(1/701)	(1/172)	(1/3306)
	Hyd Full levee	424.2	0.0003	0.0016	0.0000
			(1/3885)	(1/633)	(1/28188)
Section E410 / RAS XS 146534	Toe Elev	412.0	0.360	0.449	0.281
			3yr	2yr	4yr
	Hyd 1/2 levee	422.1	0.009	0.024	0.003
			(1/109)	(1/42)	(1/323)
	Hyd 3/4 levee	428.6	0.001	0.006	0.000
			(1/685)	(1/169)	(1/3213)
	Hyd Full levee	434.3	0.0003	0.0016	0.0000
			(1/3817)	(1/625)	(1/27564)
Section E311 / RAS XS 136515	Toe Elev	407.0	0.480	0.562	0.400
			2yr	2yr	3yr
	Hyd 1/2 levee	419.8	0.009	0.023	0.003
			(1/114)	(1/43)	(1/342)
	Hyd 3/4 levee	426.6	0.001	0.006	0.000
			(1/717)	(1/175)	(1/3400)
	Hyd Full levee	432.2	0.0002	0.0015	0.0000
			(1/4023)	(1/652)	(1/29354)
Section E220 / RAS XS 127994	Toe Elev	403.0	0.554	0.632	0.474
			2yr	2yr	2yr
	Hyd 1/2 levee	418.0	0.008	0.022	0.003
			(1/120)	(1/45)	(1/364)
	Hyd 3/4 levee	424.9	0.001	0.005	0.000
			(1/754)	(1/182)	(1/3623)
	Hyd Full levee	430.5	0.0002	0.0014	0.0000
			(1/4485)	(1/712)	(1/33344)
Section E74 / RAS XS 113563	Toe Elev	398.0	0.688	0.759	0.607
			1yr	1yr	2yr
	Hyd 1/2 levee	413.7	0.009	0.023	0.003
			(1/115)	(1/43)	(1/345)
	Hyd 3/4 levee	420.2	0.001	0.005	0.000
			(1/800)	(1/190)	(1/3914)
	Hyd Full levee	425.2	0.0002	0.0011	0.0000
			(1/5859)	(1/880)	(1/46151)

***This table represents the stage-frequency information at specific cross sections based on the levee height each section (crest elevation minus toe elevation). They differ from the frequencies of the nominal ½, ¾, and Full levee inflow scenarios shown in Table 26, which were used for the purposes of consequence assessment.

Hydraulic Modeling of Levee Failure Scenarios

Overtopping Locations

In its present state, the top of levee profile (or crest profile) of the Trinity River Levee System at Dallas has significant variability compared to expected water surface profiles. The original design grade (circa 1952) of the levee has been altered due to a combination of settlement, sloughing, local crest restoration projects, and construction tolerances. It should be noted that the 1950s design of the project left the tie-back floodwall on the downstream end of the East Levee, and by doing so allowed for a natural first overtopping location of the system. The profile of the East and West Levee systems including design grade, surveyed crest grade (2003 survey), and the modeled "threshold" flood water surface profile is shown in Figure 75 and Figure 76 below.

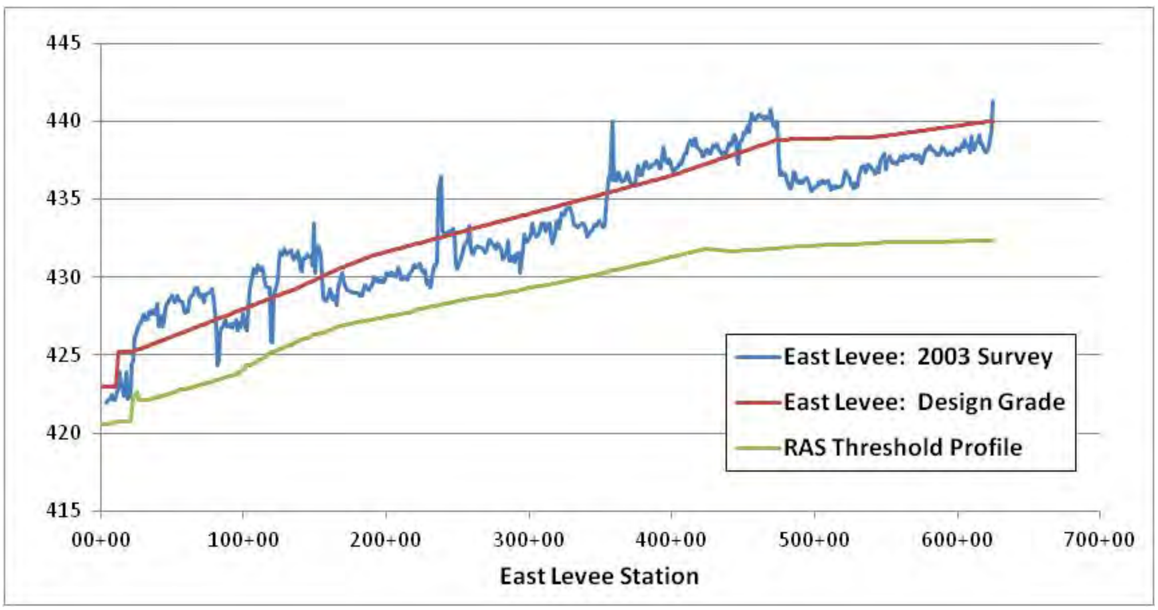


Figure 73 - East Levee Crest Profile

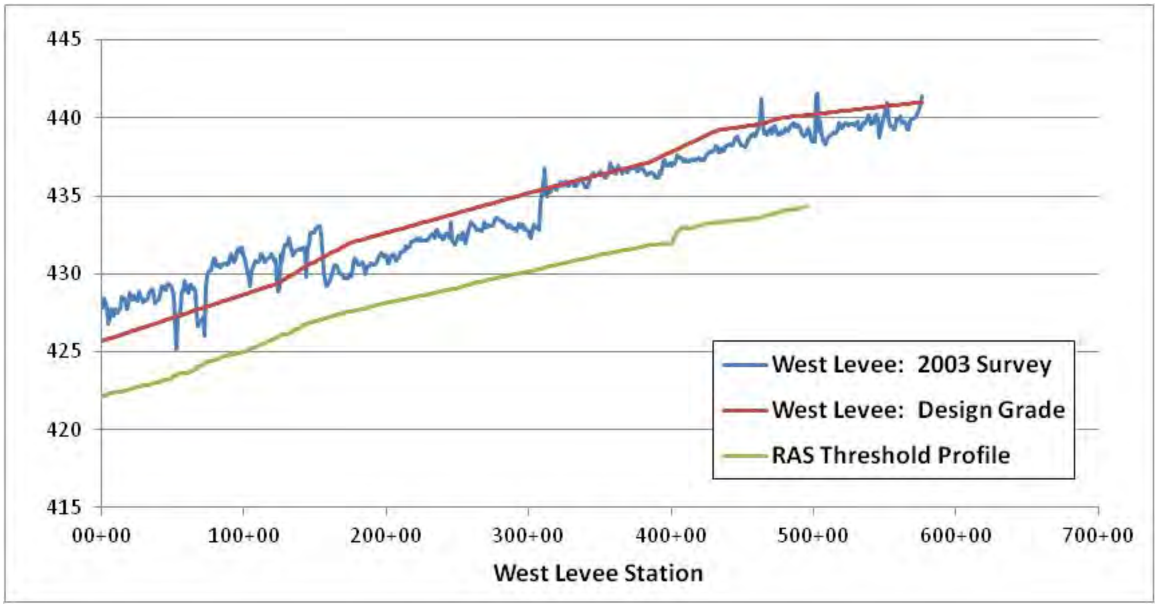


Figure 74. West Levee Crest Profile

It is expected that overtopping will occur at the east downstream floodwall first. The largest modeled flood that does not cause overtopping at the downstream floodwall was labeled as the "threshold flood". For a larger flood, it is expected that levee sections further upstream would also overtop.

Due to the large volume of storage capacity in the protected area, an overtopping event that does not also cause a levee breach would not completely inundate the protected area (i.e. would not equalize the water levels between the river and land side of the levee). Therefore, the risk assessment includes hydraulic and consequence modeling scenarios for Overtopping without Breach and Overtopping with Breach in order to determine the incremental consequences due to a levee failure. An example of the difference in inundation outlines for a "with" versus "without" breach is shown in Figure 78 to illustrate the meaning of incremental consequences.

Three overtopping failure locations were considered and two overtopping flood scenarios (larger than the threshold flood) were considered. A plan view location of the selected overtopping driven breach locations is shown in Figure 77 below. The maximum depth and duration of overtopping for each scenario was determined based on HEC-RAS unsteady flow modeling and was used to inform the expert elicitation for the Risk Assessment (see Table 30 below).

Hydrologic Load	Threshold		Overtopping A		Overtopping B	
Peak Inflow	**Q = 232,050 cfs**		**Q = 282,030 cfs**		**Q = 321,300 cfs**	
Overtopping Location	**Depth (ft)**	**Duration (hrs)**	**Depth (ft)**	**Duration (hrs)**	**Depth (ft)**	**Duration (hrs)**
East Floodwall Station: E 0 to 10+30	0	0	1.7	35	2.3	50
East Levee Station: E 180 to E 240	-1 **	0	1.6	24	2.2	41
West Levee Station W 140 to W 185	-2 **	0	0.6	15	1.3	33.5

** does not overtop, negative values indicate "freeboard"

Notes: - Results obtained from HEC-RAS unsteady, non-failure modeling
- All runs used the factored 2007 hydrograph pattern

Table 28. Non-Failure Overtopping Summary

Figure 75 - Modeled Overtopping Breach Locations

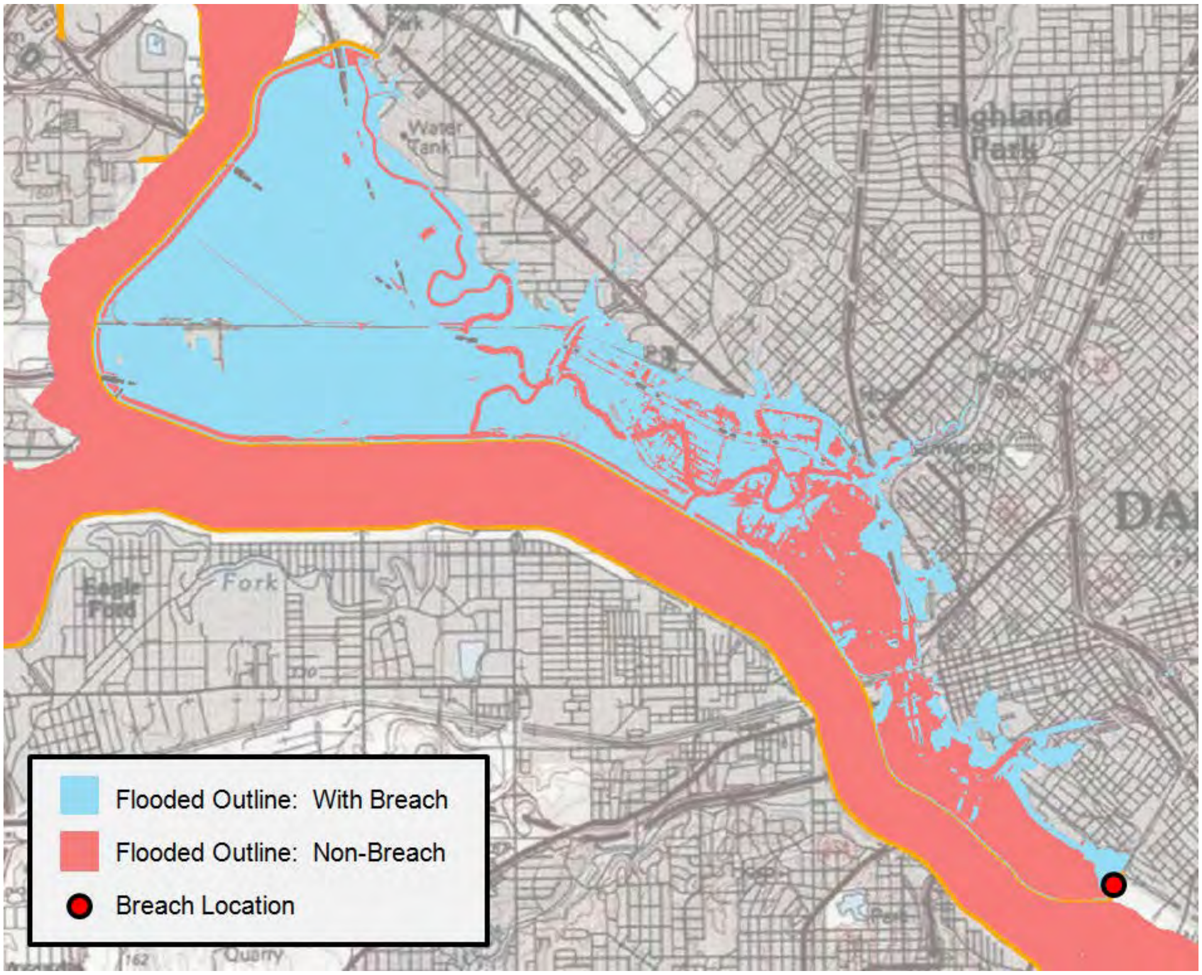


Figure 76 - Example of Overtopping With and Without Breach Inundation Scenario

Levee Erosion Due To Overtopping: WinDAMB Analysis

To assist the risk assessment team in assessing potential breach formation time and breach width estimates for the risk estimate, WinDAMB, version 1.0 was used. Although WinDAMB is designed for dam overtopping, the erosion mechanism should be similar although the hydraulic conditions may be significantly different. The results of the analysis should still be informative for this study. A range of input parameters were modeled to provide a range of breach time formations and breach widths to assist in the determination of the final values to use in consequence modeling.

For WinDAMB breach initiation begins when erosion first begins on the upstream (river) side of the levee crest. This is the point when downcutting begins to increase flow over the levee. Any erosion on the crest downstream from this point or downstream slope does not indicate breach initiation. Figure 79 shows when breach initiation begins.

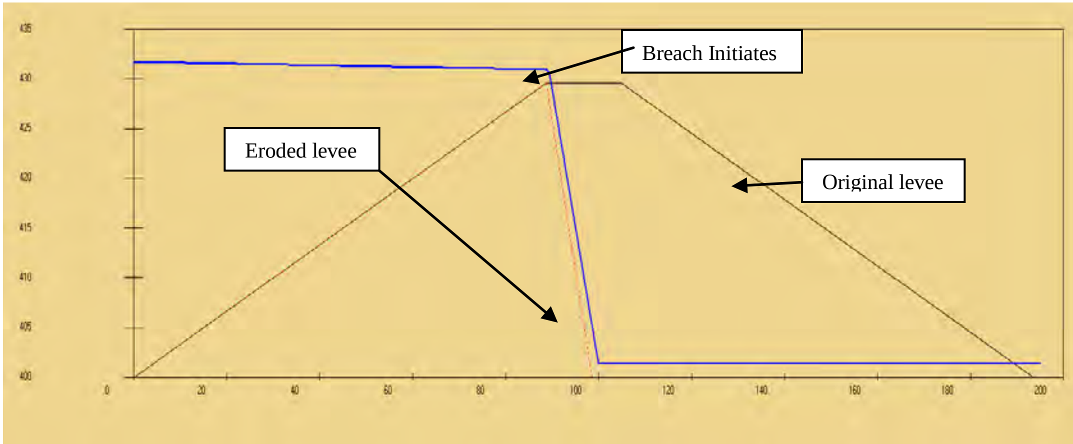


Figure 77. Graphical Depiction from WinDAMB model

Key parameters for the WinDAMB model are the inflow hydrograph, total unit weight for the soil, erodibility index (k_d), undrained shear strength, plasticity index, and particle diameter in inches. The inflow hydrograph and storage volume upstream of the dam were entered to provide up to 3 feet of overtopping over a period of 40 hours. Storage was held to a low value so the hydrograph with overtop the levee without attenuation to simulate a levee overtopping flow. A peak inflow of 20,000 cfs for 40 hours was used and a levee length of 1000 feet was used for the levee overtopping profile. At the center of the profile, the levee top was lowered 0.4 feet so flow would be similar to flow overtopping the levee at its lowest point.

Soil parameters were estimated using a range of values from the HNTB report, "Dallas Floodway System – 100-Year Levee Remediation, 408 Application, DRAFT, Geotechnical Information, June 3, 2011". For clay soils found in the levee, the range of undrained shear strengths was 1250 pounds per square foot (psf) to 3000 psf. Moist unit weight ranged from 118 pounds per cubic foot (pcf) to 128 pcf. Plasticity Index (PI) ranged from 36 to 71. For the clay materials particle size ranges were 0.01 mm to 0.166 mm, both are below the model threshold of 0.5 inches (1.27 mm) so 0.4 inches was used in the model. Using an ARS qualitative description for k_d (See Figure 80), the range for material with more than 25% clay content (identified range was 40% to 80% clay) were 0.1 to 1. Some other guides for selecting k_d indicated the clay content would have even more erosion resistant values as low as 0.01 $_{kd}$. These values were not used in this sensitivity analysis but would indicate the levee was more erosion resistant.

Qualitative Description of Values For k_d

k_d (ft/h)/(lb/ft^2)	Description
>10	Extremely Erodible
1 – 10	Very Erodible
0.1 – 1	Moderately Erodible
0.01 – 0.1	Moderately Resistant
0.001 – 0.01	Very Resistant
< 0.001	Extremely Resistant

Figure 78 Qualitative Description of Erodibility Index

The following summarizes the model runs shown in Table 31 - Summary of WindamB Model. All hours noted are from the start of the simulation.

Good vegetation cover delays the start of headcut initiation from 0.15 hours (shortly after overtopping initiates) to between 1.25 and 7.6 hours. No grass cover allows the headcut to start immediately upon overtopping (0.05 hours

From start of overtopping to initiation of breach ranged from 6.4 hours to 29.5 hours.

For the range of values examined – breach width ranges from 118 ft To 167 ft when there was a full breach to the toe of the levee. One run did not initiate breach and two runs initiated breach but did not completely fail the levee by downcutting to levee toe

Table 29 - Summary of WindamB Model

File	Kd	Undrained Shear Strength	PI	Maint. Code*	Vegetal Cover Code*	Part Diameter in inches	Vegetation Slope Failure Time hrs	Breach Initiation Time hrs	Breach Formation Time hrs	Final Breach Width Feet	Overtopping Depth feet	Breach
Dallaslow1kdBare	1	1250	na	na	na	na	0.15	6.4	13.10	168.5	2.1	Yes
Dallaslow1kdBare2	1	3000	na	na	na	na	0.15	18.30	33.15	153.1	3.88	Yes
Dallaslow01kdBare2	0.1	1250	na	na	na	Na	0.15	26.85	Na	54.22	3.88	Not a full breach
Dallaslow1kd	1	1250	36	2	0.9	0.04	7.64	10.5	16.10	152.1	3.88	Yes
Dallaslow1kdA	1	1250	36	3	0.9	0.04	1.25	6.7	13.2	166.7	2.14	Yes
Dallaslow1kdB	1	1250	36	2	0.5	0.04	7.64	10.5	16.1	152.1	3.88	Yes
Dallaslow1kdC	1	1250	36	2	0.9	0.1	7.64	10.5	16.1	152.1	3.88	Yes
Dallashi1kd	1	3000	71	2	0.9	0.04	7.64	19.95	34.35	117.8	3.88	Yes
Dallaslow01kd	0.1	1250	36	2	0.9	0.04	7.64	29.45	Na	47.7	3.88	Not a full breach
Dallashi01kd	0.1	300	71	2	0.9	0.04	7.64	Na	Na	6	3.88	Does not reach landward crest

Note all times are shown from start of the simulation at 0 hours
* Maintenance Code 2 indicates minor discontinuities in the vegetation -- Maintenance Code 3 indicates major discontinuities.

** Vegetal cover Code of 0.9 is a very uniform rooted grass like Bermuda or Centipede grass with more erosion resistance
Vegetal cover Code of 0.5 is for a less uniform rooted grass like alfalfa or Sudan grass and is easier to erode

Breach Characteristics

The breach parameters used for the hydraulic modeling of levee failures was heavily informed by the WinDAMB breach model results described above. The breach parameters applied for the analysis are detailed in Table 32 and Table 33 below.

Table 30. Breach Locations for Risk Analysis

Breach Location	HEC-RAS Model Lateral Structure	Lateral Structure Breach Station	Invert of Breach	Breach Initiation
East 74+00	113563	114440	400	At Peak Stage
East 220+00	127994	134750	400	At Peak Stage
East 311+00	136515	140590	405	At Peak Stage
East 410+00	146534	148000	405	At Peak Stage
West 10+00	110086	116100	400	At Peak Stage
West 188+00	127746	135100	400	At Peak Stage
West 250+00	134505	135100	400	At Peak Stage
West 335+00	143213	144300	405	At Peak Stage
East 230+00 Overtopping	129105	134750	400	At Peak Stage
East 5+00 Floodwall Overtopping	108348	114440	400	At Peak Stage
West 180+00 Overtopping	124434	135100	400	At Peak Stage

Table 31. Breach Widths and Formation Times for Risk Analysis

Hydrologic Load	Breach Width	Formation Time
½ Levee Loading	150	26
¾ Levee Loading	150	6
Threshold Levee Loading	150	6
Overtopping A	150	13
Overtopping B	150	13

Summary of Scenario Selection for Consequence Assessment

For the purpose of consequence modeling, 5 hydrologic loads were considered at 11 breach locations (8 related to seepage and piping and 3 related to overtopping). Both failure and non-failure conditions were considered for each of the scenarios. All inflow hydrographs were patterned after the 2007 flood event. A tabular summary of the 36 consequence scenarios is shown below in Table 34.

The hydraulic modeling for each of these scenarios provides the best estimate for flood depths and arrival times throughout the floodplain. These outputs from the hydraulic model were passed to the consequence model in order to estimate life loss and economic damages for each scenario. An example of the maximum depth and arrival time output is shown below in Figure 81 and Figure 82. Note that Table 24 lists the scenarios provided for the purposes of transient seepage analysis, which is a sub-set of the non-failure scenarios used for consequence assessment with the exception that only 1 hydrograph shape was used for the consequence assessment runs (2007 pattern).

Table 32. Summary of All Consequence Scenarios

Breach Location	½ Levee	¾ Levee	Threshold Levee Height	Overtopping A	Overtopping B
East 74+00	X	X	X	n/a	n/a
East 220+00	X	X	X	n/a	n/a
East 311+00	X	X	X	n/a	n/a
East 410+00	X	X	X	n/a	n/a
West 10+00	X	X	X	n/a	n/a
West 188+00	X	X	X	n/a	n/a
West 250+00	X	X	X	n/a	n/a
West 335+00	X	X	X	n/a	n/a
East 230+00 Overtopping	n/a	n/a	n/a	X	X
East 5+00 Floodwall Overtopping	n/a	n/a	n/a	X	X
West 180+00 Overtopping	n/a	n/a	n/a	X	X
Non-Breach	X	X	X	X	X

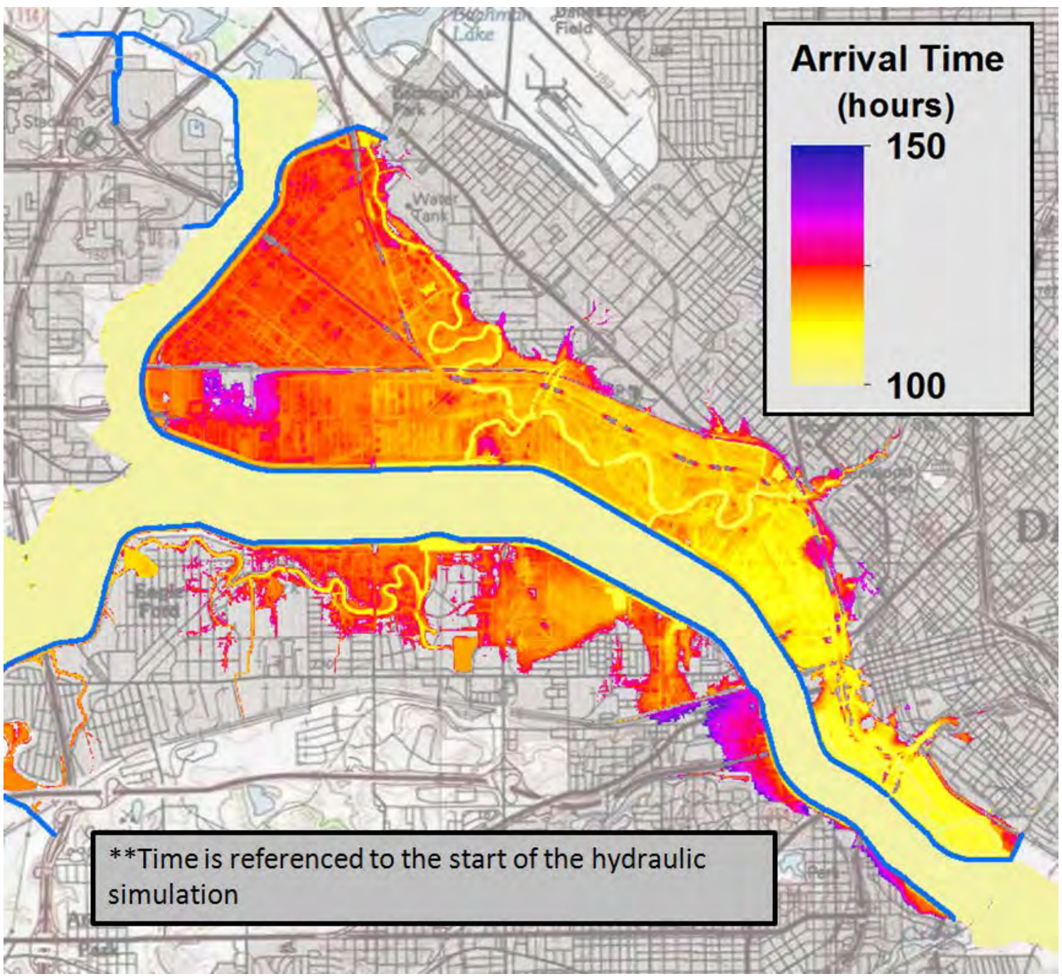


Figure 79. Example Arrival Time Output from Hydraulic Model

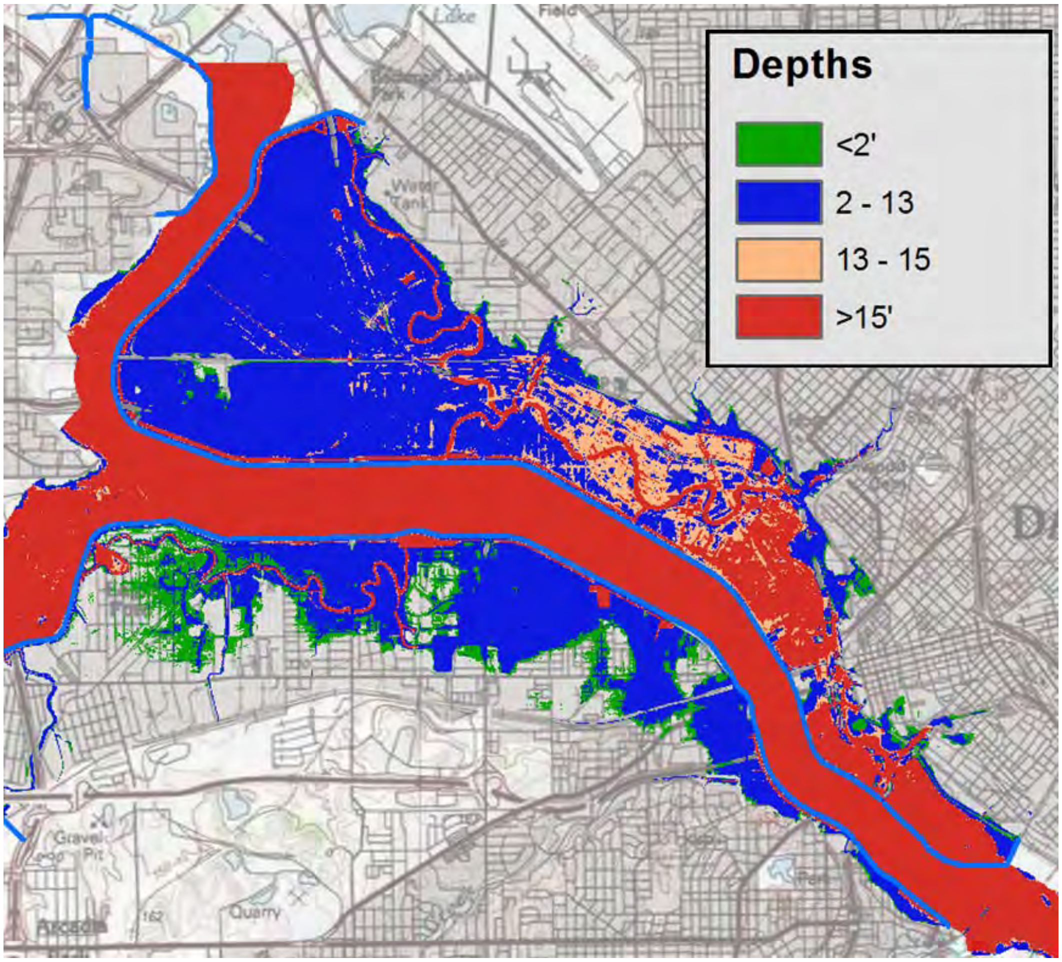


Figure 80. Example Maximum Depth Output from Hydraulic Model

References

1. US Army Corps of Engineers, Fort Worth District. Dallas Floodway Extension Feasibility Study, Appendix A. Hydrology and Hydraulics. – Draft, June 2011

2. US Army Corps of Engineers, Fort Worth District, Dallas Floodway Extension, General Reevaluation Report, Appendix A – Hydrologic and Hydraulic Analyses, circa 1992.

3. US Army Corps of Engineers, Omaha District. Digital Database of Large Rainfall Events.

4. USGS Gage Data for Trinity River at Dallas , TX. Gage #08057000. http://waterdata.usgs.gov/tx/nwis/nwisman/?site_no=08057000

Appendix E – Consequences

Introduction

For this analysis, impacts of 56 events are estimated under three different conditions for a total of 168 simulations. Six overtopping failures were considered, two separate hydrologic events at three possible locations each; the two hydrologic events non-failure simulations were also estimated in order to estimate the incremental consequences. Two geotechnical failure modes were considered, internal erosion and global instability. While the H&H inputs used for each of the two geotechnical failure modes were identical, warning issuance assumptions were varied. Each failure mode had 24 different simulations as there were eight locations considered along with three different hydrologic conditions. Best Case, Worst Case and Most Likely simulations were made for each event. The reported best estimate is the mean of a PERT distribution which uses the three scenarios as the Min, Max and Most Likely.

Within the study area, 9,000 residential, commercial, industrial and public structures are estimated to at risk of being inundated by either a West or East Overtopping failure event. The affected areas include the largely residential western leveed area and the predominately-commercial eastern leveed area. The western impact area contains roughly 6,350 structures with 19,600 Day Populations at Risk (PAR) and 23,500 Night PAR. The eastern impact area contains 2,650 structures with 91,400 Day PAR and 35,500 Night PAR. The combined study area totals to roughly 111,000 potential Day PAR and 59,000 potential Night PAR. Table 35 summarizes the potential consequences of the discussed events.

The two primary subject matters considered in this analysis are economic loss and the loss of life associated with each flood event, the latter being the dominant concern. The following sections cover in detail the estimates and methodology used for both categories. A map of the study area is shown below in Figure 83.

Table 33 - Best Estimates of Consequences

Failure Location	Nominal RAS Loading	Faulure Mode	Urban Damage	Structures Flooded	Day PAR	Night PAR	Loss of Life
East Station 410+00	1/2 Levee Height	Internal Erosion	$2,233,646,108	2,326	88,616	34,427	33
East Station 410+00	3/4 Levee Height	Internal Erosion	$2,772,199,130	2,459	90,423	35,311	433
East Station 410+00	Threshold	Internal Erosion	$2,948,052,629	2,536	92,277	35,981	242
East Station 310+00	1/2 Levee Height	Internal Erosion	$1,882,145,582	2,232	86,220	33,420	14
East Station 310+00	3/4 Levee Height	Internal Erosion	$2,527,921,695	2,405	89,305	34,612	349
East Station 310+00	Threshold	Internal Erosion	$2,777,496,244	2,461	90,427	35,314	198
East Station 222+00	1/2 Levee Height	Internal Erosion	$1,575,889,383	2,185	85,491	33,071	7
East Station 222+00	3/4 Levee Height	Internal Erosion	$2,335,599,422	2,358	89,001	34,550	505
East Station 222+00	Threshold	Internal Erosion	$2,627,211,508	2,418	89,590	34,717	171
East Staton 74+00	1/2 Levee Height	Internal Erosion	$867,221,180	1,751	74,068	32,111	1
East Staton 74+00	3/4 Levee Height	Internal Erosion	$1,785,811,163	2,239	86,327	33,442	42
East Staton 74+00	Threshold	Internal Erosion	$2,111,325,444	2,297	88,182	34,308	30
West Station 335+00	1/2 Levee Height	Internal Erosion	$244,959,037	5,017	15,268	20,296	55
West Station 335+00	3/4 Levee Height	Internal Erosion	$454,173,160	6,061	20,462	24,208	489
West Station 335+00	Threshold	Internal Erosion	$532,131,936	6,265	21,020	24,773	394
West Station 250+00	1/2 Levee Height	Internal Erosion	$193,795,758	4,145	13,453	18,148	37
West Station 250+00	3/4 Levee Height	Internal Erosion	$412,632,903	5,924	18,679	22,827	523
West Station 250+00	Threshold	Internal Erosion	$499,727,636	6,208	20,760	24,516	350
West Station 188+00	1/2 Levee Height	Internal Erosion	$186,367,297	3,637	11,971	16,124	24
West Station 188+00	3/4 Levee Height	Internal Erosion	$431,999,069	5,824	18,193	22,545	431
West Station 188+00	Threshold	Internal Erosion	$534,654,230	6,137	20,673	24,410	298
West Station 10+00	1/2 Levee Height	Internal Erosion	$36,260,440	406	2,870	2,004	5
West Station 10+00	3/4 Levee Height	Internal Erosion	$180,170,730	3,170	10,556	13,184	37
West Station 10+00	Threshold	Internal Erosion	$267,443,292	4,517	14,721	19,184	65
East Station 410+00	1/2 Levee Height	Global Instability	$2,233,646,108	2,326	88,616	34,427	33
East Station 410+00	3/4 Levee Height	Global Instability	$2,772,199,130	2,459	90,423	35,311	182
East Station 410+00	Threshold	Global Instability	$2,948,052,629	2,536	92,277	35,981	238
East Station 310+00	1/2 Levee Height	Global Instability	$1,882,145,582	2,232	86,220	33,420	13
East Station 310+00	3/4 Levee Height	Global Instability	$2,527,921,695	2,405	89,305	34,612	109
East Station 310+00	Threshold	Global Instability	$2,777,496,244	2,461	90,427	35,314	198
East Station 222+00	1/2 Levee Height	Global Instability	$1,575,889,383	2,185	85,491	33,071	6
East Station 222+00	3/4 Levee Height	Global Instability	$2,335,599,422	2,358	89,001	34,550	87
East Station 222+00	Threshold	Global Instability	$2,627,211,508	2,418	89,590	34,717	170
East Staton 74+00	1/2 Levee Height	Global Instability	$867,221,180	1,751	74,068	32,111	1
East Staton 74+00	3/4 Levee Height	Global Instability	$1,785,811,163	2,239	86,327	33,442	13
East Staton 74+00	Threshold	Global Instability	$2,111,325,444	2,297	88,182	34,308	30
West Station 335+00	1/2 Levee Height	Global Instability	$244,959,037	5,017	15,268	20,296	54
West Station 335+00	3/4 Levee Height	Global Instability	$454,173,160	6,061	20,462	24,208	252
West Station 335+00	Threshold	Global Instability	$532,131,936	6,265	21,020	24,773	393
West Station 250+00	1/2 Levee Height	Global Instability	$193,795,758	4,145	13,453	18,148	33
West Station 250+00	3/4 Levee Height	Global Instability	$412,632,903	5,924	18,679	22,827	242
West Station 250+00	Threshold	Global Instability	$499,727,636	6,208	20,760	24,516	349
West Station 188+00	1/2 Levee Height	Global Instability	$186,367,297	3,637	11,971	16,124	21
West Station 188+00	3/4 Levee Height	Global Instability	$431,999,069	5,824	18,193	22,545	198
West Station 188+00	Threshold	Global Instability	$534,654,230	6,137	20,673	24,410	298
West Station 10+00	1/2 Levee Height	Global Instability	$36,260,440	406	2,870	2,004	2
West Station 10+00	3/4 Levee Height	Global Instability	$180,170,730	3,170	10,556	13,184	19
West Station 10+00	Threshold	Global Instability	$267,443,292	4,517	14,721	19,184	65
East Levee Breach	Overtop A	Overtopping	$3,253,502,908	2,515	90,807	35,525	192
East Wall Breach	Overtop A	Overtopping	$2,592,131,706	2,372	89,817	35,114	39
West Levee Breach	Overtop A	Overtopping	$821,406,019	6,951	36,272	26,472	320
East Levee Breach	Overtop B	Overtopping	$3,598,060,722	2,734	109,240	36,636	311
East Wall Breach	Overtop B	Overtopping	$3,359,411,536	5,647	102,462	49,760	176
West Levee Breach	Overtop B	Overtopping	$1,666,691,448	8,100	94,933	56,996	562
No Failure / No Breach	Overtop A	N/A	$427,658,613	989	52,708	30,781	0
No Failure / No Breach	Overtop B	N/A	$2,565,076,304	5,515	99,045	48,508	37

Figure 81 Map of Study Area

HEC-FIA Consequence Analysis

The Hydrologic Engineering Center's Flood Impact Analysis software (HEC-FIA) was used to estimate consequences associated with potential failures of Dallas Levee System. The life loss methodology in HEC-FIA is based on the LifeSim methodology developed by Utah State University's Institute for Dam Safety Risk Management. The process of computing loss of life within FIA is to identify the population at risk from a given event and then divide this PAR into those cleared from the danger area, those caught evacuating, and those not mobilized. This division is based on a host of factors, including time warned relative to the flood wave arrival time, mobilization, and distance to a safe zone. Those who do not escape the hazard area are subjected to fatality rates that are a function of evacuation status, water surface elevation, foundation height, structure height, and whether the PAR is elderly. This process will be described in greater detail below. The consequence data presented in this report were generated using HEC-FIA version 2.1 Beta, dated 13 December 2010.

Failure Scenarios

For this analysis, impacts of 56 events are estimated under three different conditions for a total of 168 simulations. Six overtopping failures were considered, two separate hydrologic events at three possible locations each; the two hydrologic events non-failure simulations were also estimated in order to estimate the incremental consequences. Two geotechnical failure modes were considered, internal erosion and global instability. While the H&H inputs used for each of the two geotechnical failure modes were identical, warning issuance assumptions were varied. Each failure mode had 24 different simulations as there were eight locations considered along with three different hydrologic conditions. Best Case, Worst Case and Most Likely simulations were made for each event.

Populations at Risk

The population at risk is comprised of those people within the inundated area for a given scenario. FEMA's Hazards U.S. Multi-Hazard model (HAZUS-MH) database includes a structure inventory accurate to the census block level as well as population counts and other associated census data. This is input into FIA, which then places structures within the assigned census block using an even distribution. The structure nodes created are assigned a ground elevation from an underlying digital terrain model (DTM). Population is assigned to structures based on the HAZUS-MH data. The distribution of the population at risk between the structures in a census block is assumed to vary from night to day.

Because census blocks can often be very large, FIA's random distribution of structures may bias the results. To correct for this, aerial imagery was imported into FIA. Once done, it is possible to manually "drag and drop" structures to better approximate their correct locations. This adjustment was especially necessary in census blocks with extreme shifts in elevations and depth, such as those near the stream bank.

Parcel data was supplied by the Fort Worth, which greatly refined the quality of the structure inventory. This data, from 2010, was used to create parcel centroids for structure placement.

Fields from the parcel data also greatly refined assumptions on structure counts, number of housing units, structure values and structure categories. When the data for certain outlying areas was unavailable, standard HAZUS data was used.

While HAZUS population is from 2000 census, 2010 census data was used to index PAR counts as much as possible. The west levee area was indexed by the change in total population within the area. The east levee area, which is more commercial, was indexed by the change in population for the city of Dallas.

Foundation Heights

Foundation heights may have a limited impact on loss of life at a structure, as they are a component of determining the relevant depth at the structure. Foundation heights are input relative to a structure's ground elevation, as determined from the digital terrain model. To determine foundation heights Google Earth's Street View feature was used to survey severely impacted areas of residential, industrial, and commercial structures. The use of mean foundation height does not account for variation. Ultimately, a generic assumption of 1-foot foundation heights for each structure was made. Sensitivity results suggested minimal impact, but future studies could reduce uncertainty by completing a detailed survey that added foundation height information to the available parcel data.

Warning Issuance Time

Time windows are input in HEC-FIA which define beginning and end dates for the simulation, the time step used in the H&H model (in this case 15 minutes), the time when the breach occurs for each scenario, and the warning issuance time relative to breach initiation. Warning issuance is a critical factor in determining the percentage of PAR that is able to mobilize in response to a levee failure and subsequently evacuate the hazard area. For this study, an Expert Elicitation was held to estimate the probable range of warning issuance times for the best estimate of when an evacuation advisory would be issued, how long it would take the local Emergency Management Agencies (EMA) to begin issuing warnings and finally when the first warning to the public would go out relative to breach.

Figure 84 illustrates a warning issuance process for an instantaneous event – it takes time for site personnel to contact EMAs and for those agencies to begin issuing warnings to the public. However, there is tremendous uncertainty regarding when a warning would be issued for the analyzed failure modes.

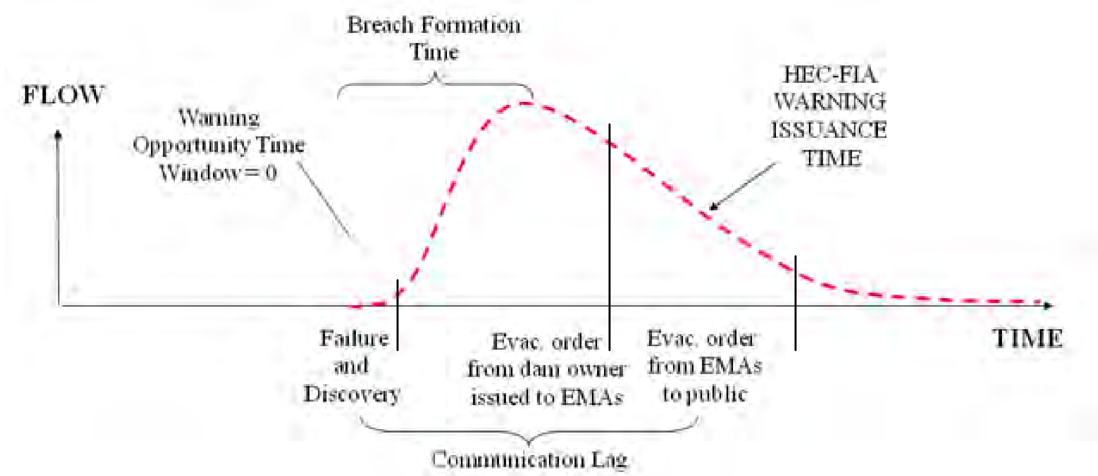


Figure 82 Warning Issuance Process

Factors related to likely warning issuance at a given load level include: probable surveillance at the site, likely opportunity between acknowledgement of major problem and failure, speed at which problem would develop, and degree to which "communication lag" between parties might be mitigated. These factors were assessed for significant failure modes and were used to inform the warning issuance time within FIA.

In extreme events, interior flooding is also possible, leading to small-scale evacuations before levee failure is an issue. As a result, during the levee breach scenario, a percentage of PAR would have already evacuated in response to this advance warning, and would not be at risk from the levee breach flood. To capture this, two evacuations stages were used, with 10% of the PAR assumed to be evacuated prior to failure.

The most relevant warning and evacuation process, however, is related to the failure warning. For overtopping and near overtopping events, significant warning is expected. The results from the EOE led to an assumption that site personal would advise evacuation 8 hours prior to breach. Global Instability failure modes were also assumed to be slowly developing modes leading to 8 hours of advanced warning from the site. Meanwhile, for Internal Erosion scenarios at 75% loading or below, warning after breach was anticipated. This is because it is likely that no signs of warning would present themselves until breach; exit to the sumps is likely, and this area would already be rising due to interior flooding. The range of likely warning advisory from the site, in hours, relative to breach are shown in Table 36.

Table 34 - Warning Advisory from Site Relative to Breach in Hours

Failure Mode	Nominal RAS Loading	Low	Most Likely	High
Internal Erosion	1/2 Levee Height	-3	0	0
	3/4 Levee Height	-3	0	0
	Threshold	0	8	12
Global Instability	1/2 Levee Height	0	8	12
	3/4 Levee Height	0	8	12
	Threshold	0	8	12
Overtopping	Overtopping A	0	8	12
	Overtopping B	0	8	12

Once site personnel advise evacuation, time is needed for EMAs and city leaders to issue an evacuation order. The best estimate for this delay is assumed to be 45 minutes, but the probable range is from 30 minutes to 60 minutes. By subtracting this delay from the site warning, we obtain the range of warning issuance times to the public shown in Table 37.

Table 35 -- Warning Issuance to Public Relative to Breach in Hours

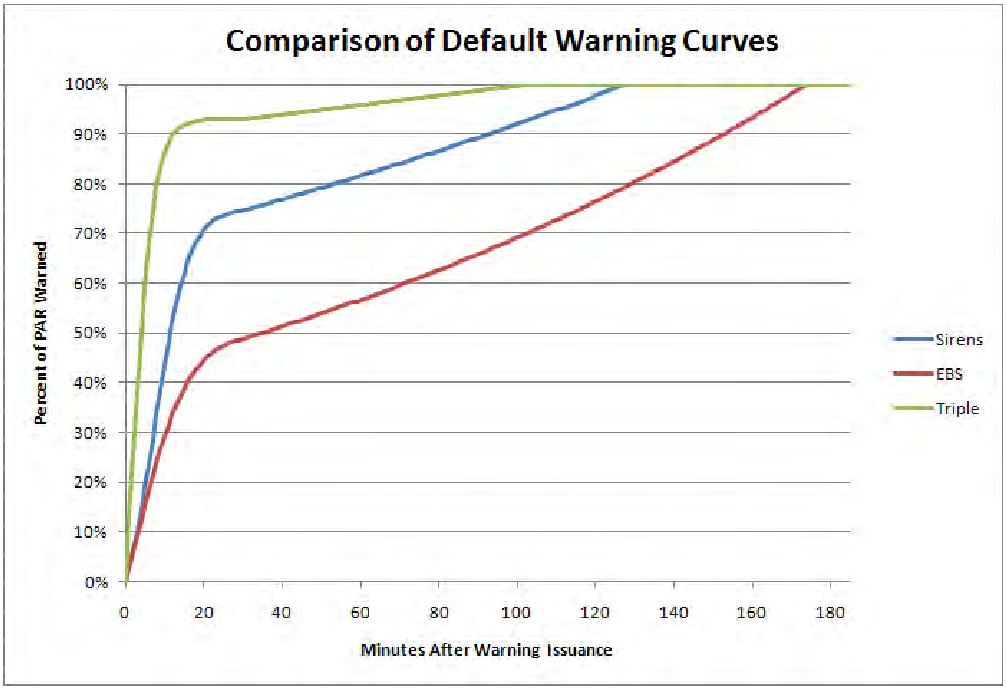
Failure Mode	Nominal RAS Loading	Low	Most Likely	High
Internal Erosion	1/2 Levee Height	-4	-0.75	-0.5
	3/4 Levee Height	-4	-0.75	-0.5
	Threshold	-1	7.25	11.5
Global Instability	1/2 Levee Height	-1	7.25	11.5
	3/4 Levee Height	-1	7.25	11.5
	Threshold	-1	7.25	11.5
Overtopping	Overtopping A	-1	7.25	11.5
	Overtopping B	-1	7.25	11.5

Again, Internal Erosion scenarios have higher loss of life than Global Instability cases due to less warning opportunity time. It is important to note that for Internal Erosion scenarios, there is a decrease in Loss of Life as loading moves from “3/4 Height” to “Threshold”, or full loading. This is because when freeboard becomes an issue, overtopping concerns control the warning issuance assumption. Meaning, more advanced warning would be available for these extreme events. This increase in warning more than offsets the increased danger due to higher eventual depths.

Warning and Mobilization Curves

Warning issuance and mobilization curves determine for each scenario the percentage of PAR that mobilizes and those that do not. As residents become warned, they begin the mobilization process. Residents cannot begin mobilizing until they are first warned. Within HEC-FIA, a warning system with an associated warning effectiveness curve and a mobilization curve are defined. These curves assume how quickly and what percentages of the population at risk will receive warning of a hazard, and how quickly they will mobilize after receiving the warning. Warning curves are based on six possible warning systems; sirens, tone-alert radios, auto-dial telephones, emergency broadcast system (EBS), EBS and tone-alert radios, and EBS, sirens, and auto-dial telephones. Shown below in are three separate warning diffusion curves.

Figure 83 Warning Rate Curve



Impact areas have access to auto-dial telephone system and warning would also rely on mobile sirens, media and in some cases door-to-door notification. However, due to successful hourly call volume of the auto-dial system being significantly lower than the likely PAR, it is not anticipated that the auto-dial system will be a driver in the warning process. Furthermore, the warning curves themselves are decades old and do not lend themselves to modification with site-specific data. Instead of using the very aggressive auto-dials system, it was deemed appropriate to use the Siren curve as a proxy for residential area. The siren curve is moderately effective and assumes 75% of the PAR would be warned after 30 minutes. For commercial areas, the tone-alert radio system was used as a proxy. The Tone-alert system is an above average curve that assumes 90% of the PAR warned at 30 minutes. The relative advantage of the commercial proxy is justified because warning contagion is likely much greater in these areas (co-workers and those in urban areas are more likely to warn each other and may do so more quickly than general residential PAR).

In scenarios where there is significant advanced warning, such as the overtopping scenarios modeled in the most likely condition, the majority of the population can be expected to evacuate. Given sufficient depths to result in fatalities, the driver becomes the size of the minority who are physically incapable or otherwise unwilling to evacuate before arrival of floodwaters.

The maximum percent of the PAR from a zone that will attempt evacuation provided sufficient time is known as the “max mobilization rate”. While there is considerable uncertainty as to what the mobilization rate would be for a given hazard, the hazards literature suggests 95% as a useful average of community wide evacuation rates for a preventive evacuation due to a forecasted levee failure[8]. To account for site-specific variance from this rate, several different impact areas were constructed.

For the primarily residential population behind the West Levee, an index was created to weigh various factors against each other. While there is uncertainty around relative significance, variables used in this index have generally been shown in the literature to be correlated with evacuation status[9]. Example demographic variables include percentage of elderly households, percentage of households below 150% of the poverty line, and percentage of households without vehicles. Non-demographic variables were also used to weigh site specific factors that may have an impact on risk perceptions; examples of such variables include the average distance from the levee, presence of environmental cues (extreme weather), and quality of warning message. Ultimately, while an evacuation rate of 94.5% for the most extreme hydrologic conditions and 94% for less extreme (no threat of overtopping) were used in these residential zones in the west levee.

The index uses 18 different variables, with most tied to census data. Each county's standard deviation from the national county mean is used to determine how bad or good things are in a given community versus a "typical" community considered to be represented by the default curve. Block Group data for the leveed area was compared to this collection of national data. The remaining variables are community specific and attempt to account for geographic and institutional factors that may influence decisions to mobilize at various moments (a variable may be significant in the first hour of mobilization but not necessarily 8 hours after the warning was received, etc).

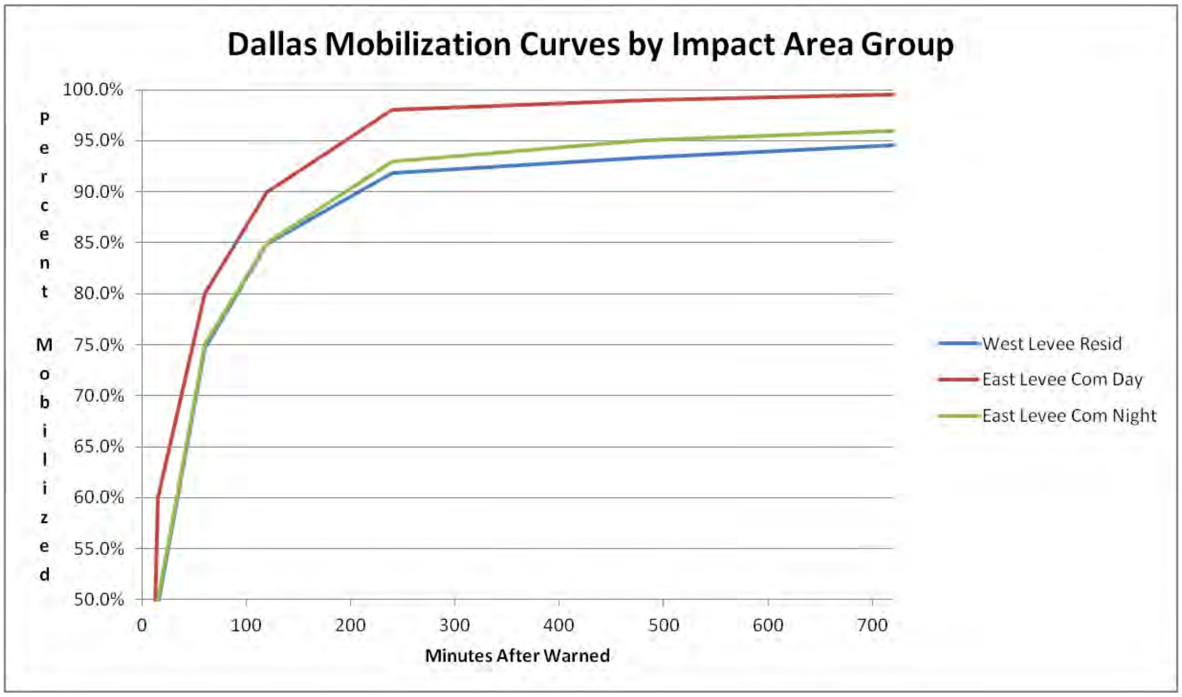
The census based variables are as follows: population age 17 and younger, population over 65, population living in group quarters, households with single occupant, households linguistically isolated, population density, mean travel time to work, population taking group transit to work, percent of population with no diploma, population below 150% of the poverty line, and disabled population. Meanwhile, the event specific variables are as follows: distance to evacuate, quality of warning message, prolonged detected failure, level of community awareness, and severe rainfall event.

The commercial zones behind the east levee did not lend themselves to a similar method. Instead, a likely aggregate maximum evacuation rate was estimated by assuming the vast majority of commercial workers would be willing and able to evacuate, but only the standard 95% rate of other categories of PAR would be willing and able to evacuate. The resulting aggregated max mobilization rate for predominately-commercial areas is 99.5% during the day and 96% at night. A mainly residential zone behind the east levee used a 95% mobilization rate for both day and night.

[8] Jonkman, Sebastian Nicolaas. *Loss of Life Estimation in Flood Risk Assessment: Theory and Application*. 2007.
[9] Mileti, Dennis and Sorenson, John. *Communication of Emergency Public Warnings: A Social Science Perspective and State-of-the-Art Assessment*. 1990.

Slight differences between curves can lead to dramatic changes in Loss of Life results, particularly when considering the tail end of the curve. For example, given sufficient warning, a max mobilization rate of 97% would have three times as many people left behind as a max mobilization rate of 99%.

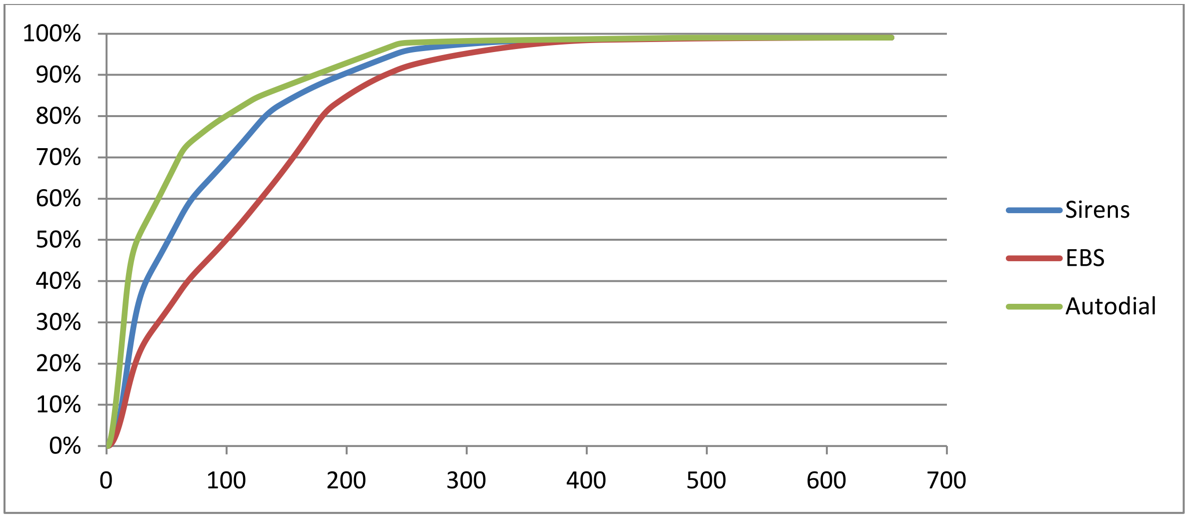
Figure 84 Mobilization Curve



These vulnerable populations help explain why the curves never reach 100% mobilized. A small minority will always lack the means to escape from the hazard. It is likely that those remaining will be a mix of elderly (many in nursing homes), institutionalized PAR, and the impoverished. Some PAR might not personalize the threat until water begins to rise and they become trapped. In addition, among those who both have the physical means to escape and knowledge of the threat, there will be those who choose to remain behind. Such PAR may do so to protect their property or because of a fatalistic outlook on life or any number of personal reasons. Even during a historic flood, it is likely that some percentage of PAR will remain when the flood wave arrives. This is supported by a number of case studies on natural disasters.

Combined "warned and mobilized" curves are shown below in Figure 87. FIA does not use cumulative probability curves to assign structures as either "warned" or "not warned". Nor does it use cumulative probability curves to assign structures as either "mobilized" or "not mobilized". Instead, FIA uses an expected value approach, wherein an increasing number of fractions of a structure's PAR begin the "warned and mobilized" process. The PAR at a structure continues to mobilize until either, 1) the water surface elevation at the structure reaches 2 feet, 2) the structure reaches the max mobilization limit, or 3) the flood wave arrives at their evacuation point.

Figure 85 Warned and Mobilized Curves by Warning System



Hazard Areas and Evacuation

Once the population at risk is determined and warning issuance and mobilization rates are established, the process of evacuating PAR out of the hazard area must be modeled. The hazard area is defined as the boundary within which depth of inundation is greater than 2 feet. Hazard area boundaries were generated from inundation grids using ArcMap. To generate these, depth grids for each event were reclassified by depth to separate flooding greater than and less than 2 feet. The resulting grids were converted to polygon layers, and segments with a depth of two or less were removed.

A straight-line evacuation route is then calculated within FIA for each structure to the nearest "safe zone", or the shortest distance to exit the hazard area boundary. This distance was then doubled to obtain an estimated effective travel distance. An average evacuation speed of 10 miles per hour is assumed for the best estimate. By dividing the evacuation distance by evacuation velocity we can obtain the time needed to successfully evacuate. It was assumed that the PAR that leaves their structure between the time of flood arrival and the time needed to evacuate would be caught evacuating.

Evacuation assumptions had negligible impact on scenarios with significant advanced warning and were most relevant for worst-case scenarios and Internal erosion scenarios.

Fatality Rates

Fatality rates determine the percentages of those mobilized and caught, and those not mobilized that are assumed as fatalities. Population at risk cleared, or successfully mobilized and evacuated have a zero percent fatality rate. Those who are mobilized but caught during evacuation have a 91% fatality rate. For those who do not mobilize, fatality rates are based on the maximum inundation depth at the structure. If the depth is less than 2 feet or if it is less than the structures foundation height, the fatality rate is assumed zero percent, as with "cleared" PAR. For single-story structures, if the depth is less than 13 feet above the structure's foundation height, the fatality rate is 0.02%, i.e. 99.98% of PAR in the structure would survive. If the depth is greater than 13 feet and less than 15 feet above the foundation height, what is considered the "compromised zone", the fatality rate is 12%. At any higher depth, the fatality rate is equivalent to those caught evacuating, 91%. For multistory structures, 9 feet are added to these increments for each additional story. All fatality rates used by FIA are based on historical data[10]. Number of stories data was available for parcel data.

One last assumption relevant to fatality thresholds regards whether the PAR trapped in a structure would be able to access an attic or roof or whether there might be reason to believe certain PAR would have lower fatality thresholds than the majority. No adjustment in the current version of FIA is made for those over 65 or disabled PAR, however FIA 2.2 will incorporate a modification which allows this PAR to have reduced fatality thresholds. To account for varying levels in the PAR's ability to vertically evacuate, a post-FIA adjustment was made to change the fatality thresholds for this PAR. Assignment was based on parcel data fields that indicate the particular structures with elderly or disabled status (property tax-exemption).

Results – Loss of Life

The risk of lost lives is the primary consequence concern in this risk analysis. In the event of a failure of the Dallas Levee System, significant flooding velocity would occur in heavily populated areas, likely leading to numerous fatalities. Table 38 displays the loss of life consequences; calculations were made using the adjustments described above, for a complete list of major assumptions used, refer to Table 39 (found at the end of the sensitivity section). Because of a wide range of possible results, the best estimate takes the mean of a distribution of results. Values do not reflect any particular simulation's actual output, but instead a weighted average of multiple simulations.

[10] McClelland, D.M., and D.S. Bowles. (2002). 'Estimating Life Loss for Dam Safety Risk Assessment - a Review and New Approach.' Institute for Water Resources, U.S. Army Corps of Engineers, Alexandria, VA.

Table 36 - Total Life Loss

Failure Location	Nominal RAS Loading	Faulure Mode	Loss of Life
East Station 410+00	1/2 Levee Height	Internal Erosion	33
East Station 410+00	3/4 Levee Height	Internal Erosion	433
East Station 410+00	Threshold	Internal Erosion	242
East Station 310+00	1/2 Levee Height	Internal Erosion	14
East Station 310+00	3/4 Levee Height	Internal Erosion	349
East Station 310+00	Threshold	Internal Erosion	198
East Station 222+00	1/2 Levee Height	Internal Erosion	7
East Station 222+00	3/4 Levee Height	Internal Erosion	505
East Station 222+00	Threshold	Internal Erosion	171
East Staton 74+00	1/2 Levee Height	Internal Erosion	1
East Staton 74+00	3/4 Levee Height	Internal Erosion	42
East Staton 74+00	Threshold	Internal Erosion	30
West Station 335+00	1/2 Levee Height	Internal Erosion	55
West Station 335+00	3/4 Levee Height	Internal Erosion	489
West Station 335+00	Threshold	Internal Erosion	394
West Station 250+00	1/2 Levee Height	Internal Erosion	37
West Station 250+00	3/4 Levee Height	Internal Erosion	523
West Station 250+00	Threshold	Internal Erosion	350
West Station 188+00	1/2 Levee Height	Internal Erosion	24
West Station 188+00	3/4 Levee Height	Internal Erosion	431
West Station 188+00	Threshold	Internal Erosion	298
West Station 10+00	1/2 Levee Height	Internal Erosion	5
West Station 10+00	3/4 Levee Height	Internal Erosion	37
West Station 10+00	Threshold	Internal Erosion	65
East Station 410+00	1/2 Levee Height	Global Instability	33
East Station 410+00	3/4 Levee Height	Global Instability	182
East Station 410+00	Threshold	Global Instability	238
East Station 310+00	1/2 Levee Height	Global Instability	13
East Station 310+00	3/4 Levee Height	Global Instability	109
East Station 310+00	Threshold	Global Instability	198
East Station 222+00	1/2 Levee Height	Global Instability	6
East Station 222+00	3/4 Levee Height	Global Instability	87
East Station 222+00	Threshold	Global Instability	170
East Staton 74+00	1/2 Levee Height	Global Instability	1
East Staton 74+00	3/4 Levee Height	Global Instability	13
East Staton 74+00	Threshold	Global Instability	30
West Station 335+00	1/2 Levee Height	Global Instability	54
West Station 335+00	3/4 Levee Height	Global Instability	252
West Station 335+00	Threshold	Global Instability	393
West Station 250+00	1/2 Levee Height	Global Instability	33
West Station 250+00	3/4 Levee Height	Global Instability	242
West Station 250+00	Threshold	Global Instability	349
West Station 188+00	1/2 Levee Height	Global Instability	21
West Station 188+00	3/4 Levee Height	Global Instability	198
West Station 188+00	Threshold	Global Instability	298
West Station 10+00	1/2 Levee Height	Global Instability	2
West Station 10+00	3/4 Levee Height	Global Instability	19
West Station 10+00	Threshold	Global Instability	65
East Levee Breach	Overtop A	Overtopping	192
East Wall Breach	Overtop A	Overtopping	39
West Levee Breach	Overtop A	Overtopping	320
East Levee Breach	Overtop B	Overtopping	311
East Wall Breach	Overtop B	Overtopping	176
West Levee Breach	Overtop B	Overtopping	562
No Failure / No Breach	Overtop A	N/A	0
No Failure / No Breach	Overtop B	N/A	37

Results – Assumptions Sensitivity

Estimates for the provided results use the mean, but the most likely condition is large factor of the distribution. However, due to the uncertainty surrounding these assumptions, the below discussions and outputs are provided to the reader to indicate which assumptions have significant impacts on the results and to suggest a range of possible impacts following a breach at Dallas Levee System. Due to the large number of permutations possible, discussed assumptions were typically varied while holding other assumptions equal to those reported above in the main section – the most likely values.

Warning Issuance Time

Warning issuance assumptions often have a large impact on the results. In addition, due to the unpredictability of the response to a potential breach, there is tremendous uncertainty related to when a warning is likely to be issued. While the utilized issuance times reflect the cadre's best estimate, based on historical evidence and site-specific information, the figures below provide results based on a range of possible inputs.

Figure 86 -- Loss of Life for Varying Warning Issuance Assumptions – E222 75%

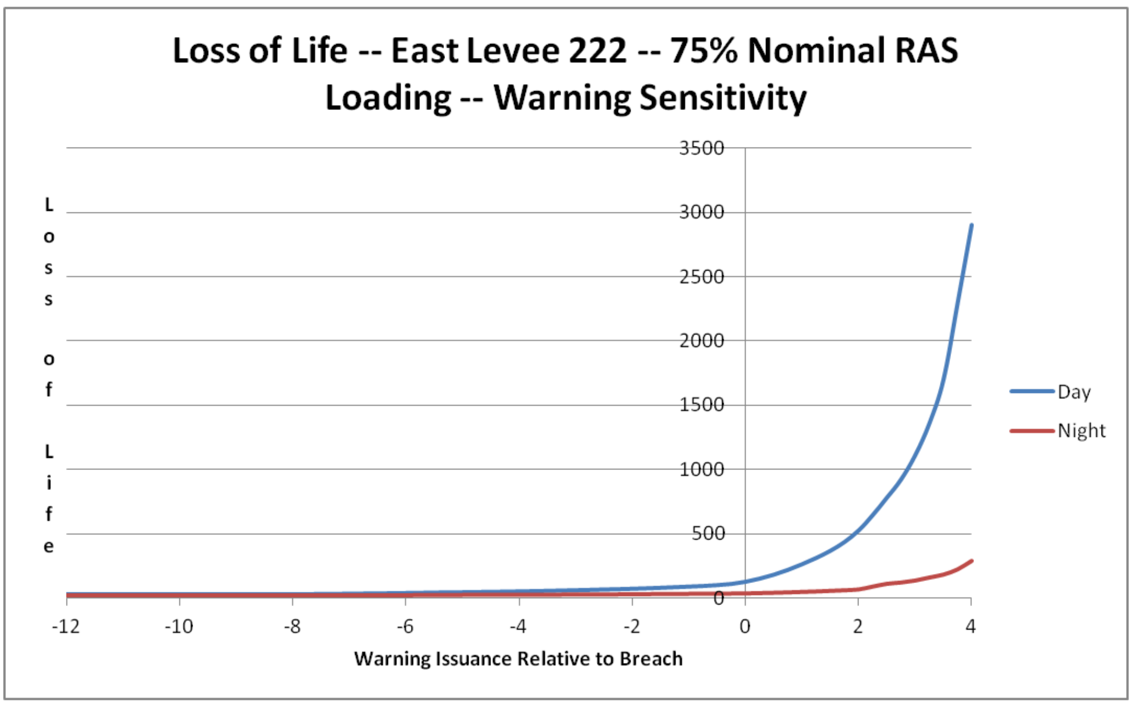


Figure 88 shows that results are relatively insensitive to the warning issuance assumption unless warning goes out after breach. This suggests that Global Instability results, which have significant warning opportunity time, are not sensitive to this assumption. Meanwhile, results for Internal Erosion failures, which are not expected to have advanced warning, are significantly impacted by this assumption. The large difference between day and night results is both a factor of the gross difference in PAR from Day to Night, but also the spatial difference in PAR – Day

PAR is disproportionately found in low-lying areas and night PAR is disproportionately found in hi-rises.

Figure 87 -- Loss of Life for Varying Warning Issuance Assumptions – E222 50%

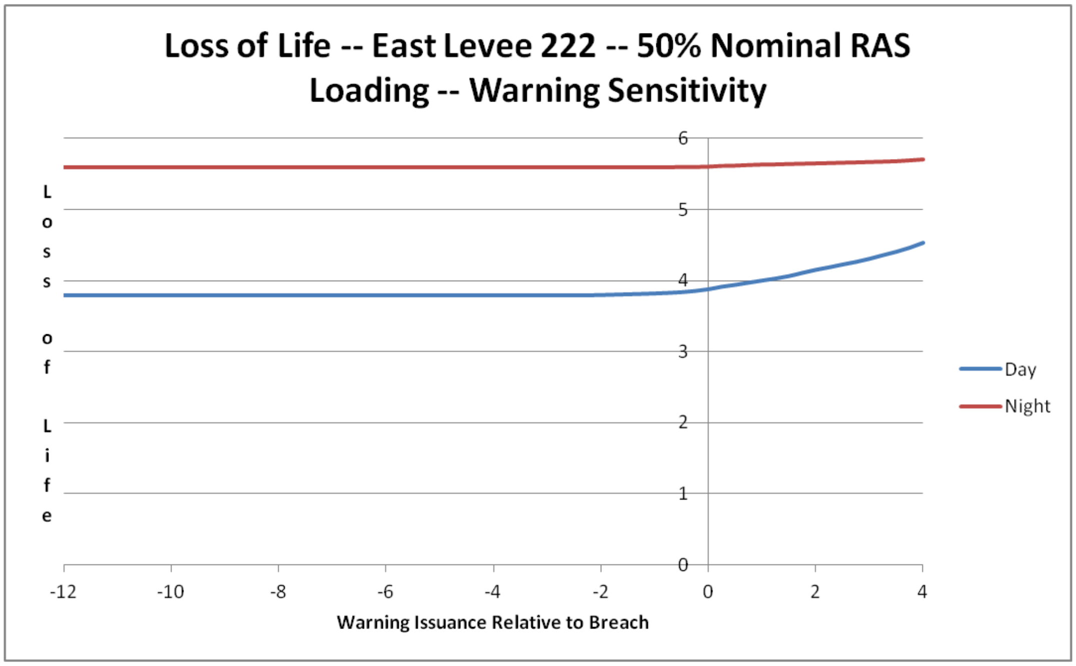


Figure 89 shows that less severe events are particularly indifferent to warning issuance assumptions. This is factor of the vast amount of time it takes for the breach to widen and the leveed area to fill.

Figure 88 -- Loss of Life for Varying Warning Issuance Assumptions – W335 75%

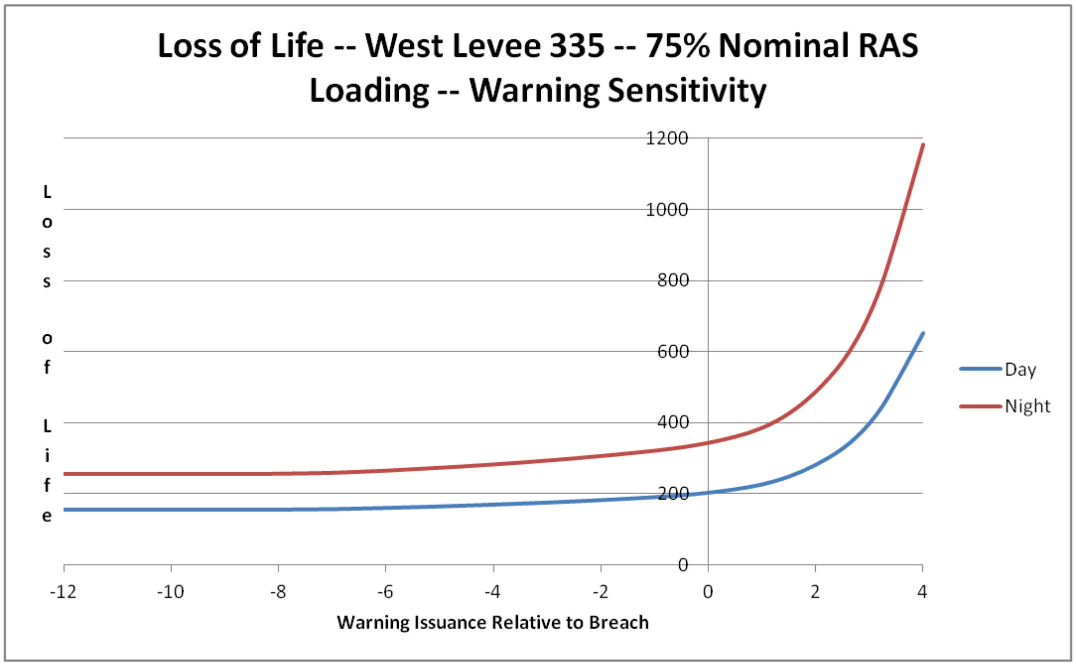


Figure 90 is similar to Figure 88 in that results are relatively insensitive until after breach. This again highlights that the rate of rise is relatively slow, and that even if a warning went out shortly before breach, the majority of PAR will be able to evacuate safely. However, in this case, Day and Night remain in the same order of magnitude. This is because, for this primarily residential impact area, differences in day and night PAR are less severe.

Figure 89 -- Loss of Life for Varying Warning Issuance Assumptions – West Overtopping B

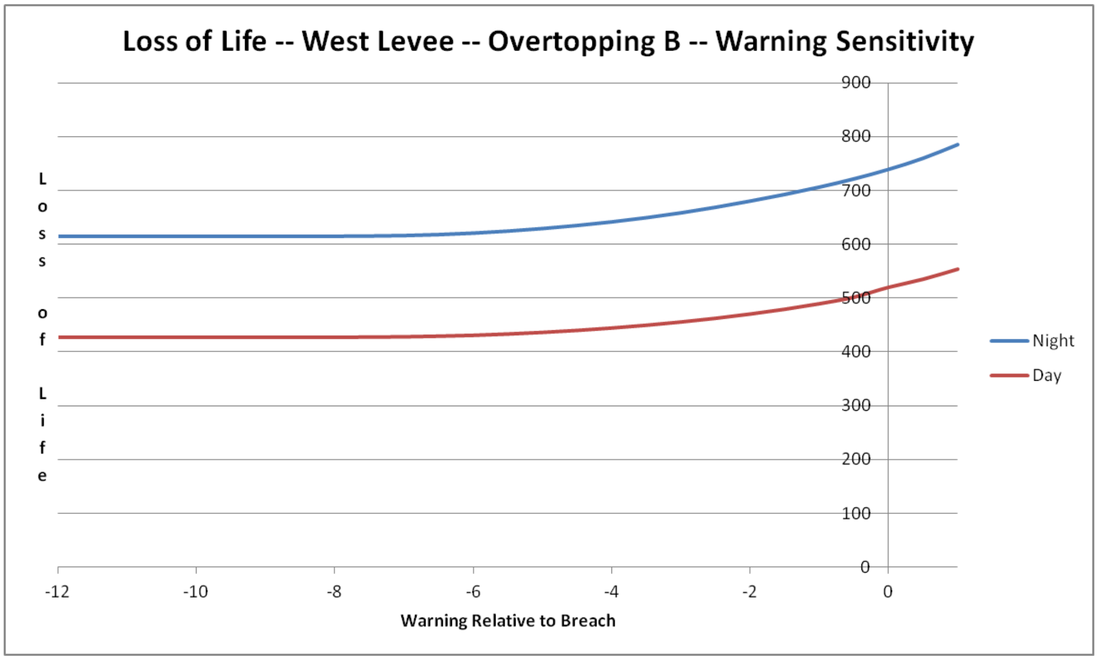
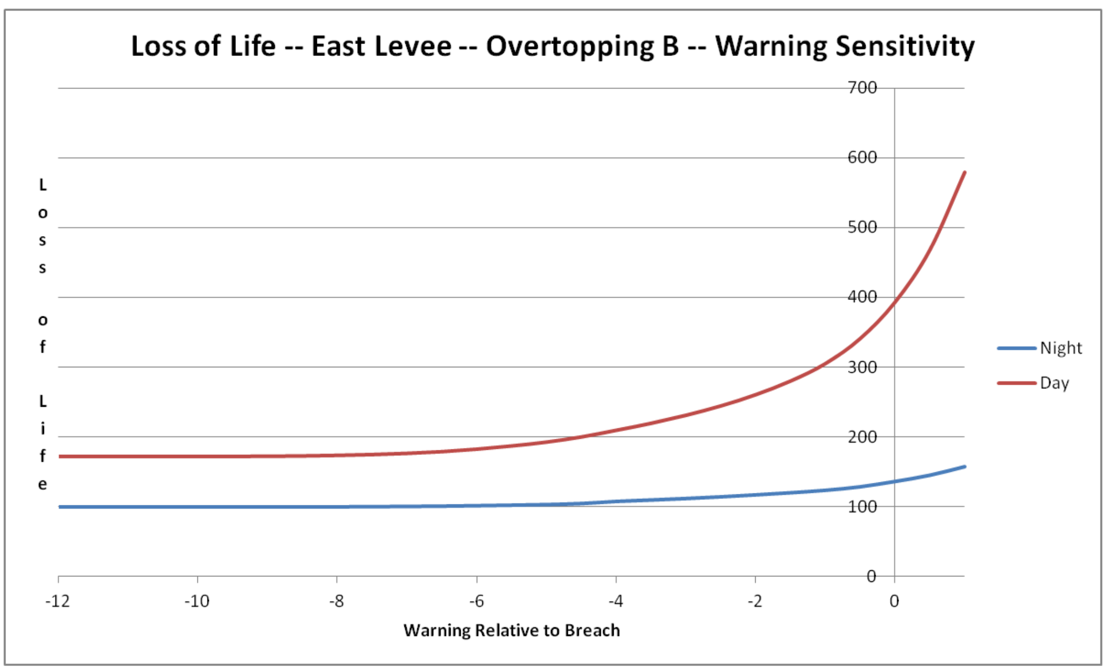


Figure 90 -- Loss of Life for Varying Warning Issuance Assumptions – East Overtopping B



When Figure 91 and Figure 92 are compared with earlier figures, it may appear to the eye that Overtopping events are less sensitive to warning issuance than the geotechnical failures. However, the main difference is that the range of likely warning issuance times does not extend as far after breach as the earlier events (which include Internal Erosion scenarios). The relative insensitivity of overtopping results to warning issuance assumptions, when compared with the relative certainty that significant warning would be available, emphasizes the importance of the mobilization assumption; with a few hours warning, the max mobilization assumption becomes the constraint on reducing Loss of Life.

Population Mobilized

As with the warning issuance, the percentage of the population that ultimately mobilizes following the receipt of warning, and the speed at which they do mobilize, is dependent on human factors that are difficult to predict. Depending on the scenario and impact area, this study assumed between 90% - 99.5% of the PAR would be mobilized 12 hours after receiving a warning. While the utilized mobilization rates are informed by historical evidence and the cadre's judgment, Table 39 provides a sample of the effects of potential lower or higher mobilization rates.

Table 37 - Mobilization Assumption Comparison

Scenario	Type	Max Mobilization	Day LoL	Night LoL
OT_B_East	Sensitivity	100%	3.5	0.3
OT_B_East	Sensitivity	Best Case	62.5	24.4
OT_B_East	Sensitivity	99%	278.7	23.7
OT_B_East	Most Likely	Most Likely	128.6	76.0
OT_B_East	Sensitivity	98%	560.9	47.8
OT_B_East	Sensitivity	95%	1,397.0	118.9
OT_B_East	Sensitivity	Worst Case	1,433.4	181.6
OT_B_West	Sensitivity	100%	0.9	0.9
OT_B_West	Sensitivity	99%	78.1	112.1
OT_B_West	Sensitivity	Best Case	78.4	113.0
OT_B_West	Sensitivity	98%	156.2	224.2
OT_B_West	Sensitivity	95%	391.3	561.4
OT_B_West	Most Likely	Most Likely	426.9	614.9
OT_B_West	Sensitivity	93%	547.5	785.6
OT_B_West	Sensitivity	90%	783.0	1,123.3
OT_B_West	Sensitivity	Worst Case	781.3	1,123.2
W_335 75% Global Instability	Sensitivity	100%	60.0	110.5
W_335 75% Global Instability	Sensitivity	Best Case	96.5	172.7
W_335 75% Global Instability	Sensitivity	99%	105.2	188.6
W_335 75% Global Instability	Most Likely	Most Likely	219.5	371.0
W_335 75% Global Instability	Sensitivity	93%	243.3	408.6
W_335 75% Global Instability	Sensitivity	90%	311.4	518.8
W_335 75% Global Instability	Sensitivity	Worst Case	359.9	605.7

The Overtopping Sensitivity results demonstrate a near linear relationship between the max mobilization rate assumption and loss of life. This is because, with ample warning, few structures are inundated while PAR are still in the process of evacuating. This phenomenon is further evidenced in the table when you examine the results for a 100% mobilization rate; any loss of life while under this assumption is due to PAR being inundated prior to reaching the end of the curve.

Meanwhile, the Global Instability scenario is less sensitive to changes to the Mobilization Curve’s tail. Scenarios such as those do not have the benefit of significant warning, so more of the total loss of life will come from unwarned PAR or individuals who are capable of evacuating and intend to, but have not yet had the time to do so.

Interaction of Variables; Best & Worst Case Scenarios

While sections above generally outline the sensitivity of the results to each variable while holding other variables at the best estimate assumptions, the results may shift more if several variables move in tandem.

Table 40 is provided below. It displays the “best reasonable case” and the “worst reasonable case” scenarios. These were found by setting all assumptions to the least or most favorable of likely values. Without a model capable of handling uncertainty, we are not able to note their statistical probability. However, it is intended that these scenarios represent unlikely, but certainly not impossible, bookends of the Loss of Life range.

Table 38 – Loss of Life Range

Failure Location	Nominal RAS Loading	Failure Mode	Best Case Day	Best Case Night	Best Case Expected	Most Likely Day	Most Likely Night	Most Likely Expected	Worst Case Day	Worst Case Night	Worst Case Expected
East Station 410+00	1/2 Levee Height	Internal Erosion	5	3	**4**	22	16	**18**	232	35	**124**
East Station 410+00	3/4 Levee Height	Internal Erosion	107	43	**72**	305	102	**193**	3,363	447	**1,760**
East Station 410+00	Threshold	Internal Erosion	42	17	**29**	141	81	**108**	1,891	235	**981**
East Station 310+00	1/2 Levee Height	Internal Erosion	1	2	**2**	8	9	**9**	75	20	**45**
East Station 310+00	3/4 Levee Height	Internal Erosion	63	23	**41**	212	55	**126**	2,979	359	**1,538**
East Station 310+00	Threshold	Internal Erosion	32	13	**21**	119	71	**92**	1,502	220	**797**
East Station 222+00	1/2 Levee Height	Internal Erosion	0	1	**1**	4	6	**5**	22	14	**18**
East Station 222+00	3/4 Levee Height	Internal Erosion	77	18	**45**	222	43	**124**	4,992	500	**2,521**
East Station 222+00	Threshold	Internal Erosion	19	7	**12**	90	46	**66**	1,451	172	**748**
East Staton 74+00	1/2 Levee Height	Internal Erosion	0	0	**0**	0	0	**0**	5	3	**4**
East Staton 74+00	3/4 Levee Height	Internal Erosion	12	4	**7**	26	10	**18**	344	37	**175**
East Staton 74+00	Threshold	Internal Erosion	4	2	**3**	16	13	**14**	226	33	**119**
West Station 335+00	1/2 Levee Height	Internal Erosion	4	8	**6**	34	68	**53**	71	140	**109**
West Station 335+00	3/4 Levee Height	Internal Erosion	63	115	**92**	220	371	**303**	1,153	2,009	**1,624**
West Station 335+00	Threshold	Internal Erosion	36	56	**47**	245	369	**313**	814	1,275	**1,068**
West Station 250+00	1/2 Levee Height	Internal Erosion	2	5	**4**	21	43	**33**	53	108	**83**
West Station 250+00	3/4 Levee Height	Internal Erosion	84	159	**126**	223	392	**316**	1,201	2,174	**1,736**
West Station 250+00	Threshold	Internal Erosion	28	46	**38**	203	319	**267**	710	1,207	**984**
West Station 188+00	1/2 Levee Height	Internal Erosion	1	3	**2**	13	27	**21**	37	77	**59**
West Station 188+00	3/4 Levee Height	Internal Erosion	73	139	**109**	177	327	**259**	962	1,811	**1,429**
West Station 188+00	Threshold	Internal Erosion	24	40	**33**	173	283	**233**	593	1,021	**829**
West Station 10+00	1/2 Levee Height	Internal Erosion	2	2	**2**	4	4	**4**	9	11	**10**
West Station 10+00	3/4 Levee Height	Internal Erosion	8	9	**9**	30	28	**29**	100	97	**98**
West Station 10+00	Threshold	Internal Erosion	4	5	**5**	35	49	**43**	260	175	**213**
East Station 410+00	1/2 Levee Height	Global Instability	5	3	**4**	22	16	**18**	229	34	**122**
East Station 410+00	3/4 Levee Height	Global Instability	32	12	**21**	115	68	**89**	1,344	193	**711**
East Station 410+00	Threshold	Global Instability	42	17	**29**	141	81	**108**	1,891	235	**981**
East Station 310+00	1/2 Levee Height	Global Instability	1	2	**2**	8	9	**9**	69	20	**42**
East Station 310+00	3/4 Levee Height	Global Instability	15	6	**10**	71	31	**49**	845	112	**442**
East Station 310+00	Threshold	Global Instability	32	13	**21**	119	71	**92**	1,502	220	**797**
East Station 222+00	1/2 Levee Height	Global Instability	0	1	**1**	4	6	**5**	20	14	**17**
East Station 222+00	3/4 Levee Height	Global Instability	7	3	**5**	35	20	**27**	795	85	**405**
East Station 222+00	Threshold	Global Instability	19	7	**12**	90	46	**66**	1,451	172	**748**
East Staton 74+00	1/2 Levee Height	Global Instability	0	0	**0**	0	0	**0**	5	3	**4**
East Staton 74+00	3/4 Levee Height	Global Instability	1	1	**1**	7	8	**7**	85	19	**49**
East Staton 74+00	Threshold	Global Instability	4	2	**3**	16	13	**14**	226	33	**119**
West Station 335+00	1/2 Levee Height	Global Instability	4	8	**6**	34	68	**53**	67	132	**103**
West Station 335+00	3/4 Levee Height	Global Instability	20	34	**28**	157	258	**212**	458	764	**626**
West Station 335+00	Threshold	Global Instability	36	56	**47**	245	369	**313**	814	1,275	**1,068**
West Station 250+00	1/2 Levee Height	Global Instability	2	4	**3**	19	38	**30**	46	94	**72**
West Station 250+00	3/4 Levee Height	Global Instability	15	26	**21**	123	205	**168**	549	958	**774**
West Station 250+00	Threshold	Global Instability	28	46	**38**	203	319	**267**	710	1,207	**984**
West Station 188+00	1/2 Levee Height	Global Instability	1	2	**1**	12	24	**19**	32	67	**52**
West Station 188+00	3/4 Levee Height	Global Instability	12	22	**17**	96	173	**139**	421	775	**615**
West Station 188+00	Threshold	Global Instability	24	40	**33**	173	283	**233**	593	1,021	**829**
West Station 10+00	1/2 Levee Height	Global Instability	0	0	**0**	1	1	**1**	8	7	**7**
West Station 10+00	3/4 Levee Height	Global Instability	1	1	**1**	9	14	**12**	67	63	**65**
West Station 10+00	Threshold	Global Instability	4	5	**5**	35	49	**43**	260	175	**213**
East Levee Breach	Overtop A	Overtopping	39	16	**27**	129	76	**100**	1,382	186	**724**
East Wall Breach	Overtop A	Overtopping	6	3	**4**	22	16	**19**	292	41	**154**
West Levee Breach	Overtop A	Overtopping	31	50	**42**	216	331	**279**	579	886	**748**
East Levee Breach	Overtop B	Overtopping	61	23	**40**	175	100	**134**	2,470	295	**1,274**
East Wall Breach	Overtop B	Overtopping	35	15	**24**	124	82	**101**	1,148	190	**621**
West Levee Breach	Overtop B	Overtopping	68	99	**85**	427	615	**530**	928	1,363	**1,167**
No Failure / No Breach	Overtop A	N/A	0	0	**0**	0	0	**0**	2	1	**2**
No Failure / No Breach	Overtop B	N/A	4	3	**4**	23	26	**25**	185	62	**117**

Table 39 - Values Used for Loss of Life Ranges

Failure Scenario	Assumption Category	Assumption Value - Best Reasonable Case	Assumption Value - Most Likely	Assumption Value - Worst Reasonable Case
Internal Erosion and Global Instability	Failure Warning Issuance - Full Loading	11.5 Hours Before Breach	7.25 Hours Before Breach	1 Hour After Breach
	Failure Warning Issuance - 75% Loading	Internal Erosion - 30 minutes After Breach Global Instability - 11.5 Hours Before Breach	Internal Erosion - 45 minutes After Breach Global Instability - 7.25 Hours Before Breach	Internal Erosion - 4 Hours After Breach Global Instability - 1 Hour After Breach
	East Levee Dominate Mobilization Curve	Day - 95% Night - 93%	Day - 99.5% Night - 96%	Day - 99.8% Night - 99%
	West Levee Dominate Mobilization Curve	90.0%	94.0%	99.0%
	Foundation Height	0.5 Feet	1 Foot	2 Feet
	Warning System (Used as Proxy)	Residential Areas - Tone-Alert System (Proxy) Commercial Areas - Autodial Telephone System (Proxy)	Residential Areas - Sirens (Proxy) Commercial Areas - Tone-Alert System (Proxy)	Residential Areas - EBS (Proxy) Commercial Areas - Sirens (Proxy)
Overtopping	Failure Warning Issuance	11.5 Hours Before Breach	7.25 Hours Before Breach	1 Hour After Breach
	East Levee Dominate Mobilization Curve	Day - 95% Night - 93%	Day - 99.5% Night - 96%	Day - 99.8% Night - 99%
	West Levee Dominate Mobilization Curve	90%	94.5%	99%
	Foundation Height	0.5 Feet	1 Foot	2 Feet
	Warning System (Used as Proxy)	Residential Areas - Tone-Alert System (Proxy) Commercial Areas - Autodial Telephone System (Proxy)	Residential Areas - Sirens (Proxy) Commercial Areas - Tone-Alert System (Proxy)	Residential Areas - EBS (Proxy) Commercial Areas - Sirens (Proxy)

The range between best and worst case scenarios is generally an order of magnitude or more. The reasons for the wide range vary by scenario, but the warning issuance assumption is typically the largest driver for Internal Erosion. By shifting the warning to four hours after breach instead of 0.5 hours after breach, a significant percent of the PAR is either caught evacuating or inundated before mobilizing.

The max mobilization assumption was also a larger driver of uncertainty for Global Instability and Overtopping. With ample warning time, the max mobilization rate is often reached. By lowering the max mobilization to 90% from 99% of PAR, Loss of Life saw significant increases.

Economic Impacts

Significant economic impacts would also result from a failure of the Dallas Levee System, including damages to private and public property. The damage categories included in the analysis of economic impacts are limited to damages to structures and their contents, including residential, public and commercial structures, damages to vehicles. Other potential categories such as damages to roads, emergency costs, or indirect business losses are not within the scope of this study.

In addition to the loss of life analysis, HEC-FIA was also used for this study in the estimation of damages to property. Unlike the Flood Damage Analysis software package (HEC-FDA), FIA does not perform a probabilistic calculation but rather an event analysis. HEC-FIA includes GIS functionality, allowing the use of GIS data within the program. HEC-FIA can import structure inventories as GIS layers from parcel data or generate them from FEMA's HAZUS-MH database, which includes a national inventory of essential facilities such as fire, police and other emergency facilities, utilities and transportation, and what is called "General Building Stock", which includes residential, commercial, religious, education, government, agricultural and industrial structures with associated values and other characteristics. The HAZUS-MH nonresidential structure data is provided by Dun & Bradstreet, and the US Department of Commerce's Census of Housing was used to generate residential structure data. Both residential and non-residential inventories are spatially referenced at a census block level, structures within a census block being assigned by FIA to evenly distributed locations within each census block. The Dun & Bradstreet nonresidential data, according to the HAZUS-MH MR3 Flood Model technical manual, represents 76 percent of the estimated 19 million businesses in the United States, and roughly 98 percent of the gross national product. The manual states that a portion of the remaining unaccounted for businesses are likely to be home-based, and thus accounted for in the residential structures estimate.

As discussed in the Loss of Life section, georeferenced structure inventories and associated characteristics and values from SWF was incorporated into the analysis, greatly improving assumptions on structure value and spatial location.

In the computation of property damages, HEC-FIA assigns each structure, be it a structure point or polygon or HAZUS point, a ground elevation based on its location on a digital terrain model. Flooding is computed from depth grids for each failure mode or flood event. Just as in HEC-FDA, structures are assigned percent depth-damage functions based on structure type, number of floors, the presence or absence of a basement, etc. Flood elevations are determined for each structure by comparison of the corresponding points on the depth grids and terrain model, these elevations then indicate, via the depth-damage relationships, the percentage of the replacement value that will be lost due to flood event.

Vehicle damages were also calculated using the HAZUS dataset. HAZUS provides estimated day and night vehicle counts and values for both new and used light trucks, heavy trucks, and cars. As with the structure inventory, this data is provided for every census block. The vehicle counts are totaled for every vehicle type and evenly distributed by the FIA program between every structure in a census block. The flood depth at each structure, as estimated above, is applied to the HAZUS vehicle depth-damage function to estimate vehicle damages for every flood/failure event.

Results for these combined "Urban Damages" are reported below in Table 42.

Table 40 – Total Urban Damages

Failure Location	Nominal RAS Loading	Faulure Mode	Urban Damage
East Station 410+00	1/2 Levee Height	Internal Erosion	$2,233,646,108
East Station 410+00	3/4 Levee Height	Internal Erosion	$2,772,199,130
East Station 410+00	Threshold	Internal Erosion	$2,948,052,629
East Station 310+00	1/2 Levee Height	Internal Erosion	$1,882,145,582
East Station 310+00	3/4 Levee Height	Internal Erosion	$2,527,921,695
East Station 310+00	Threshold	Internal Erosion	$2,777,496,244
East Station 222+00	1/2 Levee Height	Internal Erosion	$1,575,889,383
East Station 222+00	3/4 Levee Height	Internal Erosion	$2,335,599,422
East Station 222+00	Threshold	Internal Erosion	$2,627,211,508
East Staton 74+00	1/2 Levee Height	Internal Erosion	$867,221,180
East Staton 74+00	3/4 Levee Height	Internal Erosion	$1,785,811,163
East Staton 74+00	Threshold	Internal Erosion	$2,111,325,444
West Station 335+00	1/2 Levee Height	Internal Erosion	$244,959,037
West Station 335+00	3/4 Levee Height	Internal Erosion	$454,173,160
West Station 335+00	Threshold	Internal Erosion	$532,131,936
West Station 250+00	1/2 Levee Height	Internal Erosion	$193,795,758
West Station 250+00	3/4 Levee Height	Internal Erosion	$412,632,903
West Station 250+00	Threshold	Internal Erosion	$499,727,636
West Station 188+00	1/2 Levee Height	Internal Erosion	$186,367,297
West Station 188+00	3/4 Levee Height	Internal Erosion	$431,999,069
West Station 188+00	Threshold	Internal Erosion	$534,654,230
West Station 10+00	1/2 Levee Height	Internal Erosion	$36,260,440
West Station 10+00	3/4 Levee Height	Internal Erosion	$180,170,730
West Station 10+00	Threshold	Internal Erosion	$267,443,292
East Station 410+00	1/2 Levee Height	Global Instability	$2,233,646,108
East Station 410+00	3/4 Levee Height	Global Instability	$2,772,199,130
East Station 410+00	Threshold	Global Instability	$2,948,052,629
East Station 310+00	1/2 Levee Height	Global Instability	$1,882,145,582
East Station 310+00	3/4 Levee Height	Global Instability	$2,527,921,695
East Station 310+00	Threshold	Global Instability	$2,777,496,244
East Station 222+00	1/2 Levee Height	Global Instability	$1,575,889,383
East Station 222+00	3/4 Levee Height	Global Instability	$2,335,599,422
East Station 222+00	Threshold	Global Instability	$2,627,211,508
East Staton 74+00	1/2 Levee Height	Global Instability	$867,221,180
East Staton 74+00	3/4 Levee Height	Global Instability	$1,785,811,163
East Staton 74+00	Threshold	Global Instability	$2,111,325,444
West Station 335+00	1/2 Levee Height	Global Instability	$244,959,037
West Station 335+00	3/4 Levee Height	Global Instability	$454,173,160
West Station 335+00	Threshold	Global Instability	$532,131,936
West Station 250+00	1/2 Levee Height	Global Instability	$193,795,758
West Station 250+00	3/4 Levee Height	Global Instability	$412,632,903
West Station 250+00	Threshold	Global Instability	$499,727,636
West Station 188+00	1/2 Levee Height	Global Instability	$186,367,297
West Station 188+00	3/4 Levee Height	Global Instability	$431,999,069
West Station 188+00	Threshold	Global Instability	$534,654,230
West Station 10+00	1/2 Levee Height	Global Instability	$36,260,440
West Station 10+00	3/4 Levee Height	Global Instability	$180,170,730
West Station 10+00	Threshold	Global Instability	$267,443,292
East Levee Breach	Overtop A	Overtopping	$3,253,502,908
East Wall Breach	Overtop A	Overtopping	$2,592,131,706
West Levee Breach	Overtop A	Overtopping	$821,406,019
East Levee Breach	Overtop B	Overtopping	$3,598,060,722
East Wall Breach	Overtop B	Overtopping	$3,359,411,536
West Levee Breach	Overtop B	Overtopping	$1,666,691,448
No Failure / No Breach	Overtop A	N/A	$427,658,613
No Failure / No Breach	Overtop B	N/A	$2,565,076,304

Supplemental Imagery

Figure 91 – Census 2000 Population Counts (Residents Only)

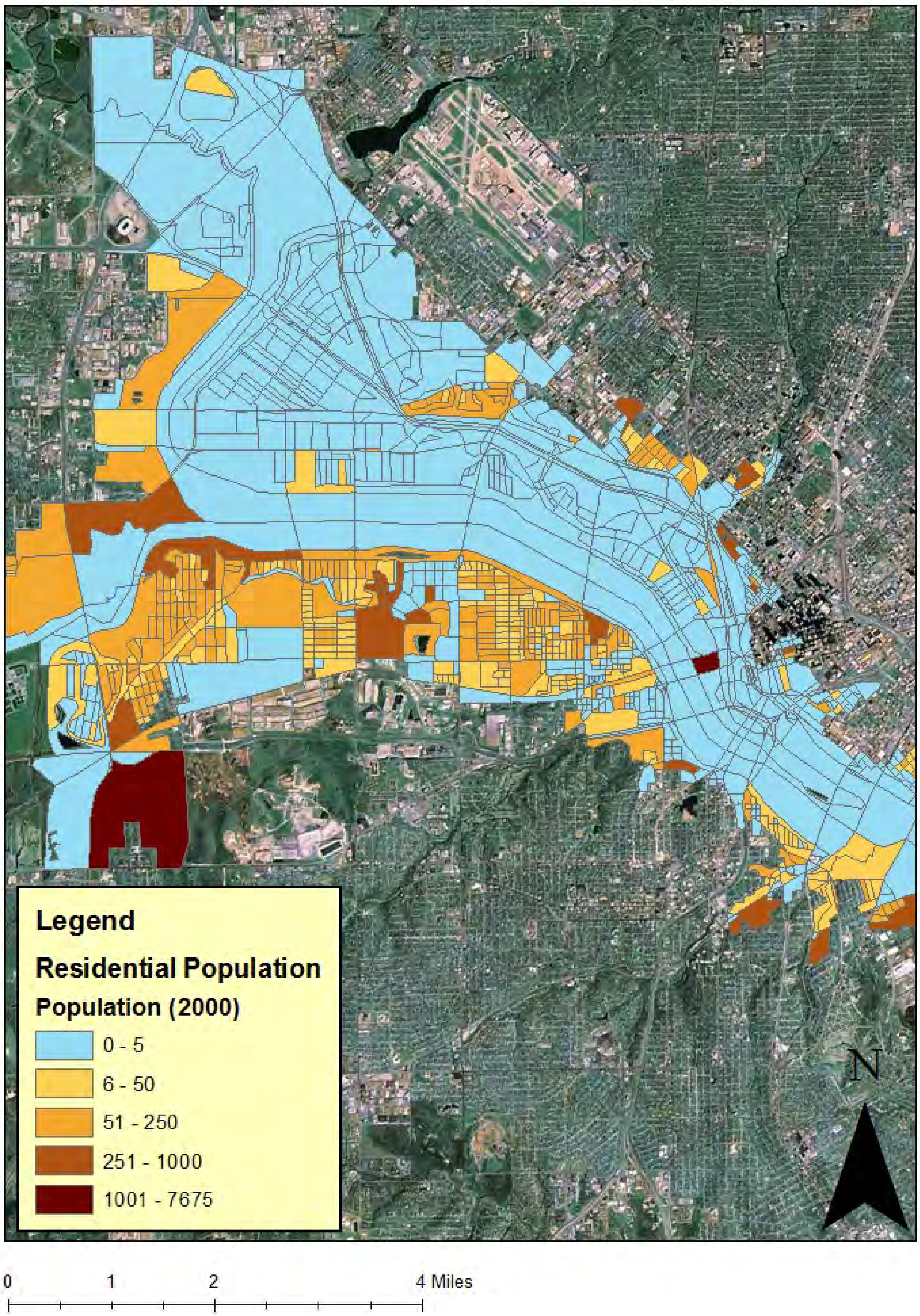


Figure 92 – Impact Areas Used in FIA

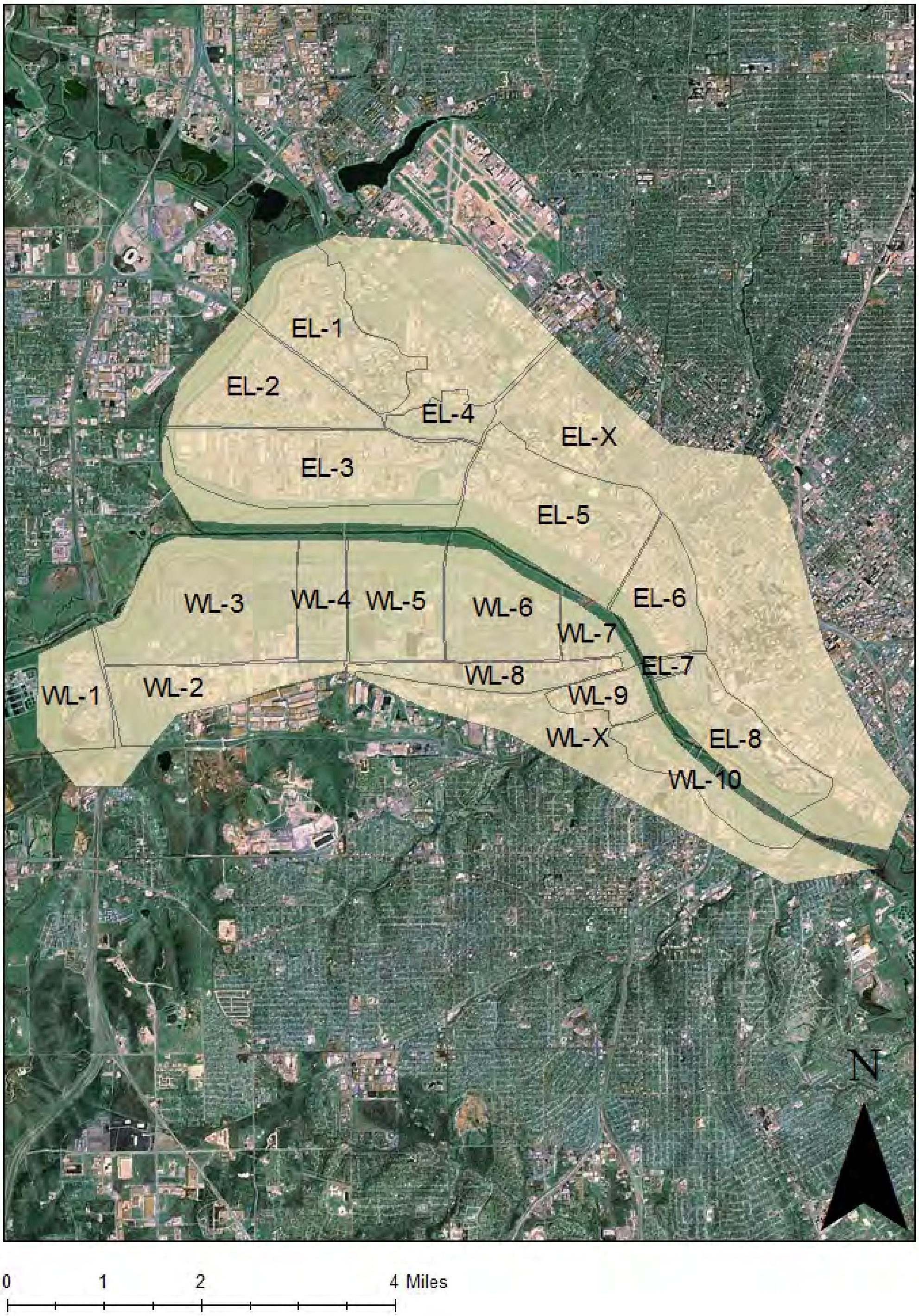


Figure 93 – Overtopping B NonFail Hazard Area with Wall Failure Hazard Area

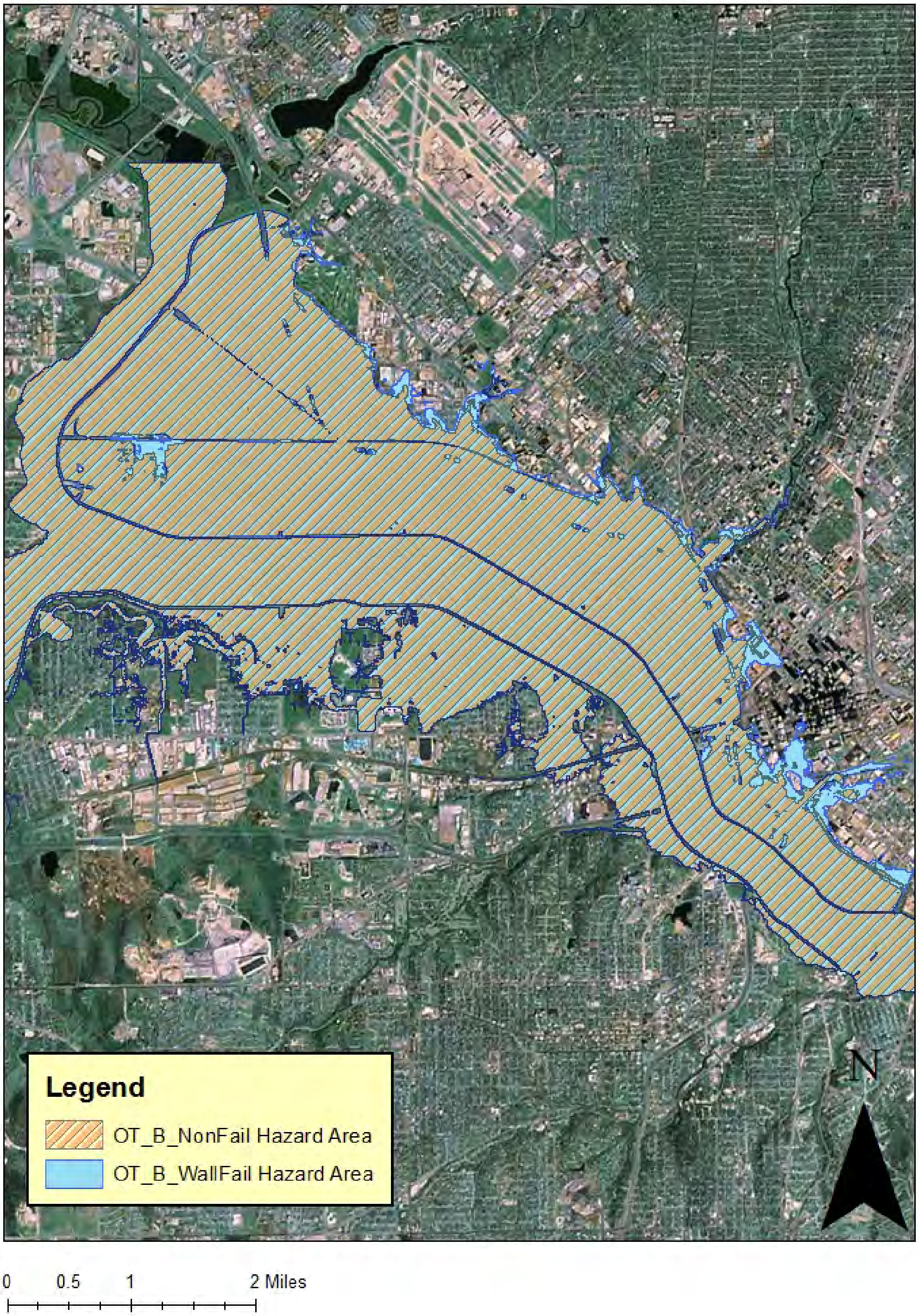


Figure 94 – Overtopping B West Levee and East Levee Failures

Figure 95 – Sample of Eastern Levee Hazard Areas

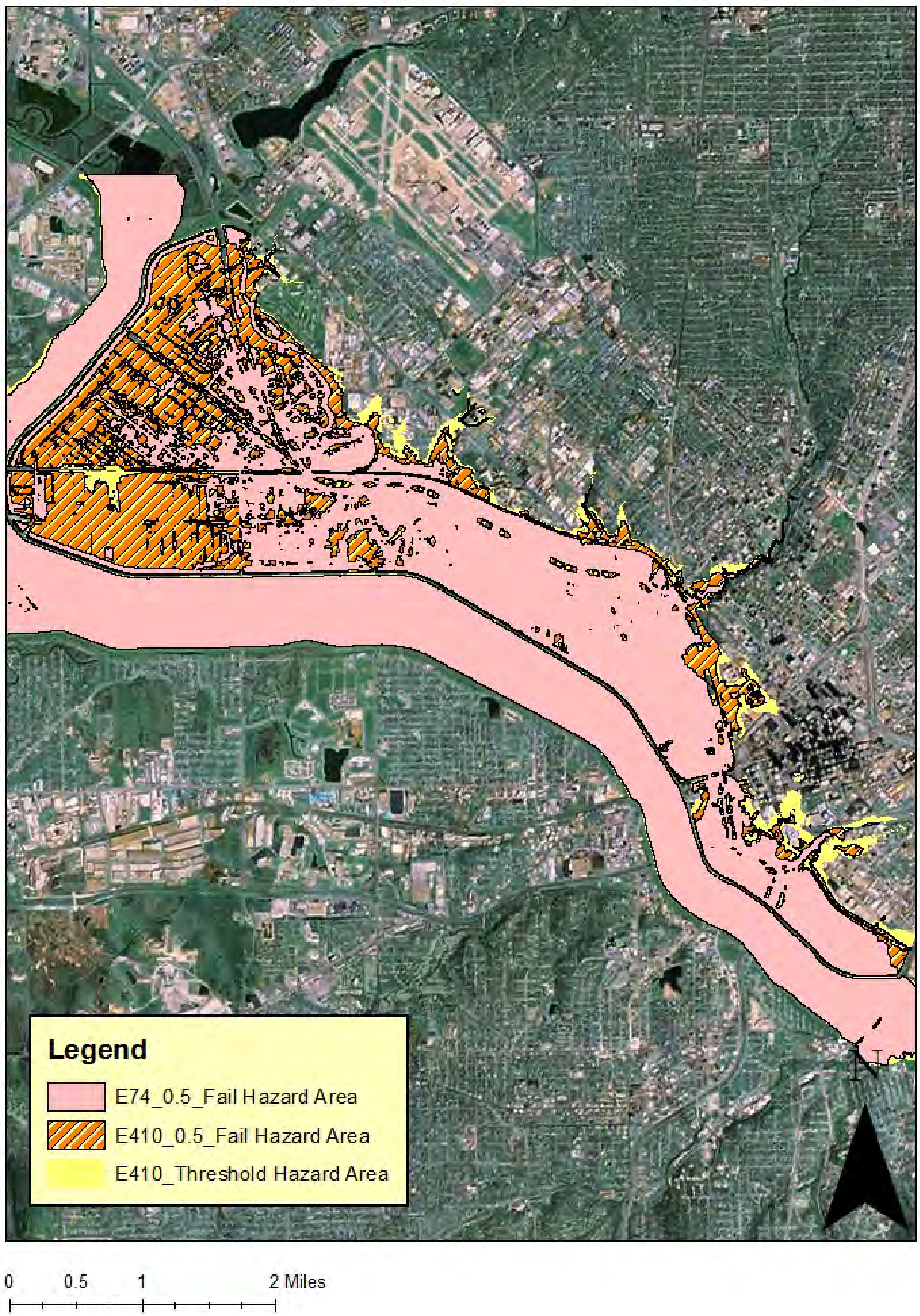
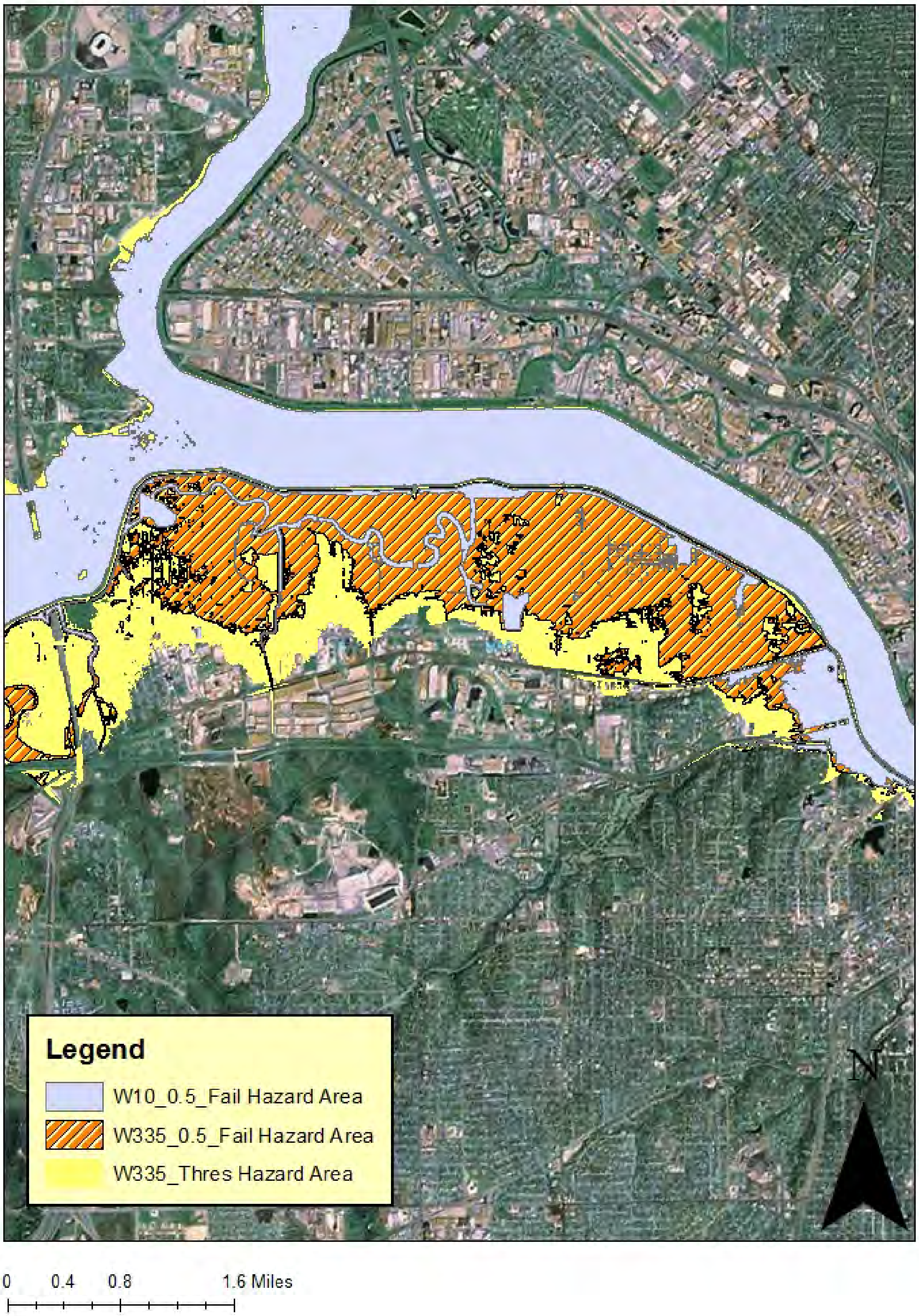


Figure 96 – Sample of Western Levee Hazard Areas



Attachment 2 – Mobilization Toolbox

Table 41 – Mobilization Index Inputs for Full Loading and Overtopping Scenarios

	Population age 17 and younger	Population over 65 and living alone or with spouse	Population living in group quarters	Households with single occupant	Households linguistically isolated	Households without telephone service	Households without vehicles	Population Density	Mean Travel Time to Work	Percent of Population who took group transit to work	Percent of Population with no diploma	Percent of population below 1.5 Poverty rate	Distance To Evacuate	Quality of Warning Message	Prolonged Detected Failure?	Level of Community Awareness	Severe Rainfall Event (e.g. PMF)?
Impact Area's Value	23.03%	4.24%	1.50%	28.78%	15.33%	8.96%	23.58%	1,724.1	30.2	40.74%	60.55%	52.20%	Medium (1 - 3 Mi)	Good	No (<24 Hours)	Significant	Yes
Nationwide County Mean	20.38%	8.20%	3.61%	26.93%	1.92%	4.86%	6.41%	252.5	24.1	12.30%	17.54%	26.15%					
Std Deviations from Mean	0.5566106	-1.4679753	-0.441803	0.40009856	3	1.34904645	3	0.9335068	1.085249	3	3	2.9217044	0	2	1.5	2	2
	Significance of Variable Across Mobilization Curve																
5	-1	-1	-1	+5	-2	-2	-2	+4	-4	-2	-2	-3	-2	+5	+4	+3	+4
15	-1	-3	-1	+4	-2	-2	-3	+3	-3	-2	-2	-3	-3	+5	+4	+3	+4
60	+1	-4	-1	+1	-2	-1	-4	+2	-2	-1	-2	-3	-4	+5	+3	+3	+4
120	+1	-4	-1	+0	-3	-1	-4	+1	-0	+0	-2	-3	-4	+5	+3	+3	+4
240	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-4	+5	+4	+2	+3
480	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-5	+5	+5	+2	+3
720	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-5	+5	+5	+2	+3

Table 42 – Mobilization Index Range and Outcome for Full Loading and Overtopping Scenarios

Minutes Afte Warned	High Mob	Default Mob	Low Mob	Index Sum	Index Max	Percent Change to Default	Modified Mob
5	25.00%	16.67%	1.00%	-2.71651	141	-0.30%	16.36%
15	65.00%	50.00%	25.00%	-3.02892	144	-0.53%	49.47%
60	90.00%	75.00%	50.00%	-1.64723	129	-0.32%	74.68%
120	95.00%	85.00%	65.00%	-0.81034	117	-0.14%	84.86%
240	99.00%	93.00%	75.00%	-8.16417	132	-1.11%	91.89%
480	99.50%	94.00%	80.00%	-6.66417	138	-0.68%	93.32%
720	99.75%	95.00%	85.00%	-6.66417	138	-0.48%	94.52%

Table 43 – Mobilization Index Inputs for Half and Three Quarters Nominal RAS Loadings

	Population age 17 and younger	Population over 65 and living alone or with spouse	Population living in group quarters	Households with single occupant	Households linguistically isolated	Households without telephone service	Households without vehicles	Population Density	Mean Travel Time to Work	Percent of Population who took group transit to work	Percent of Population with no diploma	Percent of population below 1.5 Poverty rate	Distance To Evacuate	Quality of Warning Message	Prolonged Detected Failure?	Level of Community Awareness	Severe Rainfall Event (e.g. PMF)?
Impact Area's Value	23.03%	4.24%	1.50%	28.78%	15.33%	8.96%	23.58%	1,724.1	30.2	40.74%	60.55%	52.20%	Medium (1 - 3 Mi)	Good	No (<24 Hours)	Significant	Yes
Nationwide County Mean	20.38%	8.20%	3.61%	26.93%	1.92%	4.86%	6.41%	252.5	24.1	12.30%	17.54%	26.15%					
Std Deviations from Mean	0.5566106	-1.4679753	-0.441803	0.40009856	3	1.34904645	3	0.9335068	1.085249	3	3	2.9217044	0	2	0.75	2	1
	Significance of Variable Across Mobilization Curve																
5	-1	-1	-1	+5	-2	-2	-2	+4	-4	-2	-2	-3	-2	+5	+4	+3	+4
15	-1	-3	-1	+4	-2	-2	-3	+3	-3	-2	-2	-3	-3	+5	+4	+3	+4
60	+1	-4	-1	+1	-2	-1	-4	+2	-2	-1	-2	-3	-4	+5	+3	+3	+4
120	+1	-4	-1	+0	-3	-1	-4	+1	-0	+0	-2	-3	-4	+5	+3	+3	+4
240	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-4	+5	+4	+2	+3
480	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-5	+5	+5	+2	+3
720	+1	-5	-1	-1	-3	-1	-5	+1	-0	+1	-3	-4	-5	+5	+5	+2	+3

Table 44 – Mobilization Index Range and Outcome for Half and Three Quarters Nominal RAS Loadings

Minutes Afte Warned	High Mob	Default Mob	Low Mob	Index Sum	Index Max	Percent Change to Default	Modified Mob
5	25.00%	16.67%	1.00%	-9.71651	141	-1.08%	15.59%
15	65.00%	50.00%	25.00%	-10.0289	144	-1.74%	48.26%
60	90.00%	75.00%	50.00%	-7.89723	129	-1.53%	73.47%
120	95.00%	85.00%	65.00%	-7.06034	117	-1.21%	83.79%
240	99.00%	93.00%	75.00%	-14.1642	132	-1.93%	91.07%
480	99.50%	94.00%	80.00%	-13.4142	138	-1.36%	92.64%
720	99.75%	95.00%	85.00%	-13.4142	138	-0.97%	94.03%

To help guide the selection of a Mobilization Curve an Excel based Mobilization Toolbox was used. The toolbox uses a combination of census demographic data and the economist's knowledge of the community to shift from the default curve toward either an extreme best case or extreme worst-case scenario. It uses the impact area's standard deviation from the national county mean to determine how bad or good things are in a given community versus a "typical" community, and it assumes that an impact area that is completely average (i.e. every variable has a standard deviation of zero) would use the default curve.

The number of people without vehicles should often be an important variable. It is often cited as a reason why so many people were left behind during Hurricane Katrina. Let us say, for right now, that this was the only variable, other than time, that matters. If a community has a standard deviation of positive 3 that means it has a very high number of people without vehicles. Therefore, the community's best estimate mobilization curve would shift all the way to the worst-case scenario. If, on the other hand, a community had a standard deviation of negative 3 then it would have an unusually small number of people without vehicles and it would shift all the way to the best-case scenario. The user assigns either a positive or a negative correlation between a variable and the curve.

Best and worst-case scenarios are also entered by the user. There is limited data to guide this decision. Most sources cite Katrina's mobilization rate as around 90%, perhaps 92 or 93% if those who went to places like the Superdome are considered mobilized. However, as bad as things were in this example, it could have been worse, the population could have been older or there could have been less advanced warning. However, it is hard to extrapolate too much from this, as it is only one case.

If we consider a two variable example, vehicle ownership and percentage of Pop in poverty, users assign significance values to each variable and could make it so one variable has 5 times the impact on the shift as another variable. However, for this explanation, let us say each has equal weight, one variable has a standard deviation of 3 and one variable has a standard deviation of 0. The formula takes the average. In this case, the overall demographics are only half as bad as they could be (1.5 versus a max of 3); therefore, it will only shift half way to the worst-case scenario. For example, if the default is 98% and the worst case is 90% the best estimate would be 94%.

The significance values are only for relative purposes. For instance, if the standard deviation for one variable is 3 and its significance is 5 and the other variable has a standard deviation of 0 and a significance of one then the weighted average would be 0*1 + 3*5 = 15. 15 / (1+5) = 2.5. This would mean that the curve would shift 83% toward the worst-case scenario (2.5/3 = .83). This methodology assumes a standard deviation of three is a good cutoff point.

Rather than an index, it may help the reader to consider this methodology similar to regression analysis, with the standard deviation serving as the X value and the assigned significance serving as the parameter value. The default mobilization rate is similar to the intercept.

One obstacle is, the literature does not suggest robust estimates as to how much more important one variable is relative to another and how much a given variable effects mobilization – conclusions are generally limited to whether a variable has a positive or negative impact. Justifications for utilized parameters are limited to best judgment and interpretations from literature reviews. Of course, this may be better than simply using some standard parameters. Vehicle ownership rates would certainly matter if you have to travel several miles to escape from a flood plain, but, if you only have to travel 0.05 miles, they likely would matter much less.

Non-demographic variables are also included as they play an essential role in the PAR's risk perception process. The perceived legitimacy of the warning, environmental cues, and the ability of EMAs to make arrangements for vulnerable citizens could all have dramatic impacts on the eventual evacuation rate of the PAR.

Despite limitations, the advantage of using this toolbox is that it forces the risk analysis to make transparent the assumptions on the importance of various variables on mobilization. In addition, it provides a systematic method to compare the relative vulnerability of many communities by weighing against each other many variables that have been shown by the literature to influence rates of evacuation. Nonetheless, a full range of results generated from sensitivity analysis must be considered.

Table 45 – Most Likely Mobilization Curves by Impact Area

East Levee Com Day		East Levee Com Night		East Levee Resid		East Levee Jail	
0	0.0%	0	0.0%	0	0.0%	0	0.0%
5	20.0%	5	16.7%	5	16.7%	5	1.0%
15	60.0%	15	50.0%	15	50.0%	15	5.0%
60	80.0%	60	75.0%	60	75.0%	60	15.0%
120	90.0%	120	85.0%	120	85.0%	120	25.0%
240	98.0%	240	93.0%	240	93.0%	240	50.0%
480	99.0%	480	95.0%	480	94.0%	480	75.0%
720	99.5%	720	96.0%	720	95.0%	720	90.0%
B		B		C		D	

Best Estimates

West Levee Resid		West Levee Com		WL Resid - 3qt and below		Prelim Mob	
0	0.0%	0	0.0%	0	0.0%	0	0.00%
5	16.4%	5	16.7%	5	15.6%	5	10.00%
15	49.5%	15	50.0%	15	48.3%	15	10.00%
60	74.7%	60	75.0%	60	73.5%	60	10.00%
120	84.9%	120	85.0%	120	83.8%	120	10.00%
240	91.9%	240	95.0%	240	91.1%	240	10.00%
480	93.3%	480	97.0%	480	92.6%	480	10.00%
720	94.5%	720	98.0%	720	94.0%	720	10.00%
A		E		A			

Table 46 – Worst Case Mobilization Curves by Impact Area

East Levee Com Day		East Levee Com Night		East Levee Resid		East Levee Jail	
0	0.0%	0	0.0%	0	0.00%	0	0.0%
5	16.7%	5	16.7%	5	16.67%	5	1.0%
15	50.0%	15	50.0%	15	50.00%	15	5.0%
60	75.0%	60	75.0%	60	70.00%	60	15.0%
120	85.0%	120	85.0%	120	80.00%	120	25.0%
240	93.0%	240	90.0%	240	85.00%	240	30.0%
480	94.0%	480	92.0%	480	87.50%	480	30.0%
720	95.0%	720	93.0%	720	90.00%	720	30.0%
B		B		C		D	

Worst Case

West Levee Resid		West Levee Com		Prelim Mob	
0	0.00%	0	0.0%	0	0.00%
5	16.67%	5	16.7%	5	5.00%
15	50.00%	15	50.0%	15	5.00%
60	70.00%	60	75.0%	60	5.00%
120	80.00%	120	85.0%	120	5.00%
240	85.00%	240	90.0%	240	5.00%
480	87.50%	480	92.0%	480	5.00%
720	90.00%	720	93.0%	720	5.00%
A		E			

Table 47 – Best Case Mobilization Curves by Impact Area

East Levee Com Day		East Levee Com Night		East Levee Resid		East Levee Jail	
0	0.0%	0	0.0%	0	0.0%	0	0.0%
5	20.0%	5	16.7%	5	16.4%	5	1.0%
15	60.0%	15	50.0%	15	49.5%	15	5.0%
60	80.0%	60	75.0%	60	74.7%	60	15.0%
120	90.0%	120	85.0%	120	84.9%	120	25.0%
240	99.0%	240	95.0%	240	95.0%	240	50.0%
480	99.5%	480	97.0%	480	98.0%	480	75.0%
720	99.8%	720	99.0%	720	99.0%	720	95.0%
B		B		C		D	

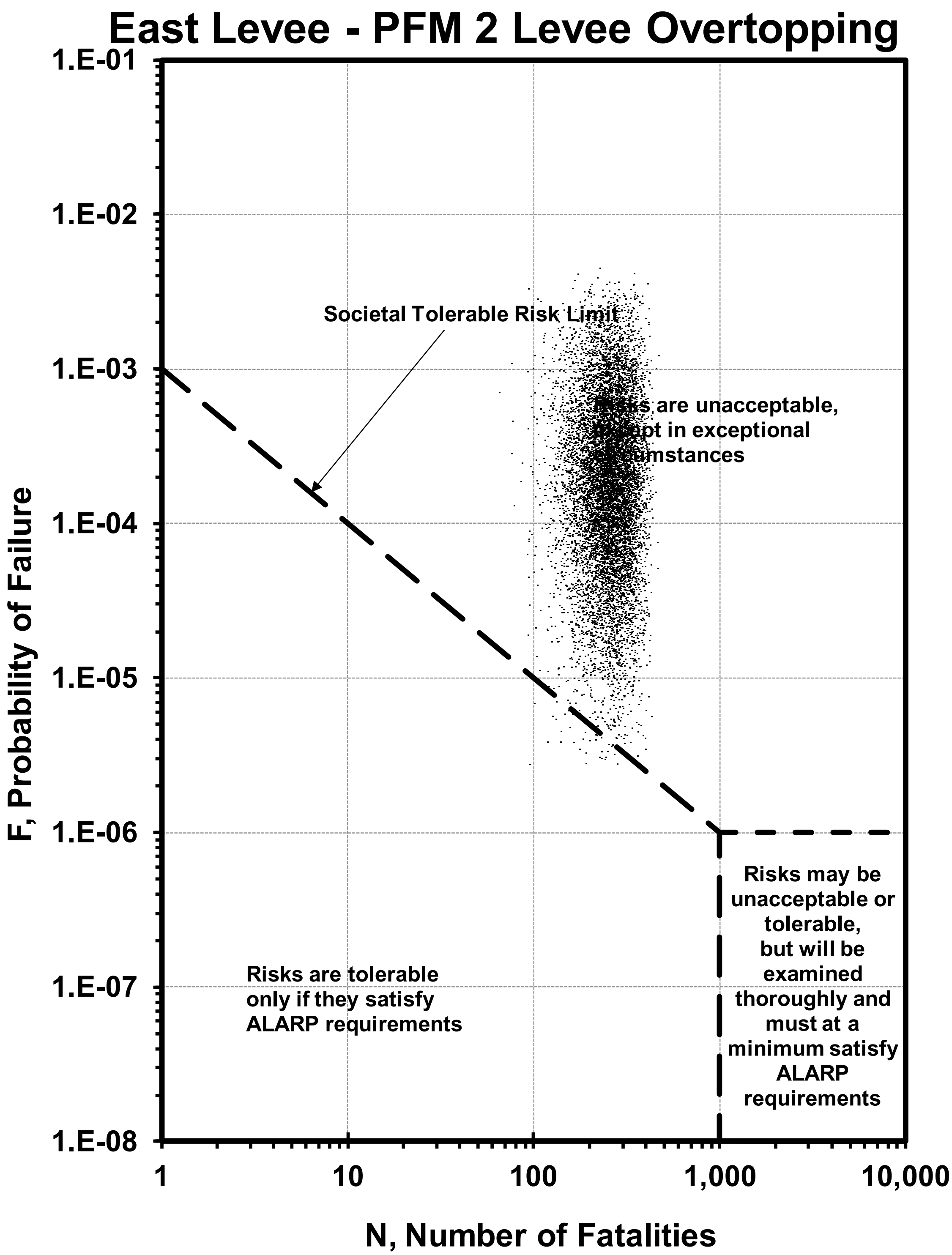
West Levee Resid		West Levee Com		Prelim Mob	
0	0.0%	0	0.0%	0	0.00%
5	16.4%	5	16.7%	5	20.00%
15	49.5%	15	50.0%	15	20.00%
60	74.7%	60	75.0%	60	20.00%
120	84.9%	120	85.0%	120	20.00%
240	95.0%	240	98.0%	240	20.00%
480	98.0%	480	99.0%	480	20.00%
720	99.0%	720	99.5%	720	20.00%
A		E			

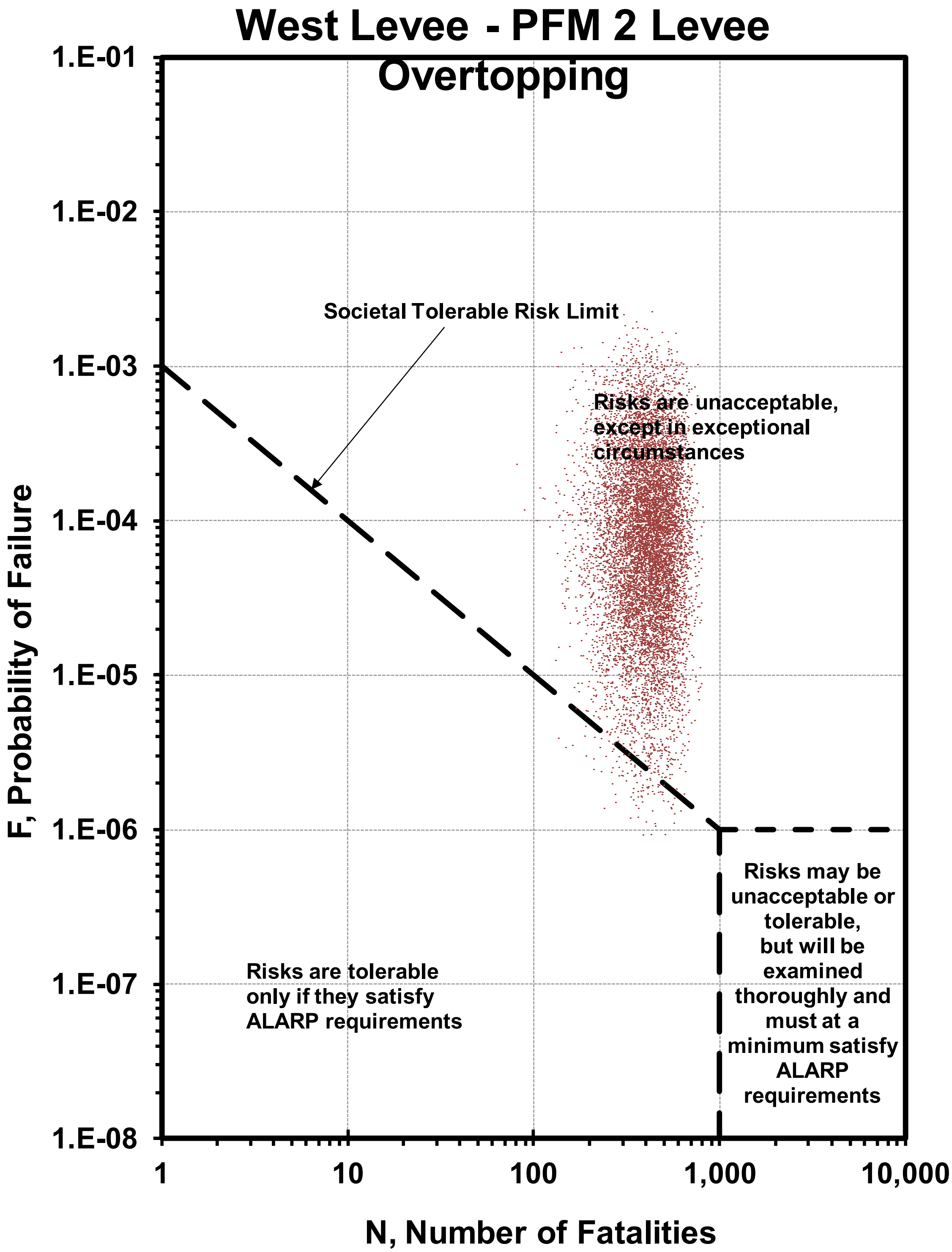
Best Case

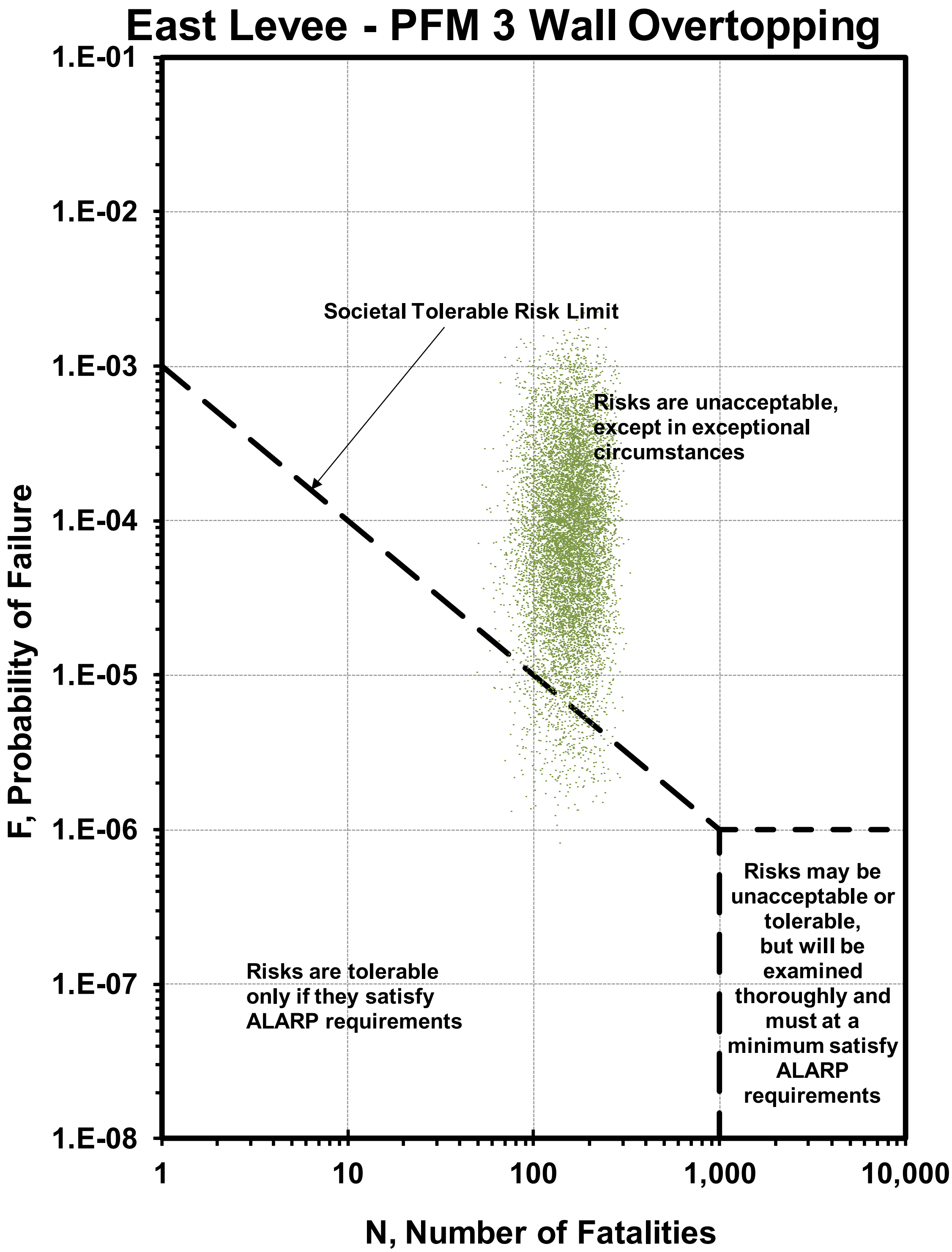
Table 48 – EAP / FIA Impact Area and Associated Warning Curve

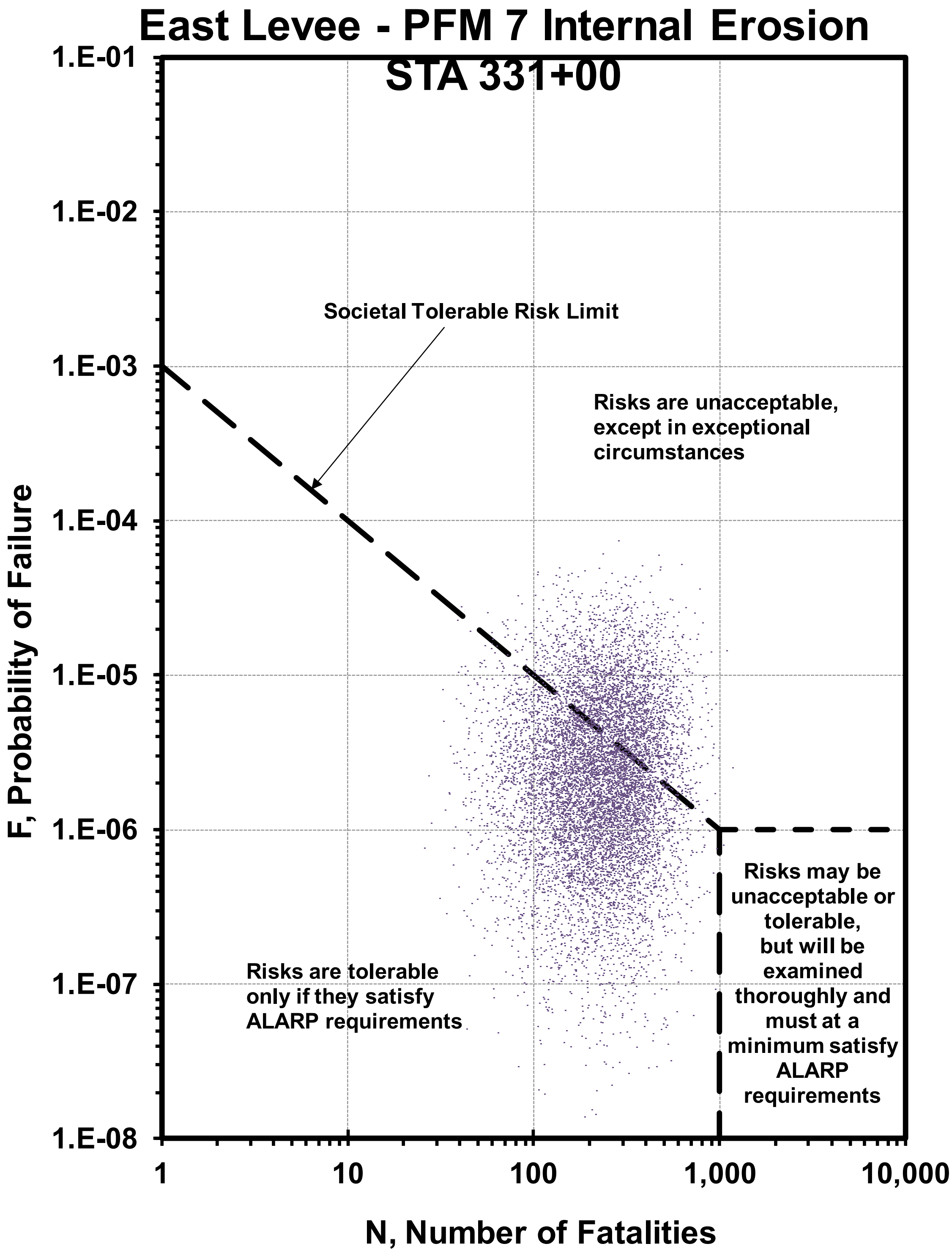
EL-1	B
EL-2	B
EL-X	B
EL-4	C
EL-5	B
EL-3	B
EL-6	B
WL-3	A
WL-5	A
WL-6	A
WL-4	A
WL-X	A
WL-7	A
EL-8	B
EL-7	D
WL-1	E
WL-8	A
WL-2	A
WL-9	A
WL-10	A

Appendix F – Uncertainty Results




West Levee - PFM 2 Levee Overtopping
1.E-01
1.E-02
1.E-03
1.E-04
1.E-05
1.E-06
1.E-07
1.E-08
F, Probability of Failure
Societal Tolerable Risk Limit
Risks are unacceptable, except in exceptional circumstances
Risks may be unacceptable or tolerable, but will be examined thoroughly and must at a minimum satisfy ALARP requirements
Risks are tolerable only if they satisfy ALARP requirements
1
10
100
1,000
10,000
N, Number of Fatalities


East Levee - PFM 3 Wall Overtopping
1.E-01
1.E-02
1.E-03
1.E-04
1.E-05
1.E-06
1.E-07
1.E-08
F, Probability of Failure
1
10
100
1,000
10,000
N, Number of Fatalities
Societal Tolerable Risk Limit
Risks are unacceptable, except in exceptional circumstances
Risks may be unacceptable or tolerable, but will be examined thoroughly and must at a minimum satisfy ALARP requirements
Risks are tolerable only if they satisfy ALARP requirements


East Levee - PFM 7 Internal Erosion
STA 331+00
F, Probability of Failure
1.E-01
1.E-02
1.E-03
1.E-04
1.E-05
1.E-06
1.E-07
1.E-08
1
10
100
1,000
10,000
N, Number of Fatalities
Societal Tolerable Risk Limit
Risks are unacceptable, except in exceptional circumstances
Risks may be unacceptable or tolerable, but will be examined thoroughly and must at a minimum satisfy ALARP requirements
Risks are tolerable only if they satisfy ALARP requirements