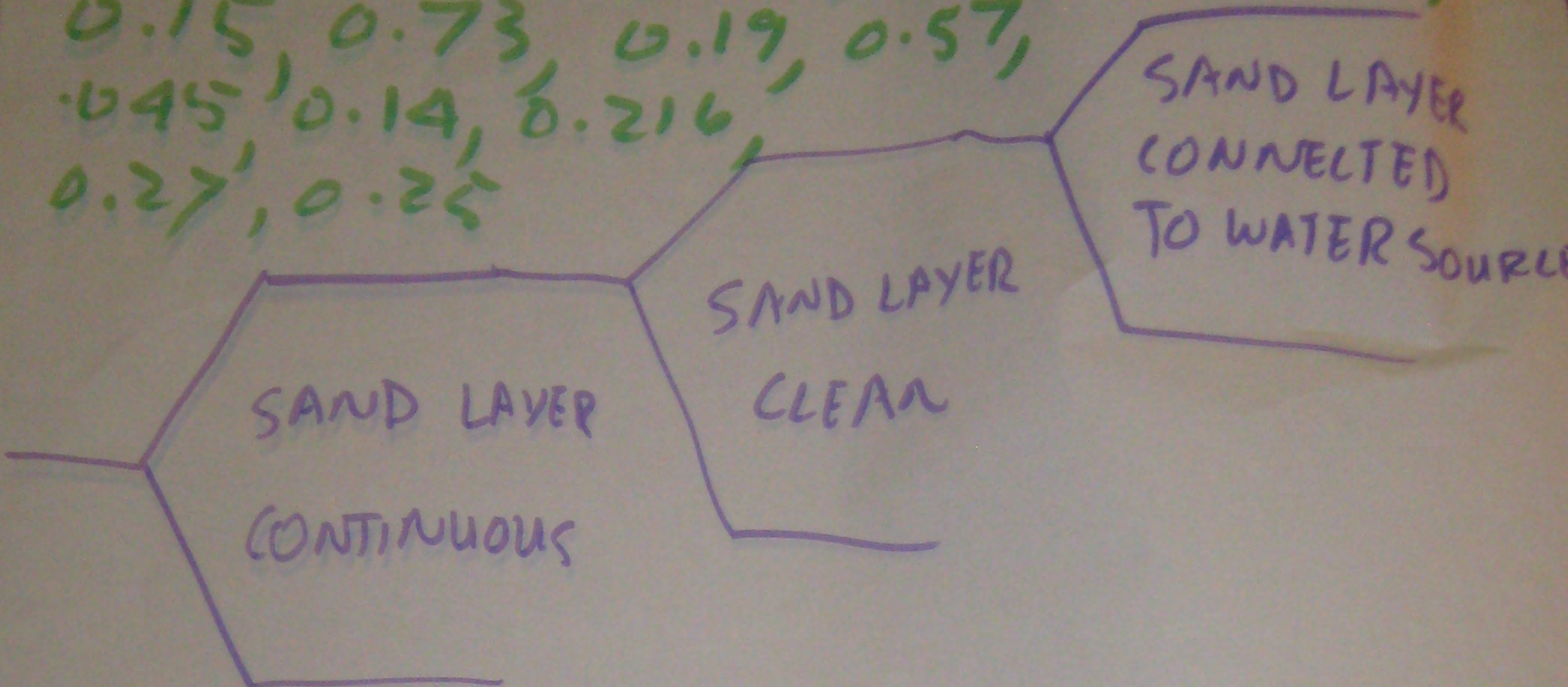
ESTIMATES - 0.1, 0.1, 0.08, 0.125, 0.15, 0.73, 0.19, 0.57, .045, 0.14, 0.216, 0.27, 0.25



STA 220E DISTANCE FROM RIVER TO LOW CLAY COVER ON PROTECTED SIDE

~ 1,450 ft

RATIONALE - ESTIMATES WENT DOWN WHEN DECOMPOSED - MOST ESTIMATED AT LEAST ONE BRANCH LOWER ~~i.e.~~ e.g. - SAND CLEAN

DALLAS FLOODW

MAY OCCUR
• CLAY BLANKET MAY BE THICKER THAN MODELED

4

DALLAS FLOODWAY PFM #6

NODE 2 – HEAVE @ TOE

MORE LIKELY	LESS LIKELY
• LANDSIDE CLAY BLANKET IS KNOWN TO BE THINNER IN THIS AREA (NEAR STA 220E)	• HEAVE F.S. CALCULATION DOES NOT ACCOUNT FOR STRENGTH OF CLAY

ESTIMATES

50% HT 0.01, 0.01, 0.1, 0.3, 0.1, 0.5, 0.5, 0.1, 0.5, 0.5, 0.1, 0.2, 0.1

75% HT 0.5, 0.6, 0.5, 0.4, 0.8, 0.5, 0.5, 0.6, 0.75, 0.5, 0.5, 0.5, 0.05

100% HT 0.8, 0.9, 0.1, 0.9, 0.8, 0.9, 0.7, 0.95, 0.9, 0.8, 0.8, 0.9, 0.8

RATIONALE – ~~TEAM PUT~~ WEIGHT GIVEN TO SEEPAGE ANALYSIS SINCE THESE ARE CONDITIONAL ON CLEAN SAND LAYER – BUT HISTORICAL BEHAVIOR REDUCED ESTIMATES SOMEWHAT.

3

DALLAS FLOODWAY PFM #8

NODE 2 - HEAVE AT TOE

~~[illegible]~~	LOW	BEST	HIGH
100% HT	0.7	0.8	0.9
75% HT	0.4	0.5	0.8
50% HT	0.01	0.1	0.5

MORE LIKELY	LESS LIKELY
• SEEPAGE ANALYSE SHOW LOW FACTORS OF SAFETY AGAINST HEAVE 50% 0.5-.6 75% 0.4-0.6 100% 0.4-0.5	• NO WATER ASSUMED IN SUMP ON DRY SIDE ~~CONSERVATIVE~~
• SUMPS NORMALLY PUMPED DOWN	• LOADING APPROACHING 50% HISTORICALLY PRODUCED NO PROBLEMS EL. 415 VS 417 @ BAKER
	• NATURAL DRAINAGE MAY OCCUR
	• CLAY BLANKET MAY BE THICKER THAN MODELED

4

DALLAS FLOODWAY PFM #8

NODE 2 - HEAVE @ TOE

MORE LIKELY	LESS LIKELY
• LANDSIDE CLAY BLANKET IS KNOWN TO BE THINNER IN THIS	• HEAVE F.S. CALCULATION DOES NOT ACCOUNT FOR STRENGTH OF CLA[illegible]

→ SAND EXPOSED IN BANK	RIVER TO FLOOD SIDE PIEZOMETER

DALLAS FLOODWAY PFM #8 HEAVE (CONT.) (2)

NODE 1 - CONT. CLEAN SAND LAYER CONNECTED TO WATER SOURCE

MORE LIKELY	LESS LIKELY
	• HAVE NOT OBSERVED AND SEEPAGE @ TOE TO DATE

ESTIMATES: 0.7, 0.7, 0.9, 0.6, 0.7, 0.75, 0.7, 0.6, 0.6, 0.7, 0.6, 0.6, 0.5

RATIONALE FOR ESTIMATE: COULD BE CIRCUITOUS PATH OF CLEANER SANDS BENEATH LEVEE - ALTHOUGH EVIDENCE HAS NOT SPECIFICALLY IDENTIFIED SUCH WITH CERTAINTY

DALLAS FLOODWAY PFM #8 HEAVE (1)

STATION 220 E SELECTED AS CRITICAL

NODE 1 - CONTINUOUS CLEAN SAND LAYER WITH WATER SOURCE

LOW	BEST	HIGH
0.5	0.7	0.9

MORE LIKELY	LESS LIKELY
• GEOMORPHOLOGY SUGGESTS DEPOSITION ENVIRONMENT CONDUCIVE TO LARGE SAND DEPOSITS	• GRADATION DATA INDICATES SMALL % SANDS CLEAN (5-10%)
• GOOD BORING COVERAGE SHOW SAND REGULARLY	• DEPOSITIONAL ENVIRONMENT SUPPORTS MIXING OF MATERIALS
• 3-D CHANNELS COULD CONNECT IN CIRCUITOUS FASHION	• BORINGS SUGGEST SAND LAYERS MAY PINCH OUT
• RIVER BOTTOM CUT INTO SHALE IN THIS AREA ⇒ SAND EXPOSED IN BANK	• PIEZOMETERS SHOW HEAD DROP FROM RIVER TO FLOOD SIDE PIEZOMETER

DALLAS FLOODWAY PFM #8 HEAVE (CONT.) (2)

NODE 1 - CONT. CLEAN SAND LAYER CONNECTED TO WATER SOURCE

MORE LIKELY LESS LIKELY

VERBAL DESCRIPTORS

VIRTUALLY CERTAIN	0.999
VERY LIKELY	0.99
LIKELY	0.9
NEUTRAL	0.5
UNLIKELY	0.1
VERY UNLIKELY	0.01
VIRTUALLY IMPOSSIBLE	0.001

Risk Assessment of Proposed Remediation Methods Trinity River Corridor Dallas Floodway

US Army Corps of Engineers®
Institute for Water Resources
Risk Management Center

2 November 2012

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Introduction

Alternatives Analysis Risk Assessment

An alternative analysis risk assessment was performed on the levees of the Dallas Floodway Project to determine the impact of imposing some recommended remediation methods on potential failure modes that were identified in the Base Condition Risk Assessment (BCRA) as being above tolerable guidelines.

This Base Condition Risk Assessment (BCRA) is a beta test of a proposed procedure for evaluating levee risk in more detail than the levee screening but in less detail than is required for a Levee Safety Risk Management Study. The results of this study are detailed in a report titled *Base Condition Risk Assessment, Trinity River Corridor, Dallas Floodway* dated 6 April 2012..

This BCRA provides a risk assessment for the Dallas Floodway base conditions. 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. 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 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. The levee embankments are generally comprised of lean clays and fat 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 thin basal sands and gravels. The basal sands and gravels mark the bedrock contact with the Eagle Ford Shale or the Austin Chalk.

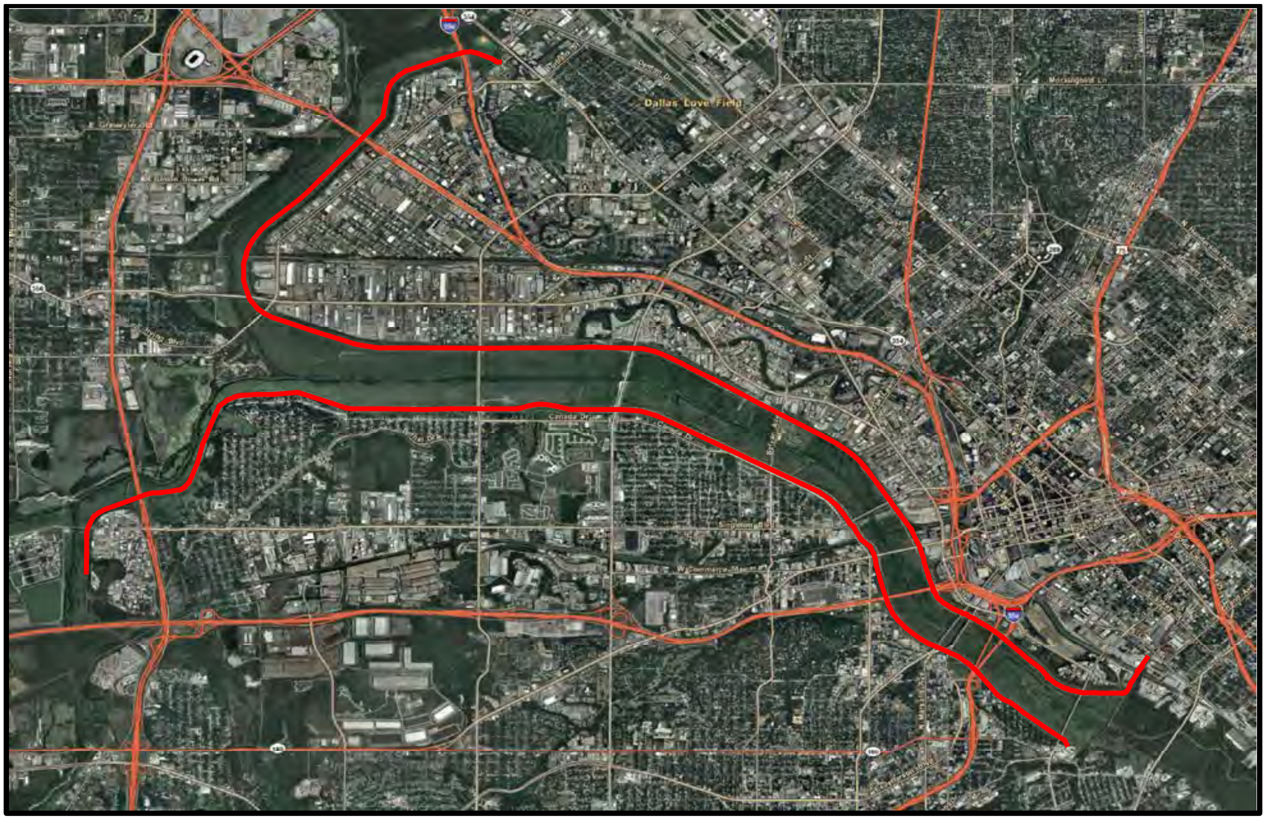


Figure 1 - East and West Levees that protect parts of Dallas shown in red.

Risk Assessment of Remediation Methods

On the east levee alignment, the BCRA report identified Potential Failure Modes (PFM's) being above recommended risk guidelines: PFM 2, overtopping, PFM 7, internal erosion, and PFM 8, heave (of the downstream toe). On the west levee alignment, PFM 2, overtopping, and PFM 7, internal erosion were identified as being above recommend risk guidelines. The f-N charts from the BCRA for both the east and west levee alignments are shown in Figures 2 and 3.

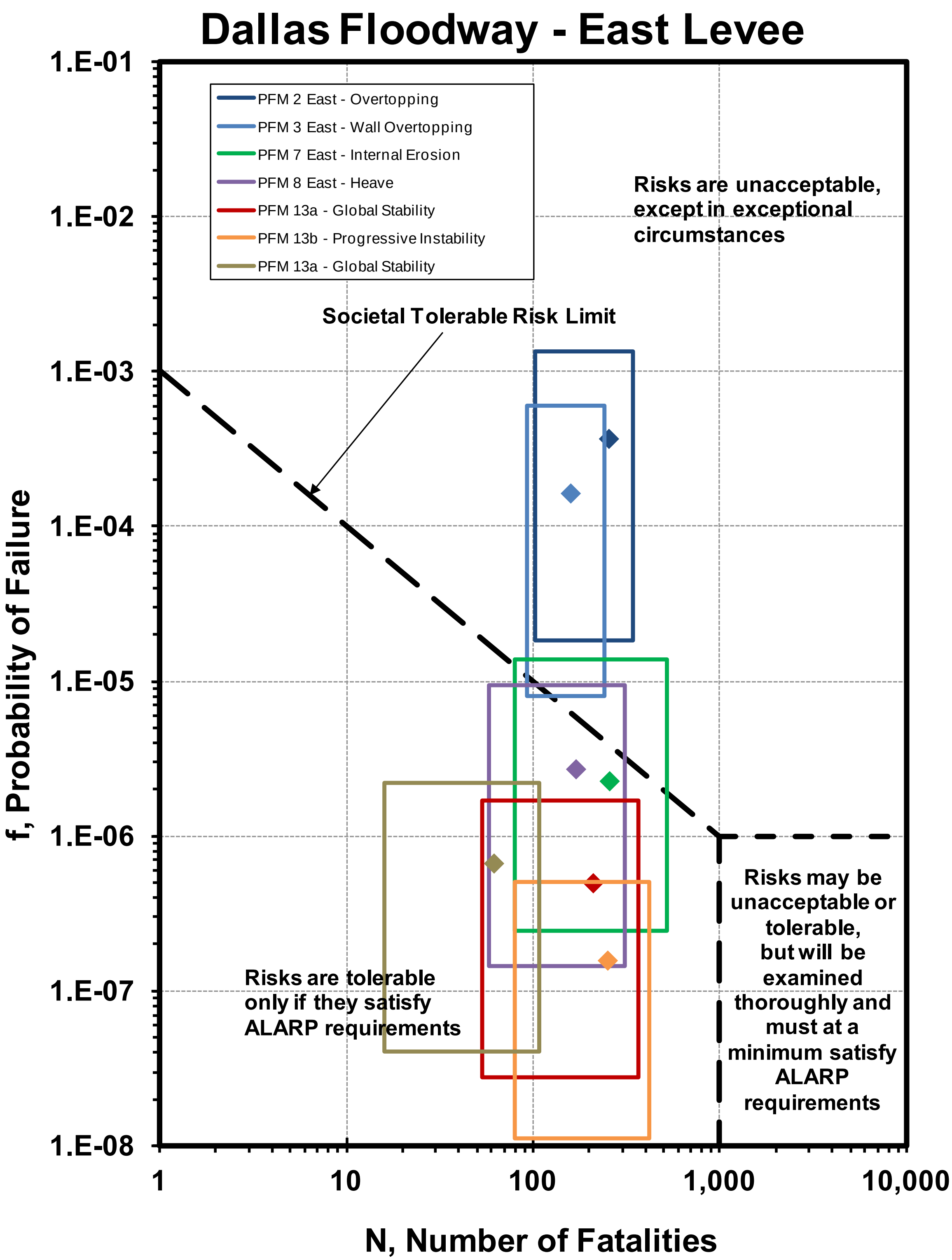


Figure 2. f-N plot of the East Levee from the BCRA report.

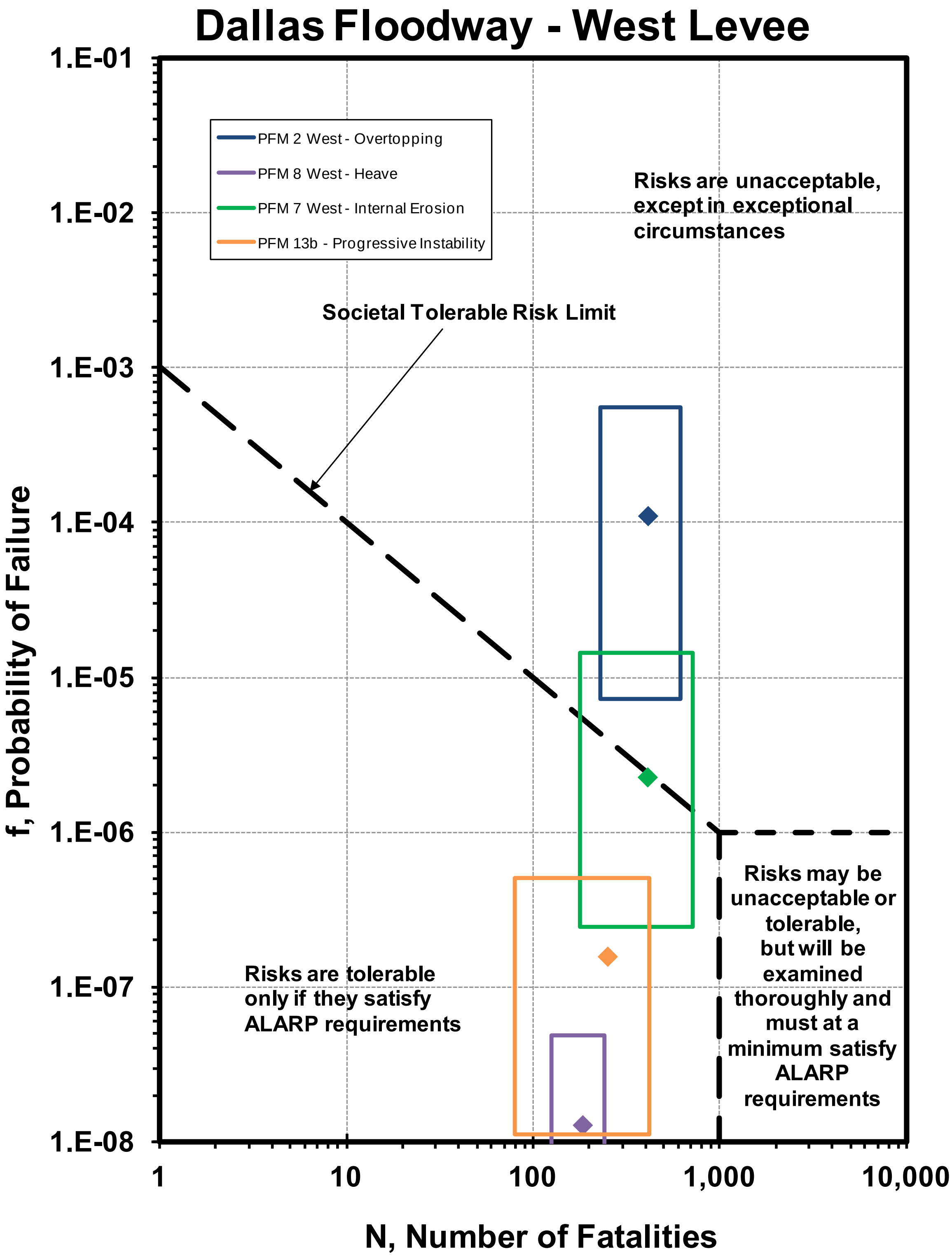


Figure 3. f-N plot of the West Levee from the BCRA report.

In response to these results, the Fort Worth District of the USACE suggested raising the height of the levee and levee armoring as two alternatives to address the PFM 2, overtopping. They also suggested two alternatives to address PFM 7, internal erosion and PFM 8, heave: a 3-ft thick sand filter berm to be placed on the land-side toe of the levee and extend approximately 300 ft from the levee toe (or as local conditions dictate in order to cover low lying areas) and soil-bentonite cutoff wall placed in front of the river-side toe of the levee. A soil-bentonite cutoff wall was also suggested by the City of Dallas. However, due to a lack of specific construction details at the time of the alternatives assessment, no distinction was made by the risk cadre in the behavior of the two proposed cutoff walls. Details of the remediation alternatives can be seen on the drawings included in Appendix D.

Due to real estate acquisition problems, it was decided not to pursue to sand filter berm alternative as it would require extensive real estate on the land-side of the levee and would be cost and time-prohibitive to actually put in place. Placement of the sand filter berm on the land-side of the levee would have also reduced the storage capacity of the sumps at the toes of the levees. The city has pump stations in place to remove any water impounded here, but the pumps are currently designed to rely on that storage capacity during large hydrologic events.

The remediation alternatives were accounted for in the risk assessment by re-assessing the risk at each node in the event tree the remediation would affect. For example, it was assumed that a cutoff wall would affect the “Sufficient Gradient to Erode Sand” node in the event tree for PFM 7, internal erosion. The risk cadre assembled for the alternative analysis was individually pooled on what their revised estimate of this node would be and their combined estimate was used to re-estimate the probability of failure. The risk estimates were collected during team meetings that took place from September 24 through 26, 2012. The members of the risk cadre are listed in Appendix E of this report.

Levee Raise

Levee raises of several different heights were considered in the alternatives analysis. The necessary levee raise heights were determined by estimating what height would be required to contain a hydrologic event of a certain magnitude. In this case, 6 different hydrologic events above the current threshold were considered while developing the alternatives. The threshold event is the hydrologic event at which the current levee would start to overtop. Currently this is an event that would induce a flow of 245,000 cfs in the Trinity River in the project area.

During the September team meeting, it was decided to select three of the six different hydrologic events to analyze the remediation methods against: storm events that would induce a flow in the Trinity River of 260,000, 277,000, and 302,000 cfs. In order to appropriately estimate the impact on the risk, a separate event tree was put together to address each levee raise height. Each event tree has branches that reflect the performance of the levee raise under several different amounts (depths) of overtopping. Each overtopping depth will be classified as either minor or major overtopping, keeping with how this analysis was performed in the BCRA. Major overtopping was defined in the BCRA as any overtopping depth greater than 2.2 ft on the east alignment and 1.3 ft on the west alignment. The minor overtopping depth is anything below these levels. Using this delineation, the actual nodal assessments for overtopping from the BCRA can continue to be used for this analysis.

Due to the fact that the crests of the levees along the Trinity River have settled to varying heights, a levee raise would actually be filling in low spots along the levees as some points on the levee are already at or above the necessary heights. The extent of the levee raises was determined using a survey of the existing levee crest heights. Fill would be placed in the areas that were are low depending on the survey.

Levee Armoring

Levee armoring was assumed to be done using articulated concrete block (ACB). This was an assumption made that helped to determine what nodal estimates would be changed and whether or not any nodes would need to be added to the existing BCRA event trees. Due to the fact that the crests of the levees along the Trinity River have settled to varying heights, one would expect that the entire east and west levee alignments would not need to be armored, only the low spots. Therefore, levee armoring of several different extents were considered in the alternatives analysis. The necessary extent of armoring was determined by how much of the levee crest would be overtopping in a hydrologic event of a certain magnitude. It was assumed that ACB would be placed wherever overtopping was occurring. As for the risk assessment of implementing levee raises, three different hydrologic events above the current threshold out of the six shown in the drawings were considered: 260k, 277k, and 302k cfs.

In addition to the consideration of extent of armoring, the size of the concrete block in the ACB was also considered. The team assumed that the overtopping protection designed for each water level, whether it's for 260k, 277k, or 302k cfs, has been designed to have a probability of failure of 0.001 when exposed to overtopping flows equal to or less than it has been designed for. The risk assessment team selected a probability of failure of 0.001 because they realized that there was still some chance of failure at the design overtopping flow (probability of failure is not zero), but it would be very low. The armor that has been designed to handle 260k is likely composed of smaller block. The armor design of 302k, however, would be composed of larger block and would have more anchorage.

The team realized that ACB has some natural resiliency to being overtopped beyond its designed amount (depth) of overtopping, but at some point would be subject to failure. To address this issue, a node was added to the PFM 2, overtopping event tree for the Initiation of Mat Failure. This node is in addition to Intervention Fails and Breach Forms. The team elicited estimates of initiation of mat failure for hydrologic events that are larger than what the armoring was designed for.

As we discussed, some areas of the levee will not be armored because they sit at a higher elevation than other sections of the levee. When a hydrologic event greater than what the armoring protection was designed occurs, overtopping will occur in these areas. To model the behavior of these unprotected areas in the event trees, nodal estimates for major and minor overtopping from the BCRA will be used. The depths of overtopping in the exposed areas assumed during the BCRA will be the same, but the length of levee crest that will be exposed to overtopping will be longer (because there were no armored areas in the BCRA). However, length effects will not be addressed at this time, similar to the BCRA.

Soil-Bentonite Cutoff Wall

The soil-bentonite cutoff wall primarily addresses PFM 7, internal erosion, but will also have implications on PFM 8, heave of the downstream toe. Essentially, the cutoff will be placed 10 to 25 ft in front of the toe of the levee in the locations of where internal erosion is thought to be a concern. The wall will be advanced from the ground surface and be keyed into bedrock to completely cutoff any continuous sand layers that extend under the levee cross-section to the land side. If there are any surficial sand layers between the top of the proposed cutoff wall and the flood-toe of the levee that leave a pathway for seepage exposed to floodwaters, it is assumed that an impermeable soil cap will be placed on the ground surface.

During the BCRA, an area on the east alignment was selected to be representative of a levee section susceptible to this failure mode. For the risk assessment of the cutoff wall, we will modify the event tree from the BCRA. It was assumed during the September 2012 meeting that the basal sand layers were 100% continuous. The effect of placing the cutoff wall will primarily be accounted for in the “Sufficient Gradient to Erode Sand (Erosion Initiates)” node.

Unlike the event trees for PFM 2, overtopping, the event trees for PFM 7 used hydrologic loading with a maximum surface elevation equal to 50, 75, and 100% of the crest elevation of the levee. The team elicited new risk assessments for the “Sufficient Gradient to Erode Sand” node at all these water levels, but decided to elicit assessments for the 75 and 50% flood heights as a group based on their estimates for the 100% levee height condition. This brought the probability of failure within the acceptable limits for this failure mode. There may be some additional effect to the “Flow Limiter” and “Early Intervention” nodes, but risk was net re-assessed for these nodes because the probability of failure for PFM 7 was already reduced below guidelines. The “Heroic Intervention” and “Flow Limiter” nodes are not used in the DFW event trees, but it’s anticipated that additional benefit will be gained from the cutoff wall in these areas. Therefore, the team decided it was not necessary to re-evaluate any additional nodes as these would only further reduce the probability of failure.

PFM 8, Heave

The results of the risk analysis performed for the BCRA indicates (Figure 2) that range of uncertainty for PFM 8 extends into the envelope of unacceptable criteria for the east levee alignment. Seepage and stability analyses carried out in support of this report and the BCRA indicate the factor of safety against heave ranges below 1.2 in some areas. However, that does not necessarily mean that heave will occur. Inherent conservatism is built into the criteria relating to heave that indicates that even if sand boils or other physical indicators of heave show up, it's unlikely that this will lead to failure of the levee system. The sinuous nature of the sand lenses that are considered point bar deposits that have been successively laid down and eroded away may or may not continuously extend under the levee cross-section from upstream to downstream and would not lend itself to conducting significant amounts of seepage. The deposits also lend themselves to creating poor conditions for roof support in some areas and can provide ample material that would act as a crack stopper. For all these reasons, the team decided not reassess the risk for this failure mode. Paramount among these reasons, the best estimate of the behavior of the levee against this type of failure is below guidelines, as is the majority of the uncertainty.

Uncertainty

Uncertainty was modeled with a similar approach used in the BCRA report, using distributions contained in @Risk. For the hydrologic loading, the Log Pearson Type III flow frequency curve along with analytical uncertainty bounds was used as the basis for the distributions.

For each event tree branch, the team estimates were used to quantify the uncertainty. For all distributions used in the alternatives analysis a normal distribution was selected. The standard deviation was calculated as normal but the ends of the normal distribution curve were truncated at the team's minimum and maximum estimate, in accordance with how the distribution was calculated in the BCRA,. Figure 4 shows an example of what that distribution looks like for a single event tree branch. These distributions were used and included in a Monte Carlo simulation with 10,000 runs per failure mode.

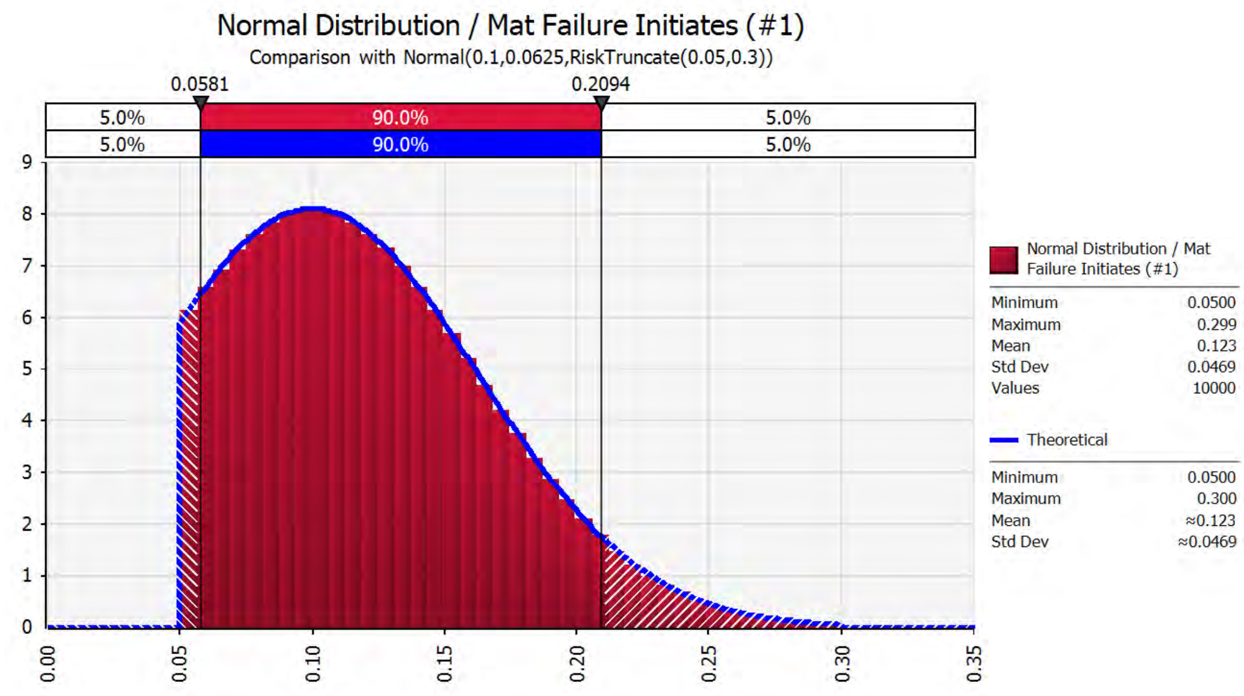


Figure 4 - Example Truncated Normal Distribution

Major Findings from Risk Calculations

The expert elicitation data taken from the September 2012 meeting was used to analyze event trees modified from the BCRA to reflect the addition of the risk reduction alternatives using the @Risk and Precision Tree software. The results are shown on the f-N plots in the following figures. Figures 5 and 6 show the results of placing levee lifts and armoring as compared to the revised BCRA estimates for PFM 2, overtopping, for the east and west levee alignments, respectively. Please note, that the results for all the remediation methods shown are not additive. They show only the affect of adding each individual remediation method to the original condition. Figure 7 shows the result of placing a cutoff wall upstream of the levee toe as compared to the revised risk estimate from the BCRA for PFM 7, internal erosion. The original risk estimates from the BCRA were revised to reflect modifications to the components of the flood control system and changes in methodology for calculating hydrologic frequency and breach modeling. Details of these revisions can be found in Appendix A.

The f-N plots shown in Figures 5 through 7 depict dotted lines on each figure that are labeled as “Societal Tolerable Risk Limit for Dams”. It should be noted that these are tolerable risk guidelines that have been accepted for dams as defined in ER 1110-2-1156, Safety of Dams – Policy and Procedures. These same limits don’t necessarily apply to levee safety and should be considered more of a context for making decisions in relation to other levee systems in the USACE levee portfolio.

The analysis shows some reduction in risk (in terms of Annualized Loss of Life) for levee raise alternatives. This risk reduction is gained primarily by lowering the frequency of overtopping events but it is offset by increased Loss of Life for overtopping events due to later warning and evacuation orders of the population. Note that this is not a general statement for all levee systems. Additionally, the analysis does not account for long term risk in that levee raise actions can encourage additional development in the floodplain, which increases the risk.

The results indicate that the addition of a cutoff wall placed at the flood-side toe of the levee will lower the risk against PFM 7, internal erosion below guidelines. The placement of levee lifts or armoring, however, do not lower the risk below guidelines with regard to PFM 2, overtopping.

The focus of this study was to evaluate the impacts of the specific individual alternatives. The study did not attempt to suggest alternatives or combinations of alternatives that would be most effective at reducing the risk. Actions such as providing for a defined, armored overtopping location may be very effective at reducing the risk; but consideration of these actions was beyond the scope of this study.

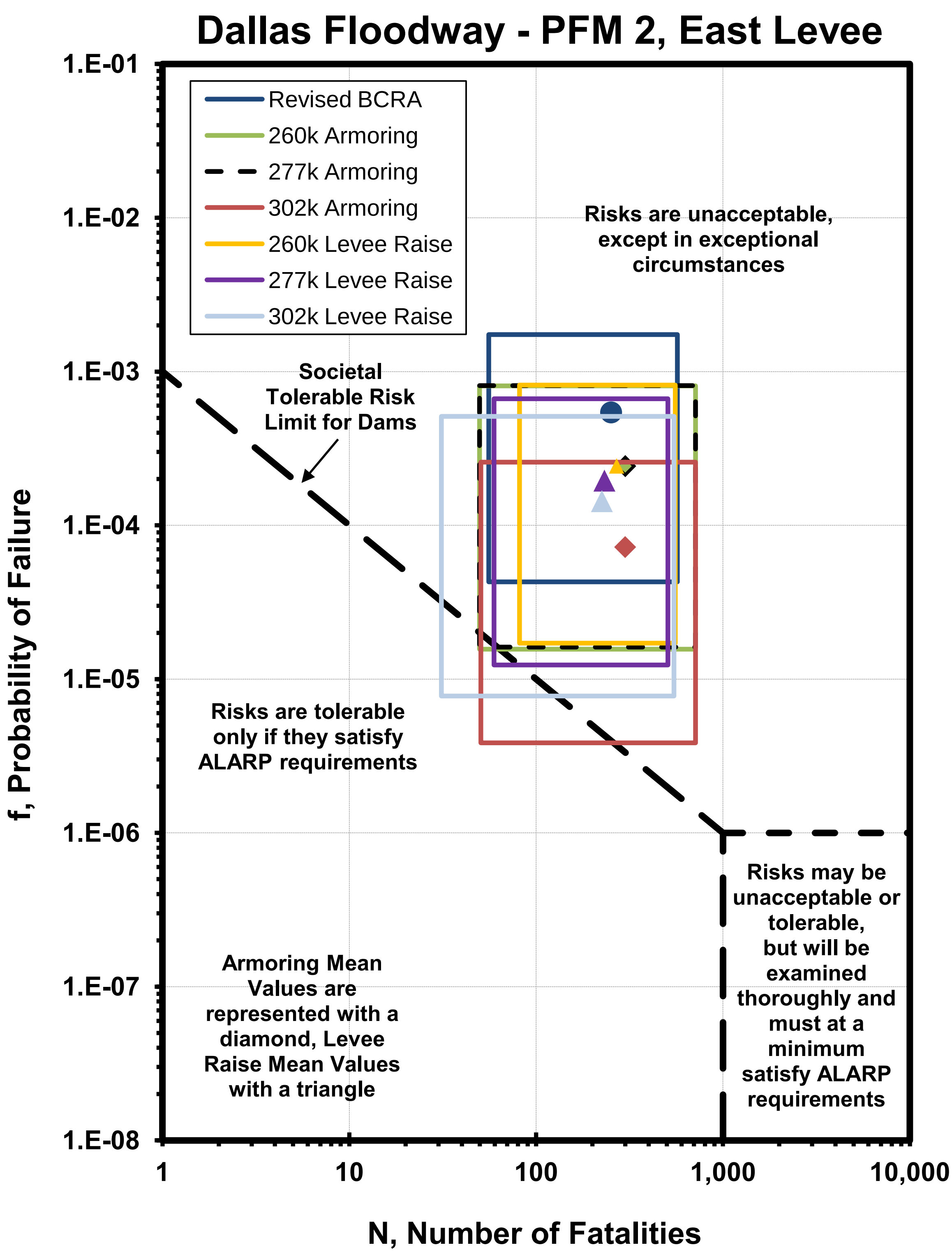


Figure 5. f-N plot for PFM 2, Overtopping, on the east levee alignment.

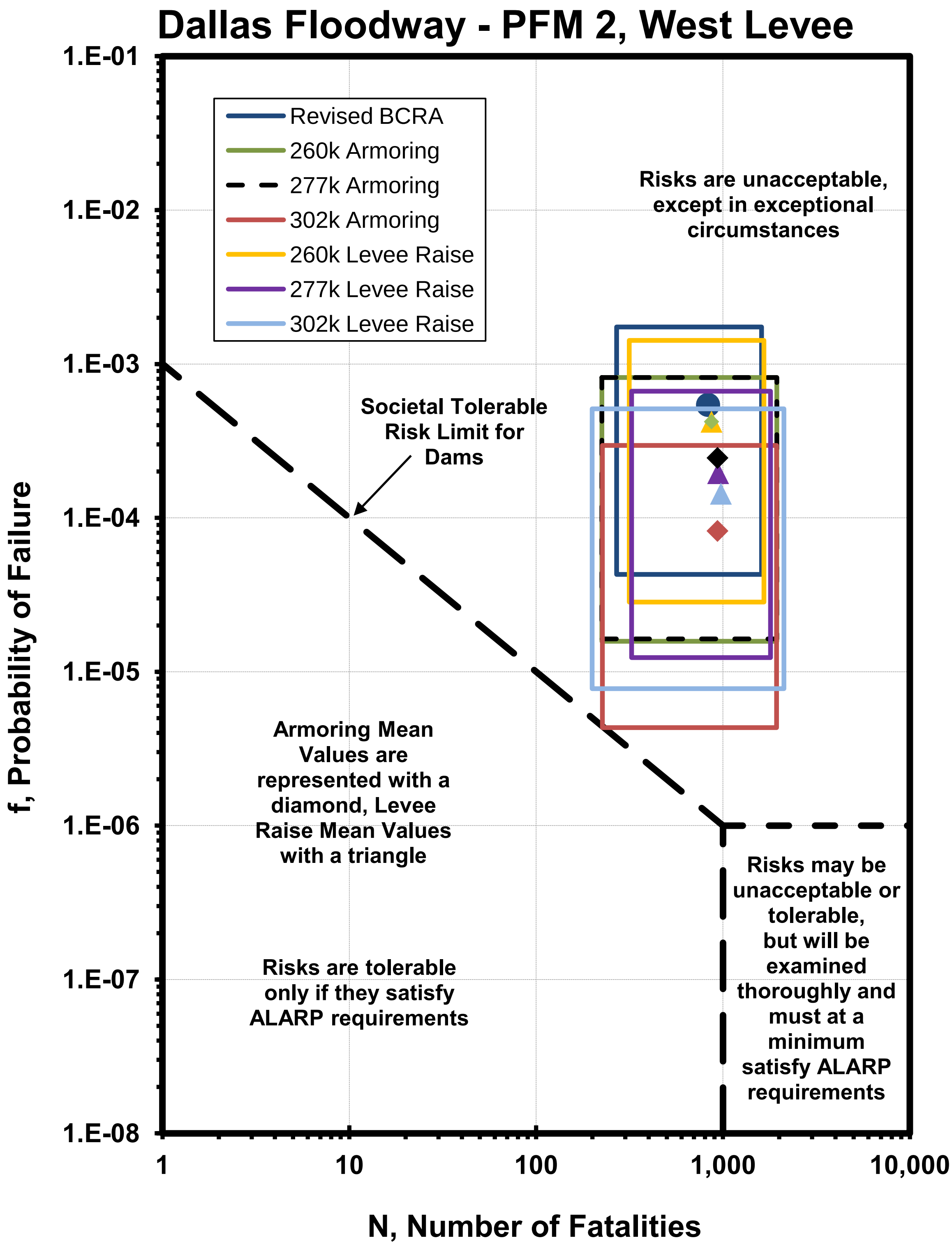


Figure 6. f-N plot for PFM 2, Overtopping, on the west levee alignment.

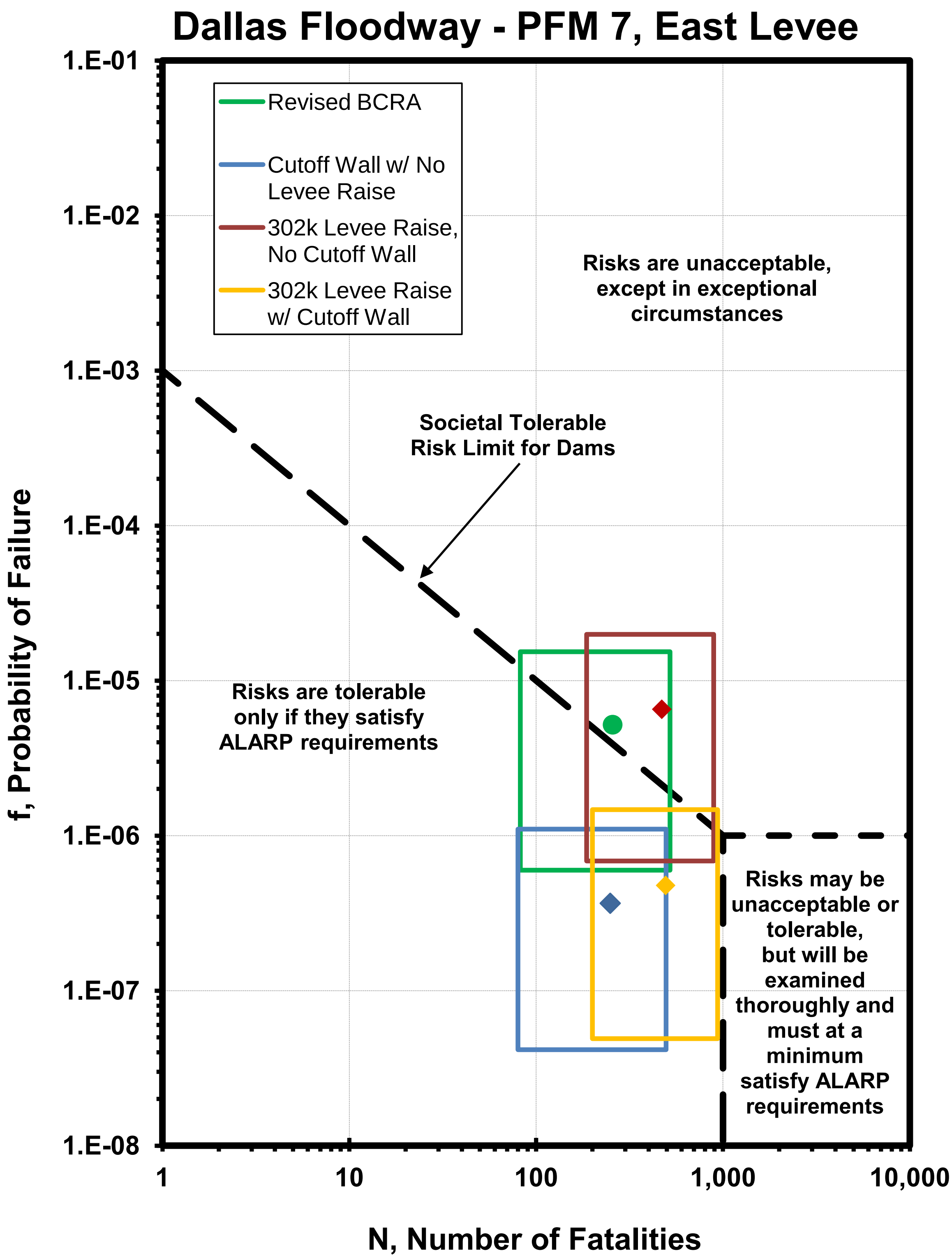


Figure 7. f-N plot for PFM 7, Internal Erosion, on the east levee alignment.

Appendix A – Hydrologic Modeling and Analysis

Purpose

Hydraulic and hydrologic analyses were performed to support the total (loss of life) risk assessment of the proposed alternatives measures associated with the current Dallas Floodway Feasibility Study. The risk assessment focused on 3 possible types of alternative measures: levee raises, levee armoring, and internal erosion mitigation.

The work is a follow-on study to the Base Condition Risk Assessment (BCRA) for the Dallas Floodway which was performed November 2011 through February 2012. The approach to the current study was consistent with the previous work.

It should be noted that the scope of the risk assessment was solely to assess the current alternatives that are being considered. The decision was made that the risk assessment would not propose possible alternative measures that would be the most effective in terms of reducing the risk to loss of life for the floodway system.

Assumptions

Modeling system

The hydraulic modeling was performed using HEC-RAS in unsteady mode. The general application is described in the BCRA report.

Baseline Model Geometry

The baseline c0ndition for the current study assumes that the Dallas Floodway Extension (DFE) project is in place. The HEC-RAS geometry was obtained from the Fort Work District and is identical to that being utilized for the feasibility study efforts. This differs from the BCRA work in two significant ways:

1. The left bank floodwall tie-in (BCRA primary overtopping location) is no longer part of the system as it would be effectively cut-off from the line of protection.
2. Some downstream channel improvements, including removal of a railroad bridge are incorporated into the system.

Hydrograph shapes

The current study used a standard project flood (SPF) patterned hydrograph for all hydraulic model runs. This is a departure from the BCRA, where there was discussion regarding the impacts of possible long duration floods on the internal erosion failure modes. The results of the BCRA showed that any increased probability of internal erosion failures was offset by the decreased probability of experiencing a long-duration flood. The SPF pattern is considered the most representative shape for the large magnitude floods considered in the risk analysis.

Overtopping Breach Locations

The BCRA considered 3 probable overtopping breach locations: The concrete floodwall and the initial overtopping location for both the east and west levee reaches. For the current risk analysis, the floodwall is not considered. For the levee raise alternative measures, the “low spots” are essentially filled in and the initial levee overtopping locations are more difficult to identify.

For the purposes of this study, it was assumed that the breach locations would not change regardless of the levee raises. Generally, the inundation depths would be higher if the breach were to form on the upstream reaches versus the downstream reaches. The breach locations (near the low spots) used for the BCRA, and carried forward for the current study are generally near the middle reaches for both the east and west alignments. These locations provide consequence results that will be somehat representative for the system, i.e. they are not biased toward either high or low consequence results.

Simplified Physical Breach Modeling for Overtopping

Recently, HEC-RAS has incorporated a “simplified physical” breach option that ties the erosion of an embankment to the hydraulic conditions at the breach location. This option has been used for the Dallas Floodway Feasibility Study economic analysis and was also applied to the current risk assessment for all overtopping breach scenarios. The erosion rate versus velocity relationship was determined by the Fort Worth District in collaboration with staff from the RMC, MMC, and HEC.

The method was not applied for the breach prior to overtopping failure modes. The application of this method for an internal erosion type failure mode includes additional assumptions regarding the trigger for failure and the time required for the failure to develop. These assumptions are being developed by the Fort Worth District, but were not available for the current risk assessment modeling. For these scenarios, the assumptions used for the BCRA were applied (150-foot wide breach, fully developing in 6-26 hours).

Hydrologic Frequency Curve

The hydrologic frequency curve at Dallas has been a topic of discussion for the USACE Hydrology Committee and additional analysis to finalize the frequency curve is ongoing. An interim Log-Pearson Type III analytical frequency curve was adopted for the current risk analysis. The curve is defined by the mean, standard deviation, skew, and equivalent years of record values of 4.31, 0.302, 0.2, and 50, respectively.

Summary of Model Runs

Multiple model runs were performed in order to estimate consequences of failure (life-loss) for a range of breach and non-breach scenarios. The consequence runs are summarized below. Note that the event trees used to compute the quantitative risk used fewer consequence nodes than what was available from the hydraulic and consequence modeling.

Breach Prior to Overtopping

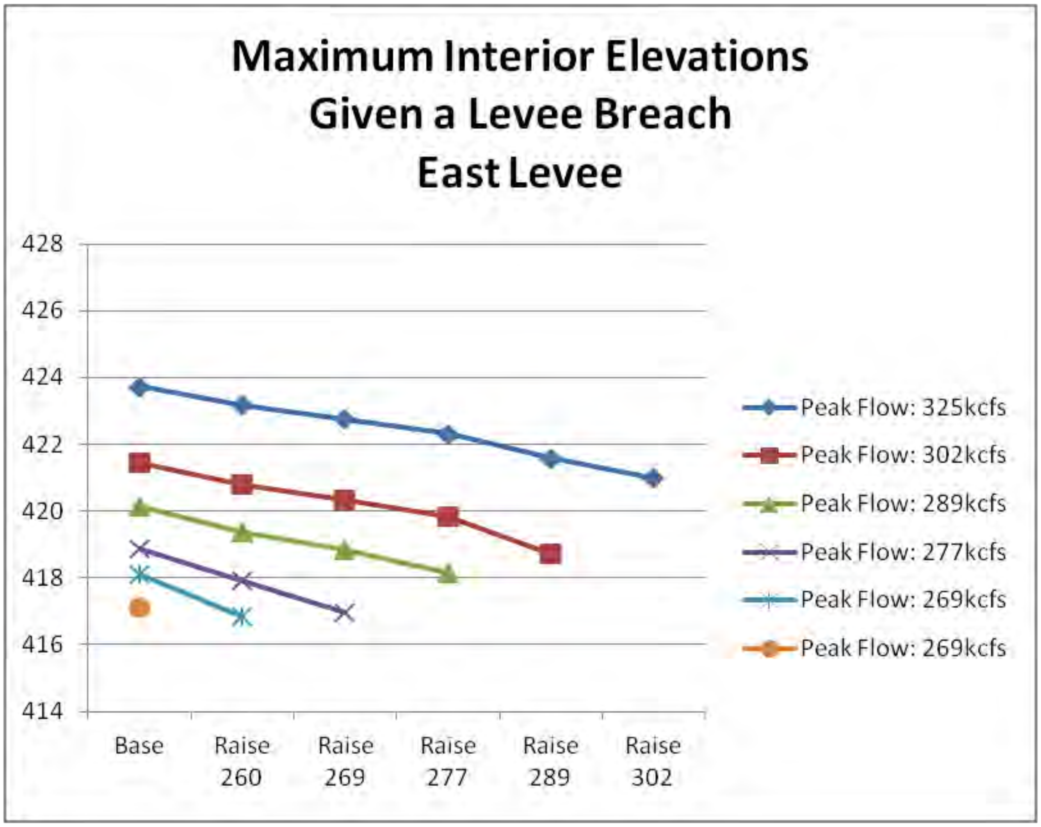
- 8 Failure Mode Locations (identified in BCRA)
- 4 Hydraulic Loads
    - ½ Levee (no raise)
    - ¾ Levee (no raise)
    - Full Levee (no raise)
    - Full Levee (302kcfs Raise)
- **32 Total Runs**

Overtopping

- 6 Hydraulic Loads
    - 260 kcfs
    - 269 kcfs
    - 277 kcfs
    - 289 kcfs
    - 302 kcfs
    - 325 kcfs
- 3 Failure Scenarios
    - East Breach
    - West Breach
    - No Breach
- 6 Levee Configurations
    - Baseline
    - Raise to contain 260 kcfs
    - Raise to contain 269 kcfs
    - Raise to contain 277 kcfs
    - Raise to contain 289 kcfs
    - Raise to contain 302 kcfs
    - Raise to contain 325 kcfs
- Minus 40 scenarios that do not overtop the levee
- **63 Total Runs**

Interior Depth Results for Overtopping with Breach

For overtopping with breach model scenarios, the maximum interior water surface elevation for a identical river flood event.were reduced for each incremental levee raise. This happens primarily because the breach (triggered by overtopping depths) initiates later in the hydrograph and less time is available for water to pass through the breach before the flood recedes. The figure below illustrates the model results.



Appendix B – Consequence Modeling and Analysis

Purpose

Consequence and loss of life analyses were performed to support the total risk assessment of the proposed alternative measures associated with the current Dallas Floodway Feasibility Study. The risk assessment focused on 3 possible types of alternative measures: levee raises, levee armoring, and internal erosion mitigation.

The work is a follow-on study to the Base Condition Risk Assessment (BCRA) for the Dallas Floodway which was performed November 2011 through February 2012. The approach to the current study was consistent with the previous work.

It should be noted that the scope of the risk assessment was solely to assess the current alternatives that are being considered. The decision was made that the risk assessment would not propose possible alternative measures that would be the most effective in terms of reducing the risk to loss of life for the floodway system.

Assumptions

Modeling system

The consequence modeling was performed using HEC-FIA 2.1 and post-processing spreadsheets. The general application is described in the BCRA report.

Loss of Life assumptions

The structure inventory, and associated population at risk, was left unchanged from the BCRA analysis. Loss of life parameter assumptions were also left unchanged in most cases. For instance, mobilization and fatality thresholds are identical for the BCRA overtopping events and this effort. Warning issuance assumptions are kept relative to breach and are unchanged for overtopping events; and internal erosion scenario warning assumptions were assumed to use the same framework as the BCRA (warning after breach unless the levee is near overtopping).

Consequence analyses under best case, worst case and most likely conditions were performed and later utilized to estimate the mean loss of life estimate.

Hydraulic conditions impact on loss of life

Given identical loss of life assumptions, differences in results between the BCRA estimates and the analyzed alternatives are driven by hydraulic factors. Changes in arrival times influence the percentage of the total population at risk that is ultimately unable to evacuate. Changes in maximum depths influence the fatality rates applied to unmobilized population at risk.

Summary of Model Runs Performed

Multiple model runs were performed in order to estimate consequences of failure (life-loss) for a range of breach and non-breach scenarios. The consequence runs are summarized below.

Breach Prior to Overtopping

- 8 Failure Mode Locations (identified in BCRA)
- 4 Hydraulic Loads
  - ½ Levee (no raise)
  - ¾ Levee (no raise)
  - Full Levee (no raise)
  - Full Levee (302kcfs Raise)
- **32 Total Runs**

Overtopping

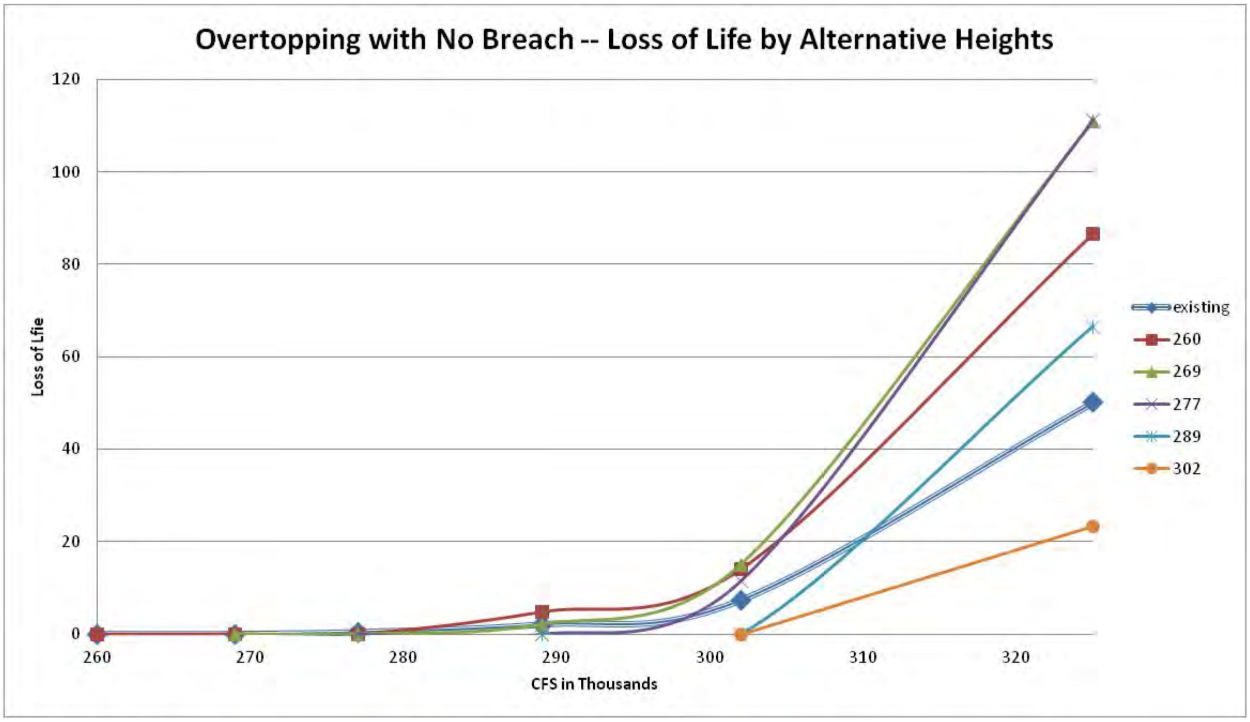
- 6 Hydraulic Loads
  - 260 kcfs
  - 269 kcfs
  - 277 kcfs
  - 289 kcfs
  - 302 kcfs
  - 325 kcfs
- 3 Failure Scenarios
  - East Breach
  - West Breach
  - No Breach
- 6 Levee Configurations
  - Baseline
  - Raise to contain 260 kcfs
  - Raise to contain 269 kcfs
  - Raise to contain 277 kcfs
  - Raise to contain 289 kcfs
  - Raise to contain 302 kcfs
  - Raise to contain 325 kcfs
- Minus 40 scenarios that do not overtop the levee
- **63 Total Runs**

Summary of Loss of Life Results

Results are displayed below for analyzed runs (those included in BCRA are not displayed here).

Alternative Measure	Breach Condition	Hydrologic Load	Best Case Expected LoL	Most Likely Expected LoL	Worst Case Expected LoL	Mean Value
Existing	Non-Fail	260k	0	0	0	0
Existing	Non-Fail	269k	0	0	0	0
Existing	Non-Fail	277k	0	0	2	0
Existing	Non-Fail	289k	0	0	12	2
Existing	Non-Fail	302k	0	0	42	7
Existing	Non-Fail	325k	2	22	214	50
Existing	East Levee OT Failure	260k	3	16	214	47
Existing	East Levee OT Failure	269k	4	20	289	63
Existing	East Levee OT Failure	277k	5	29	424	91
Existing	East Levee OT Failure	289k	9	42	733	151
Existing	East Levee OT Failure	302k	14	79	1151	249
Existing	East Levee OT Failure	325k	26	111	1630	347
Existing	West Levee OT Failure	260k	24	174	685	234
Existing	West Levee OT Failure	269k	34	245	1355	392
Existing	West Levee OT Failure	277k	43	300	1671	489
Existing	West Levee OT Failure	289k	54	381	2110	615
Existing	West Levee OT Failure	302k	67	460	2723	775
Existing	West Levee OT Failure	325k	88	586	3475	987
Raise to contain 260k	Non-Fail	260k	0	0	0	0
Raise to contain 260k	Non-Fail	269k	0	0	0	0
Raise to contain 260k	Non-Fail	277k	0	0	0	0
Raise to contain 260k	Non-Fail	289k	0	0	28	5
Raise to contain 260k	Non-Fail	302k	0	1	80	14
Raise to contain 260k	Non-Fail	325k	3	30	396	87
Raise to contain 260k	East Levee OT Failure	269k	3	15	225	48
Raise to contain 260k	East Levee OT Failure	277k	4	21	362	75
Raise to contain 260k	East Levee OT Failure	289k	7	37	719	146
Raise to contain 260k	East Levee OT Failure	302k	11	63	1369	269
Raise to contain 260k	East Levee OT Failure	325k	24	115	2185	447
Raise to contain 260k	West Levee OT Failure	269k	33	239	1294	378
Raise to contain 260k	West Levee OT Failure	277k	42	297	1704	481
Raise to contain 260k	West Levee OT Failure	289k	53	378	2222	631
Raise to contain 260k	West Levee OT Failure	302k	67	457	2675	762
Raise to contain 260k	West Levee OT Failure	325k	88	592	3849	1054
Raise to contain 269k	Non-Fail	269k	0	0	0	0
Raise to contain 269k	Non-Fail	277k	0	0	0	0
Raise to contain 269k	Non-Fail	289k	0	0	12	2
Raise to contain 269k	Non-Fail	302k	0	1	86	15
Raise to contain 269k	Non-Fail	325k	3	34	525	111
Raise to contain 269k	East Levee OT Failure	277k	3	16	252	53
Raise to contain 269k	East Levee OT Failure	289k	5	31	595	121
Raise to contain 269k	East Levee OT Failure	302k	10	56	1011	209
Raise to contain 269k	East Levee OT Failure	325k	22	109	2121	427
Raise to contain 269k	West Levee OT Failure	277k	34	250	1490	421
Raise to contain 269k	West Levee OT Failure	289k	50	346	2064	582
Raise to contain 269k	West Levee OT Failure	302k	63	443	2860	778
Raise to contain 269k	West Levee OT Failure	325k	86	586	4150	1107

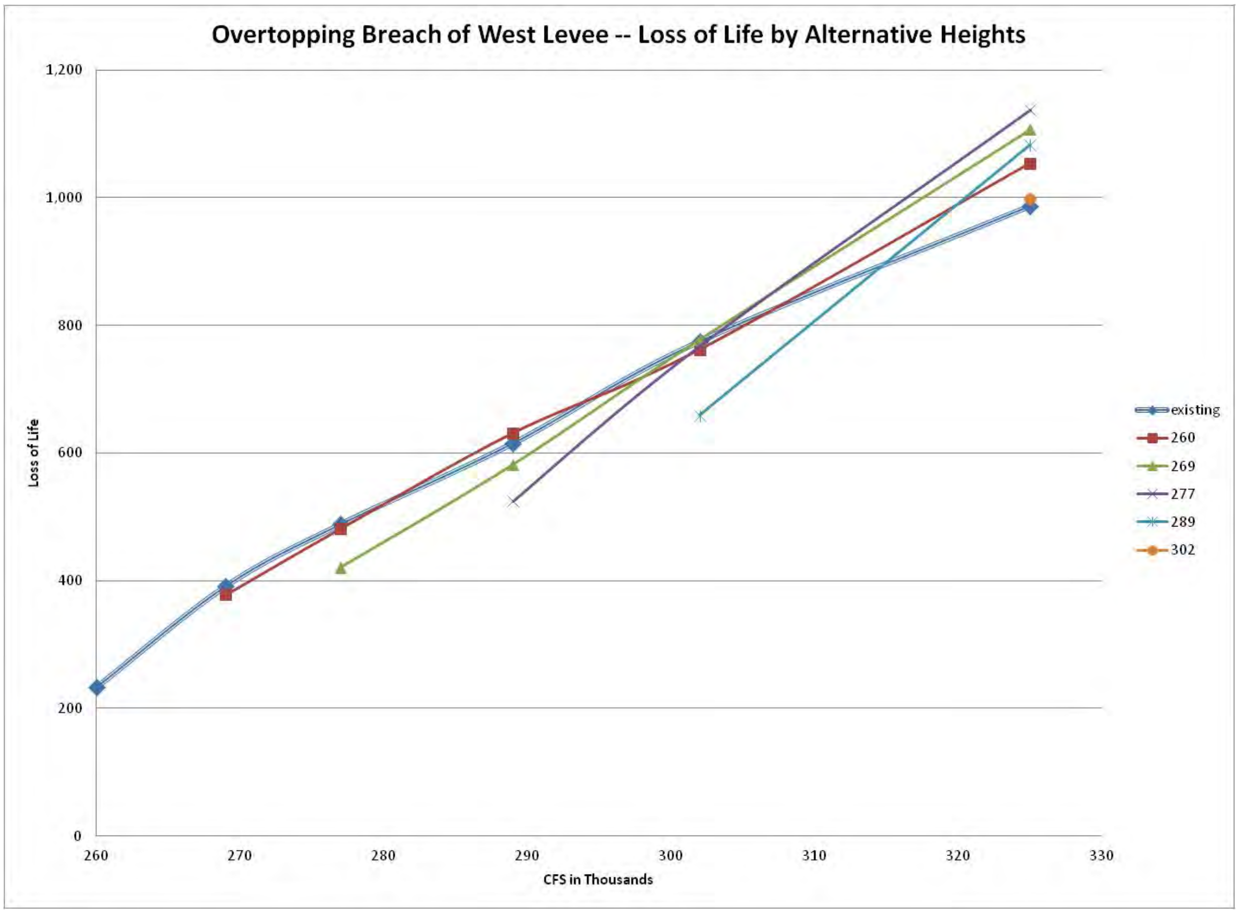
Alternative Measure	Breach Condition	Hydrologic Load	Best Case Expected LoL	Most Likely Expected LoL	Worst Case Expected LoL	Mean Value
Raise to contain 277k	Non-Fail	277k	0	0	0	0
Raise to contain 277k	Non-Fail	289k	0	0	0	0
Raise to contain 277k	Non-Fail	302k	0	1	66	11
Raise to contain 277k	Non-Fail	325k	3	33	540	111
Raise to contain 277k	East Levee OT Failure	289k	4	22	395	82
Raise to contain 277k	East Levee OT Failure	302k	8	42	795	163
Raise to contain 277k	East Levee OT Failure	325k	19	102	1870	386
Raise to contain 277k	West Levee OT Failure	289k	44	311	1860	525
Raise to contain 277k	West Levee OT Failure	302k	59	421	2882	768
Raise to contain 277k	West Levee OT Failure	325k	85	579	4407	1137
Raise to contain 289k	Non-Fail	289k	0	0	0	0
Raise to contain 289k	Non-Fail	302k	0	0	0	0
Raise to contain 289k	Non-Fail	325k	1	17	334	67
Raise to contain 289k	East Levee OT Failure	302k	5	28	527	108
Raise to contain 289k	East Levee OT Failure	325k	15	83	1580	323
Raise to contain 289k	West Levee OT Failure	302k	50	358	2475	659
Raise to contain 289k	West Levee OT Failure	325k	81	554	4142	1083
Raise to contain 302k	Non-Fail	302k	0	0	0	0
Raise to contain 302k	Non-Fail	325k	0	2	131	23
Raise to contain 302k	East Levee OT Failure	325k	12	63	1222	246
Raise to contain 302k	West Levee OT Failure	325k	73	507	3885	999
Generic Levee Raise	IE Failure	302k	10	30	381	84
Generic Levee Raise	IE Failure a	302k	31	126	1498	340
Generic Levee Raise	IE Failure a	302k	37	134	1368	321
Generic Levee Raise	IE Failure a	302k	41	144	1484	349
Generic Levee Raise	IE Failure	302k	16	109	492	157
Generic Levee Raise	IE Failure a	302k	69	470	2781	791
Generic Levee Raise	IE Failure a	302k	77	547	2953	870
Generic Levee Raise	IE Failure a	302k	89	616	2266	801
Generic Levee Raise	IE Failure	Old Full Levee	15	36	350	85
Generic Levee Raise	IE Failure	Old Full Levee	112	258	4108	886
Generic Levee Raise	IE Failure	Old Full Levee	136	248	2892	680
Generic Levee Raise	IE Failure	Old Full Levee	121	275	2864	686
Generic Levee Raise	IE Failure	Old Full Levee	32	87	328	119
Generic Levee Raise	IE Failure a	Old Full Levee	204	473	2805	815
Generic Levee Raise	IE Failure a	Old Full Levee	245	552	3300	960
Generic Levee Raise	IE Failure a	Old Full Levee	174	476	3042	855



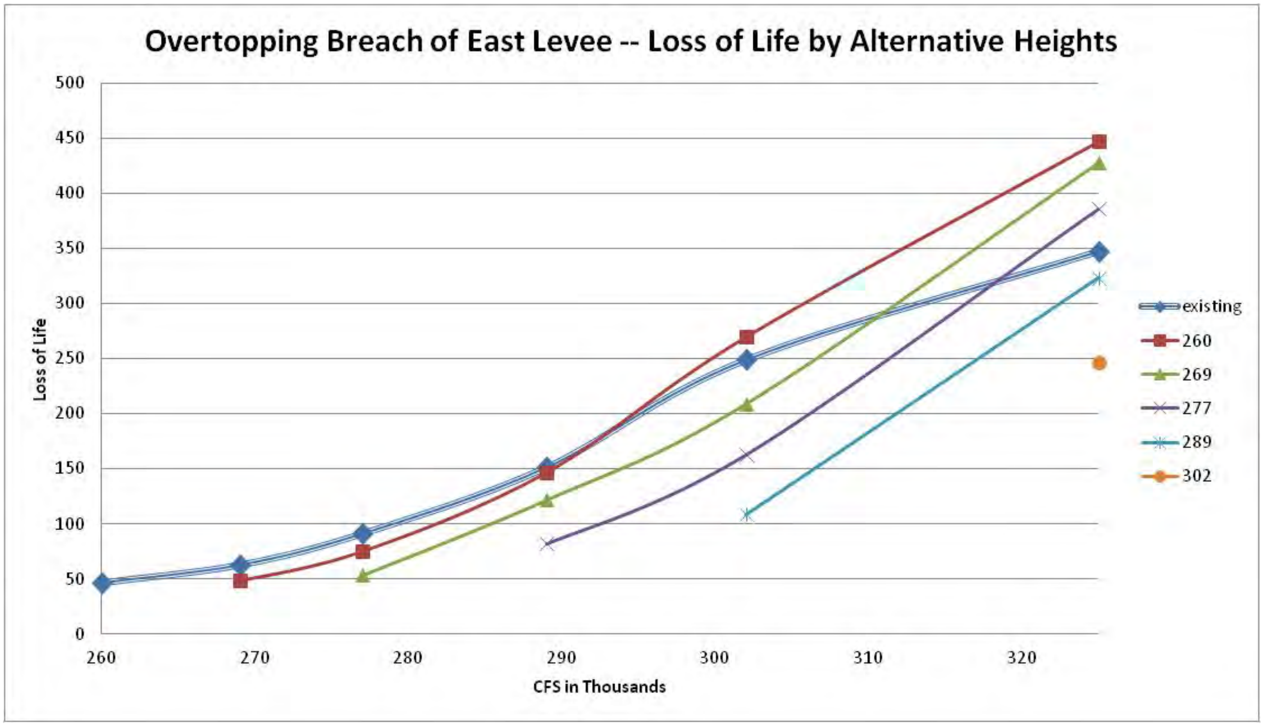
The above graph demonstrates that non-breach loss of life generally increases for levee raises. This is somewhat counterintuitive as we might expect lower depths as the levees are able to contain a larger percentage of the flood; and such a trend is indeed present for events that only slightly exceed the system's capacity, for example, the 302k raise reduces loss of life from the 325k flood event.

However, higher consequences may be seen for certain flood events with certain raises in place. This is because existing low spots in the levee profiles are disproportionately located on the east levee, which means levee raises will displace some floodwaters that would have otherwise have gone into eastern leveed area into the western leveed area instead. The eastern leveed area is more resilient to low-depth floods because the PAR is able to vertically evacuate within tall structures. The western leveed area largely contains 1-story residential structures and many residents in the area are elderly who are less likely to evacuate out of the hazard area and less likely to be able to vertically evacuate within their structure.

While certain events may see slightly higher consequence with a raise in place, this is not to say that a levee raise would increase non-breach *risk*. Indeed, a levee raise would reduce the probability of overtopping significantly, more than offsetting any risk caused by increased consequences.



For many scenarios, levee raises have minimal effect on consequences of a western levee breach. Consequences are slightly lower for relatively low-intensity floods, as the levees are able to contain more of the flood before ultimately breaching; this reduces the duration for which river stage would exceed the breach bottom elevation post-breach. However, for the 325k event modeled, a levee raise may slightly increase consequences. This is partly due to the phenomenon discussed in the non-breach section, but the increased levee height also results in a more extreme breach, decreasing the time from breach to flood arrival at structures and increasing maximum depths seen at structures.



Levee raises were generally shown to decrease the consequences of breach. Again, with more low spots, the east levee benefits more from a levee raise than the western leveed area. Because much of the modeled storm has passed the study area before the levee is ultimately overwhelmed, there is less cumulative flow present to equalize depths.

Appendix C – Preliminary Supporting Seepage Analysis

Seepage Mitigation Alternatives

Seepage analyses were carried out to explore the effect that four potential remediation measures would have on seepage and stability behavior of the Dallas Floodway (DFW). These analyses were done in support of remediation measures designed by the Fort Worth District to address deficiencies identified in the previous Risk Analysis of the DFW conducted by the RMC and the Fort Worth District.

Currently, the City of Dallas is constructing a cutoff wall along portions of the DFW levee alignments. This cutoff wall is designed to cut off any continuous sand layers that may penetrate below the levees. Continuous sand layers were identified in the previous risk analysis as potential pathways for seepage to penetrate the foundation soils beneath the levees, potentially leading to a decrease in the stability of the levees, an increased potential for internal erosion under the levees, and an increase in protected side seepage. Based on plans and specifications provided by the City of Dallas, the cutoff wall is to be placed within the floodway, a minimum of 25 ft in front of (away from) the flood side toe of the levee. The cutoff wall will be approximately 3 ft wide and will extend from the ground surface through the alluvial soils and will penetrate at least 5 ft into the Eagle Ford Shale/Austin Chalk bedrock layer. The specifications indicate that the cutoff wall would have a permeability no greater than 2.5e-4 ft/hour. It is unclear what the specific criteria that the City of Dallas used to determine where on the alignment the cutoff wall would be placed and where it would not.

To supplement the City of Dallas cutoff wall, the Fort Worth District has designed three seepage mitigation measures to further stabilize the DFW levee system. These three measures include:

A. A weighted sand seepage berm placed on the protected side of the levee. This would provide a filter on the downstream side of the levee that would pass seepage but inhibit any material from eroding from the surface of the levee or from the downstream ditch. This alternative was analyzed in the seepage models with and without the City of Dallas cutoff wall.
B. A soil-bentonite cutoff wall placed on the flood side of the levee. This would essentially be the same as the City of Dallas cutoff wall, except it would be placed approximately 5 ft from the flood side toe of the levee instead of 25 ft. It is designed to cut off the flow of seepage through the continuous sand layers, decreasing exit gradients and downstream seepage. Thickness, penetration, and permeability were assumed to be the same as the City of Dallas cutoff wall. It is the understanding of the seepage modeler that this

particular alternative would not be used in conjunction with the City of Dallas cutoff wall.

C. A flood side clay cap on the ground surface over the City of Dallas cutoff wall. This alternative would only be used in areas where there is sand exposed on the flood side surface of the floodway. The proposed clay cap would provide a horizontal impermeable blanket at the ground surface to inhibit floodwaters from penetrating the floodway surface in front of the levee, bypassing the cutoff wall, and reaching a subsurface basal sand unit. There is also a measure in this alternative to add fine-grained material to the flood side surface of the levee to decrease the embankment slope to a maximum of 4:1.

As part of the previous risk analysis done for the DFW, extensive seepage and stability modeling was performed on eight levee sections. Four sections were analyzed from the west bank and four from the east bank. In order to carry out the seepage and stability analyses for the alternatives analysis as quickly and efficiently as possible, some of these same sections were utilized for this analysis. On the east bank they are located at stations 311+00 and 410+00. On the west bank they are located at stations 10+00, 188+00, and 335+00. These sections were selected based on the locations/extents of the proposed mitigation alternatives as they were provided in the drawings from the Fort Worth District.

Analysis Assumptions

- Transient seepage analyses were conducted, similar to what was used for the seepage analyses in support of the risk analysis.
- The analysis sections mentioned above are sufficient to adequately represent the performance of the DFW in the locations where remediation measures are proposed.
- The levee raises and armoring proposed as part of the remediation effort were not investigated using seepage or stability analyses.
- The original risk cadre’s best estimates of material strength and permeability were used in the analyses.
- Desiccation depths (where surface materials are classified as CH) will be 5 ft on the levee surface, 10 ft in the free field, similar to the analyses carried out for the original risk analysis.
- The hydrographs that were used for the analyses assumed that the floodway water level reached the top of the levee. The original hydrographs were used for this effort. Analyses assuming that water level reached no more than ½ or ¾ of the levee were not conducted.
- Based on plan drawings from the City of Dallas, their cutoff wall is located 25 ft from the flood side toe of the levee towards the center of the floodway. It is 3 ft thick and penetrates 5 ft into bedrock.

- On Drawing C-501, Seepage Alternative Template “B” shows a window in the bedrock directly behind the proposed cutoff wall. This is a misprint; the cutoff wall is placed neat against the bedrock on both the upstream and downstream sides.
- The cutoff wall from Alternative B is assumed to be located 5 ft from the flood side toe of the levee.
- The clay cap placed on the ground surface in Alternative C is assumed to be 2 ft thick and extend 5 ft past the cutoff wall towards the center of the floodway.

Discussion of Results

Base Case

The majority of the base case stability analyses started out with a relatively high factor of safety. That is, most of the un-remediated sections met stability requirements (FoS >= 1.5) with the exception of the section at station 10+00 on the west levee alignment. The estimated seepage values for the base case analyses were minimal as well. The estimated vertical exit gradients at the downstream toe of the embankment sections, however, ranged from 0.15 to > 4.0.

The gradients calculated by SEEP/W and reported here are the X-Y Gradients and the results should be interpreted with care. The X-Y Gradient values that are displayed by SEEP/W are an estimate of the gradients at each integration point (node) in the direction of seepage flow (which is not necessarily in the vertical, Y, or horizontal, X, direction, but a combination, X-Y, of both). The gradient calculations depend on several factors:

- How thick the clay layer overlying the continuous basal sand layers was on the protected side of the levee. If the overlying clay layer was thin (on the order of 1 to 3 ft), the gradients were usually high. This was the case with the sections at stations 410+00 East and 10+00 West.
- If there is exposed sand on the protected side, then seepage can safely evacuate the embankment and foundation soils, resulting in some downstream seepage but a relatively low gradient. This is the case with the sections at stations 311+00 East and 188+00 West.
- The gradient calculation is influenced by the elevation of the protected side ground surface. Some sections with low estimates of surface gradient had relatively high protected side ground elevations, so much so that the piezometric surface was barely touching or just below the ground surface so that little to no seepage was actually coming out of the ground. This was the case with the section at station 335+00 West.
- The gradient calculation can be influenced by piezometric surfaces substantially below the protected side ground surface. This could provide an artificially inflated value of surface gradient. Soils above the piezometric grade line are exposed to negative pore water pressure values, or suction, up to a user-defined limiting value. Higher values of

suction result in a calculated gradient that is high but is not indicative of impending erosion or initiation of backward erosion piping as these soil elements are above the piezometric grade line thus have no seepage flowing through them. This typically occurred in the investigation when the remediation alternatives were analyzed and the flow of seepage was reduced thus lowering the protected side piezometric surface. The validity of the gradient calculation must be judged on a case-by-case basis with the graphical outputs thoroughly examined.

The estimated factors of safety against heave at the downstream toe should also be used with care. The heave FoS was only reported for sections that have a clay layer overlying a continuous sand layer on the protect side, i.e., all sections except for 311+00 East and 188+00 West. This calculation was heavily influenced by the thickness of the overlying clay layer (the thicker the better) and ground surface elevation on the protected side.

Alternative A

The addition of Alternative A, the downstream seepage berm, generally did a good job of reducing the surface gradient on the downstream side of the embankment and increasing the factor of safety against heave. The obvious exception to this was the sections that had exposed sand at the ground surface on the downstream side (stations 311+00 East and 188+00 West). The increase in surface gradient and heave FoS was due to the draining/filtering capacity and the additional weight of the sand seepage berm, respectively. If the analysis sections had a protected side ground surface that was relatively low in elevation (stations 10+00 and 335+00 West), the additional weight of the berm served to increase the stability of the section.

When Alternative A was analyzed with the City of Dallas cutoff wall in place, the stability of each section was further improved. Not only were the benefits of the extra weight of the seepage berm on the protected side ground surface retained, but the amount of seepage passing below the levee was drastically reduced. This served to increase the FoS against heave to above unity in all cases by lowering amount of head in the continuous sand layers pushing on the bottom of all the downstream clay layers. The addition of the cutoff benefited stability in most cases as well by reducing the pore water pressure in the continuous sand layers, thereby increase in the effective stress and thus the frictional resistance (shear strength) of the sand layers. In the section where stability was not increased, the critical failure surface did not penetrate into the basal sand layers. Gradient was low in most sections with just the addition of the seepage berm so it was largely unaffected with the addition of the cutoff wall. However, at the section at station 335+00 West, the piezometric surface was lowered so much that the downstream ground surface was exposed to suction and the gradient was subsequently increase giving the false impression of a more critical section.

Alternative B

Alternative B was very similar to the City of Dallas cutoff wall. The only difference between the two is that the Alternative B cutoff wall is 20 ft closer to the flood side toe of the levee (being located only 5 ft from the toe towards the center of the floodway). The Alternative B cutoff wall was assumed to have the same penetration into bedrock, thickness, and permeability as the City of Dallas cutoff wall. Therefore, this analysis was similar the analysis for Alternative A with the City of Dallas cutoff wall but with the seepage berm removed and the cutoff wall moved closer to the levee embankment. With a cutoff wall still in place, the effect of removing the seepage berm was not substantial except where the additional weight of the berm increased stability, such as in the sections at stations 10+00 and 335+00 West. The gradient was increased at the sections at station 410+00 East and 10+00 and 188+00 West due to the fact that the protected side ground surface was lowered 3 ft (with the removal of the berm), bringing it into contact with (stationary) piezometric grade line.

Alternative C

Alternative C was only examined where there was exposed sand at the ground surface in the location of the City of Dallas cutoff wall. This only occurred in the section at station 311+00 East. The difference between Alternatives B and C (essentially a run with a cutoff wall but no clay cap, B, and a run with a cutoff wall and a clay cap, C), however, is negligible. This is primarily because there was an approximately 1-ft thick naturally occurring clay layer that intersected the Alternative B cutoff wall (meaning there was never a direct connection of the floodwaters with the continuous sand layer below the levee) so the two runs were nearly identical. It is expected that if the naturally occurring clay layer or the man-made clay blanket was not in place, the results would be very similar to the base case with slightly lower stability, slightly higher downstream surface gradient, and a higher rate of seepage. This illustrates the point that a layer of material as little as one foot thick but having a permeability several orders of magnitude lower than surrounding materials can heavily influence the performance of the embankment if it remains intact. This should also serve as a warning that if that same thin, low permabililty layer is being relied upon to enhance the performance of an embankment, any sort of small defect can jeopardize the stability of that section.

General Comments

It should be mentioned that this analysis does not address the suitability of the extents of the proposed remediation measures. The seepage and stability analysis done in support of the original risk analysis identified the sections at stations 220+00 East and 10+00 West as the critical sections. Remediation measures are only proposed in the area of the section at station 10+00 West. It is recommended that the seepage and stability analyses generated during the previous investigation be used to help determine the extents of the remediation efforts. Similarly, the extents should not be based on the prevalence of continuous sand layers in bore holes and CPT soundings alone. The extent of repairs should be based on wide changes in geology and depositional environment, not on the point-by-point prevalence of high permeability soils in individual bore holes. Any time repairs are recommended over a short distance or are repeatedly started and stopped over and over again within the same relative area, these areas should be re-examined and consideration given to installing the remedial measures over the entire area.

To determine the applicability of the section at station 410+00 East to the remediation measures proposed on the Elm Fork branch of the east levee alignment, the HNTB roll plots were examined. Roll plots are figures that contain a plan view of an area showing boring and CPT sounding locations, with three longitudinal cross-sections that correspond to the plan views drawn at the center of the levee alignment and at the flood and protected side toes of the embankments. They indicated that the basal sand layers in the Elm Fork branch are approximately 25 to 30 ft deeper, relatively thinner, and potentially more sinuous than the sands observed in the section at station 410+00. It can be generally be said that the seepage and stability conditions at station 410+00 are likely of a more critical nature, which makes the results conservative when applied to Elm Fork branch, but this neglects any differences in cross-sectional shape, prevalence of protected side clay layers, levee alignment, or hydrologic differences that may affect the performance.

To determine the applicability of the section at station 335+00 West to the remediation measures proposed on the West Fork branch of the west levee alignment, the HNTB roll plots were examined. They indicated that the basal sand layers in the West Fork branch are approximately the same depth over much of the West Fork, relatively thinner, and likely not as continuous than the sands observed in the section at station 335+00, though the sands in West Fork are likely continuous in some areas. It can generally be said that the seepage and stability conditions at station 335+00 are likely of a more critical nature than those in the West Fork branch areas. The bedrock and subsequently the basal sand layers appear much closer to the ground surface from station 432+00 to 475+00, however, and these areas may be more critical than the section used to analyze them. Regardless, this neglects any differences in cross-sectional shape, levee alignment, or hydrologic differences that may affect the performance.

Appendix D – Participants

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Study of the Impact on Risk of the Proposed Balanced Vision Plan and Trinity Parkway

Trinity River Corridor Dallas Floodway

US Army Corps of Engineers®
Institute for Water Resources
Risk Management Center

26 June 2013

Table of Contents

Introduction

Balanced Vision Plan and Trinity Parkway Risk Assessment

A Base Condition Risk Assessment (BCRA) was previously performed for the Dallas Floodway Project to determine the risk of flood inundation to the population at risk around the Trinity River in Dallas, Texas. The results of this study are detailed in a report titled *Base Condition Risk Assessment, Trinity River Corridor, Dallas Floodway* dated 6 April 2012. The BCRA provides a risk assessment for base conditions. Base conditions include only currently existing measures that are taken during floods and are in accordance with normal operation.

The City of Dallas is considering modifying the existing Trinity River Corridor as proposed in the Balanced Vision Plan (BVP) and by constructing the Trinity Parkway. These changes will alleviate traffic congestion in the downtown area while also making the floodway more usable for the public. These two individual proposals are components of the city's larger Comprehensive Plan. Any modification of the existing flood protection, however, would impact the overall risk from flooding that was estimated during the BCRA. In order to capture any changes in the estimated risk, a portion of the Risk Estimating Team (RET, made of the individuals listed in Appendix C) reconvened to perform a subsequent risk assessment. The results of that risk assessment are detailed in this report.

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 left descending (East) and right descending (West) banks of the Trinity River in metropolitan Dallas.

Background

Project Description

The Dallas Floodway project is a complex system of flood risk reduction measures that includes levee embankments, a concrete floodwall, sumps and pumping stations, bridge crossings, conduits, and other penetrations. A brief description of the project is included here.

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 high water levels on the Trinity River, while the West levee protects a large portion of West Dallas (largely residential areas). The alignment of the East and West Levees are shown in Figure 1. These embankments were originally constructed by the City of Dallas 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. The levee embankments are generally comprised of lean clays and fat 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 thin 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 red.

Balanced Vision Plan Description

The Balanced Vision Plan (BVP) is a proposal for developing the floodway of the Trinity River Corridor. There is a substantial amount of green space between the East and West levee alignments along the stretch known as the Trinity River Corridor that the City of Dallas would like to develop for public recreational use. The plan calls for significant physical changes to the channel and floodway including restoration of channel meanders, creation of a mid-channel island, alterations to channel geometry, creation of several lakes, and general enhancement of aquatic and riparian habitat throughout the corridor. Designs for the BVP are provided in the Feasibility Report Appendix D – Civil and Structural Design.

Implementation of the BVP would require excavation and filling of the floor of the floodway in various locations. These excavations would be for the proposed West Dallas Lake, Natural Lake, Urban Lake, and Oxbow Lake. Excavations would also be performed to change the existing linear alignment of the river channel in an effort to create more naturally appearing river meanders.

Trinity Parkway Description

The Trinity Parkway is a proposed toll road located within the floodway of the Trinity River Corridor. The primary purpose of the Trinity Parkway project is to provide a transportation solution to manage traffic congestion and improve safety in the area of the Dallas Central Business District (CBD).

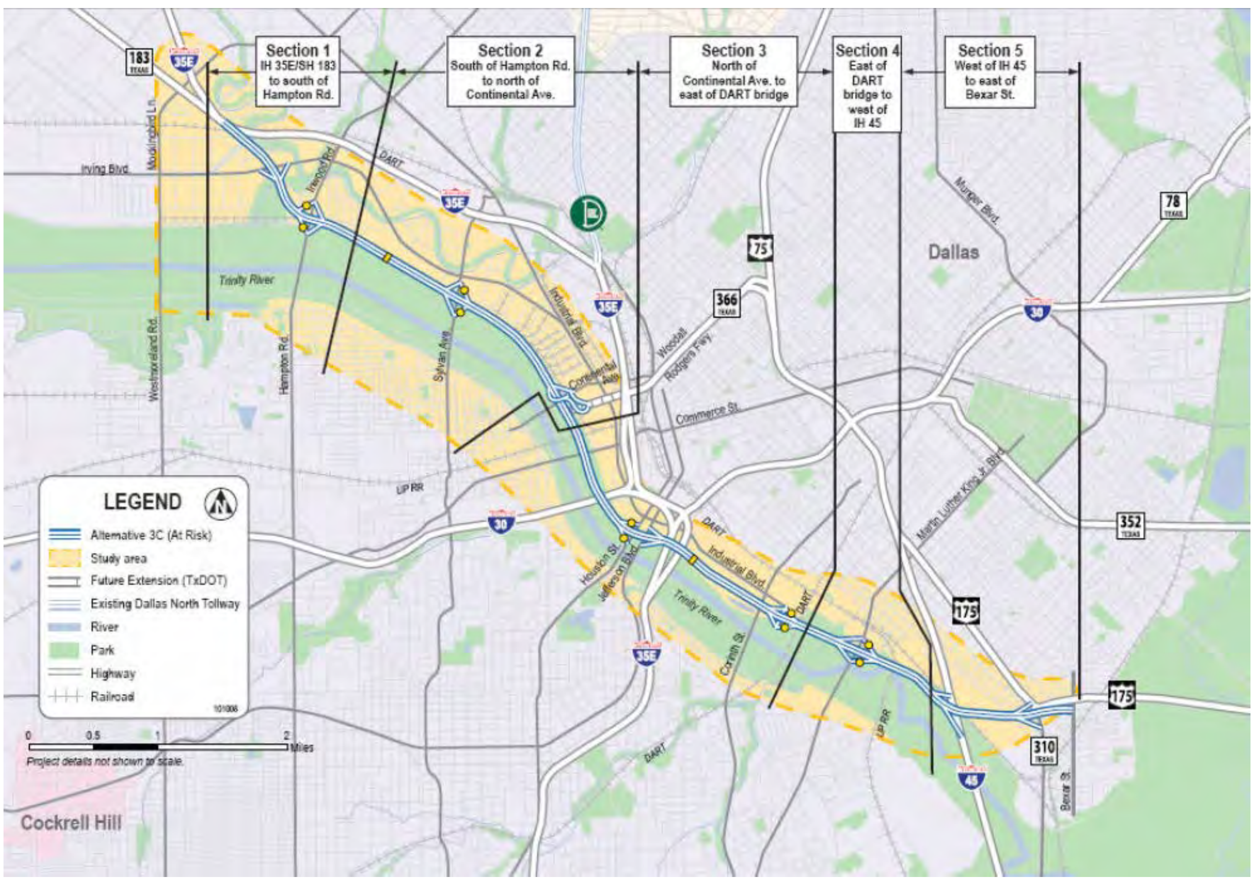

Figure 2. Plan view of the Dallas Floodway showing the Trinity Parkway along the east levee alignment.

The proposed parkway would be located along the riverside of the east levee alignment, with the main lanes placed on an earthen embankment set above the 100-year flood level along most of its length to prevent inundation of the road during high water events. The roadway will enter the floodway at the upstream end of the Trinity River Corridor near the Hampton Pump Station, passing over the levee as a pile supported structure. Some pile elements being advanced through the embankment. The roadway will exit the floodway downstream of the limits of the study area. Where the proposed alignment meets existing bridge crossings of the Dallas Floodway (from Continental Avenue to the DART bridge), the roadway grade will depress to allow traffic to pass below the existing structures. At these locations, a separation wall along the riverside of the toll road would prevent the roadway from being inundated during a 100-year flood event. Pump stations would provide drainage in these low areas during high water events.

The Trinity Parkway roadway will sit on an approximately 250-ft wide earth embankment that will be constructed within the floodway, directly against the riverside of the East Levee. The embankment will be constructed of spoil material excavated for the proposed Trinity Lakes and the relocated Trinity River alignment.

Risk Assessment

Original Baseline Condition Risk Analysis

The BCRA report identified the following potential failure modes (PFM's) as being at or above the recommended risk guidelines: PFM 2, overtopping, PFM 7, internal erosion, and PFM 8, heave (of the landside toe). These results are represented graphically on the f-N charts from the BCRA for the east and west levee alignments, included here as Figures 3 and 4. All stationing referenced in this report uses the same stationing system as the BCRA.

Comprehensive Plan Components and Impact on Risk

The BVP and the Trinity Parkway are large, complicated projects having many components, all with an abundance of individual details. For the purposes of this study, the RET focused only on those elements of the BVP and the Trinity Parkway that could impact risk from the failure modes investigated during the BCRA. Those elements included the relocation of the current river alignment to introduce meanders within the floodway, excavation of the proposed lakes, and placement of the Trinity Parkway roadway.

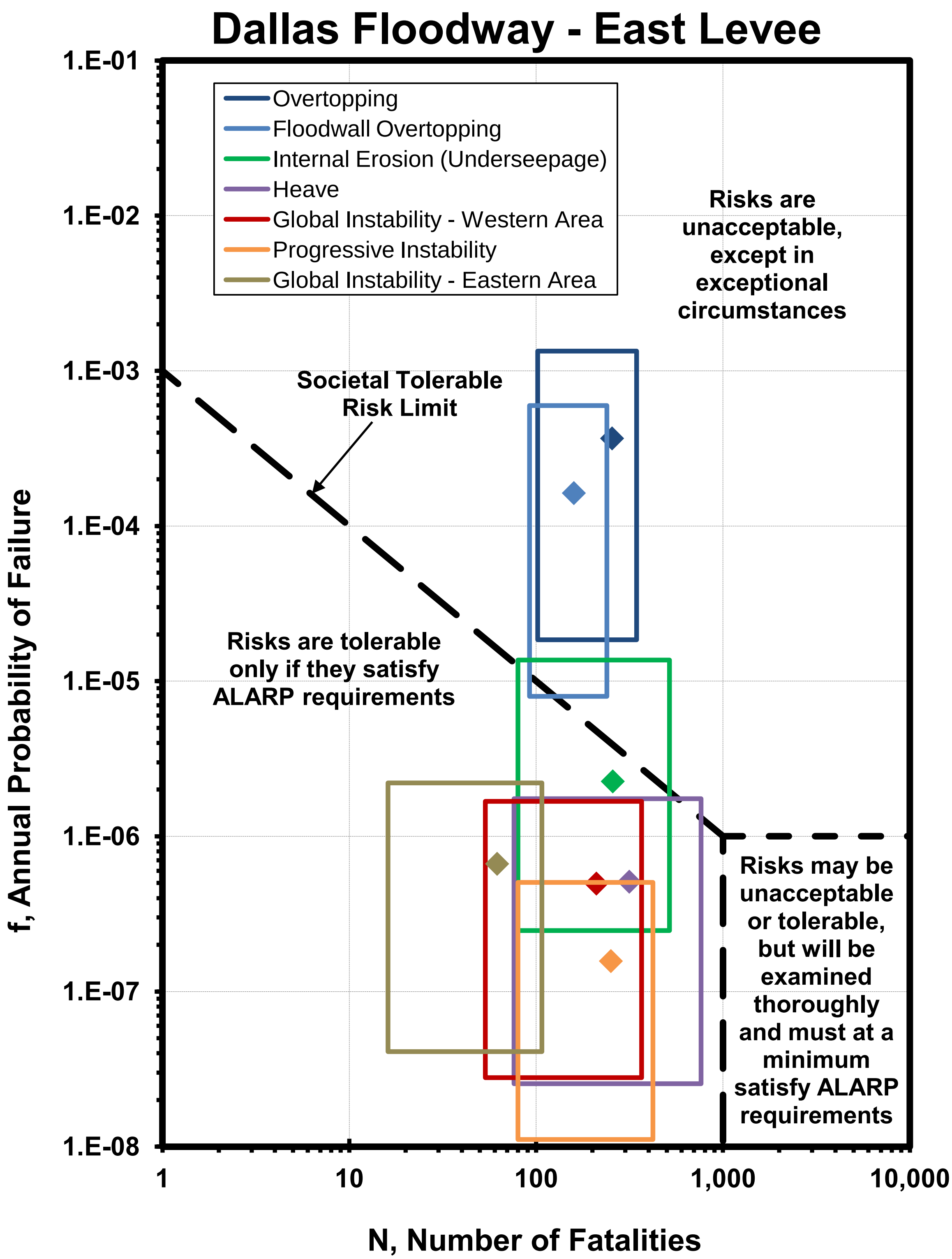

Figure 3. f-N plot of the East Levee from the BCRA report.

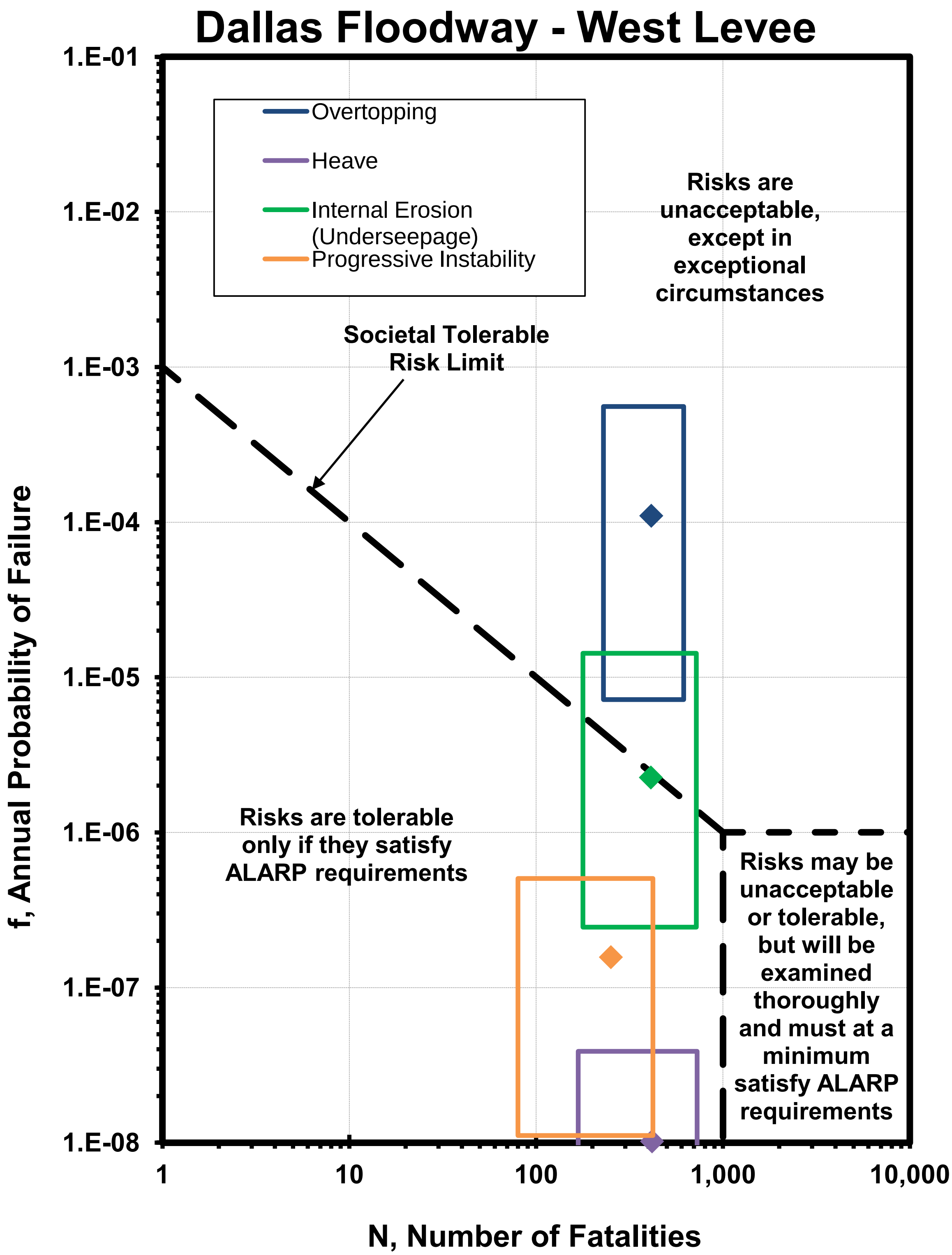

Figure 4. f-N plot of the West Levee from the BCRA report.

BVP Lakes

Based on the current plans and details of the BVP, the depths below the current floodway surface of West Dallas Lake, Natural Lake, and Urban Lake are on the order of 10 to 20 ft. Based on available subsurface data, excavations for these lakes will not advance deep enough to penetrate the surficial clay layers that provide an aquatard between the basal sand lenses that typically overlie bedrock in the area of the Dallas Floodway and any free-surface floodwaters that move into the area. Except in the area of Oxbow Lake, it is anticipated that clay thicknesses will be maintained to a minimum of 10 ft. Since there will still be a clay barrier in place, the RET anticipates that flood waters will not have the opportunity to significantly increase the piezometric pressure in the basal sand layers below the levees and destabilize the embankments. Therefore, the RET does not anticipate that the excavations for West Dallas, Natural, and Urban Lakes will affect the project risk.

As it is currently designed, excavations for Oxbow Lake will penetrate through the clay cover and underlying basal sand layers and advance into the shale bedrock. This would provide a window through the clay aquatard for floodwaters to penetrate into the basal sands and potentially increase the piezometric pressure to a critical point under the levees and under the land-side toe of the levees. However, the City of Dallas has placed a soil-bentonite cutoff wall from Station 3+00 to 29+00 along the river-side levee toe of the west levee alignment. In addition, the Trinity Lakes Geotechnical Report indicates all proposed lakes will have an 18-inch compacted clay liner placed in the bottom that will be increased to 30 inches thick where undesirable soils are encountered. Due to the existence of the cutoff wall along the west alignment, the clay liner described in the previous sentence, a relatively thick landside clay blanket, and high land-side ground surface on the east alignment, excavation for Oxbow Lake is not expected to impact the stability of the levees in this area. Therefore, it is the opinion of the RET that placement of the proposed lakes detailed in the BVP will not impact the ability of the Dallas Floodway Project to reduce the risk of flooding the surrounding parts of the city.

Relocating the River Alignment

As part of the BVP, the river channel will be moved from its current linear alignment to incorporate more naturally appearing meanders. As a consequence, the river channel will be moved closer to levee alignment in some areas and farther away in others. The newly relocated river channel will be excavated deeper than the proposed lakes and will penetrate through the surficial clay blankets. Therefore, if the river channel is relocated closer to the levee, the seepage path will be decreased and there will be less head dissipation from where floodwaters enter the relatively permeable basal sand layers through the channel bottom and the land-side toe of the levee embankments. Therefore, wherever the river was moved closer to a levee, the RET evaluated the impact on risk.

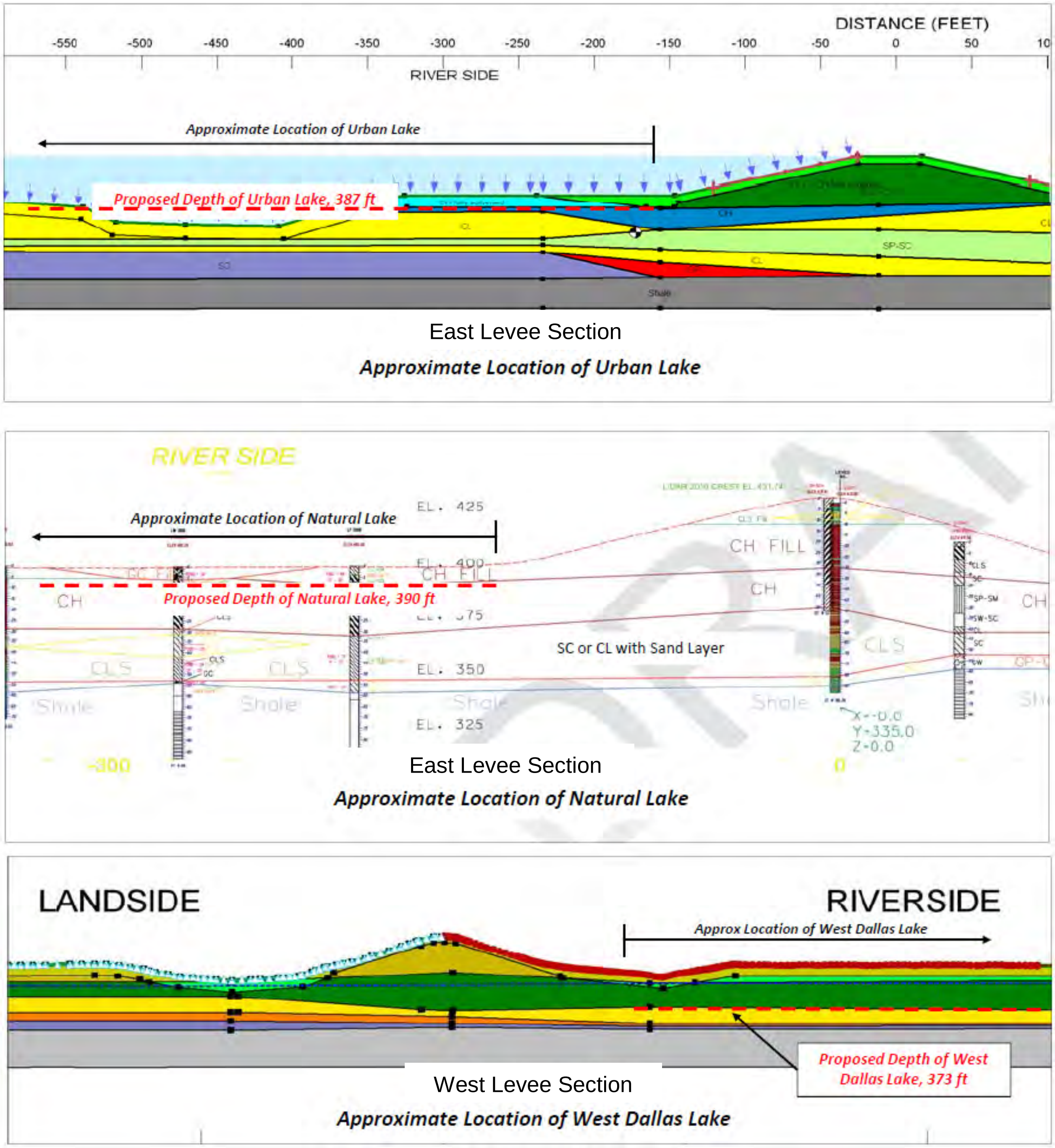

Figure 5. Shows how the currently designed excavation depths for the proposed BVP lakes compare to the elevations of the subsurface clay strata. Yellow and olive colored soil layers represent CL materials, green and blue soil layers represent CH materials.

Trinity Parkway Roadway Embankment

Approximately half of the parkway within the floodway includes an embankment placed in front of the riverside slope of the levee. The embankment is intended to keep the road higher than the

100-year flood. This portion of the roadway will widen the existing embankment, effectively fortifying that section of levee with additional fill material.

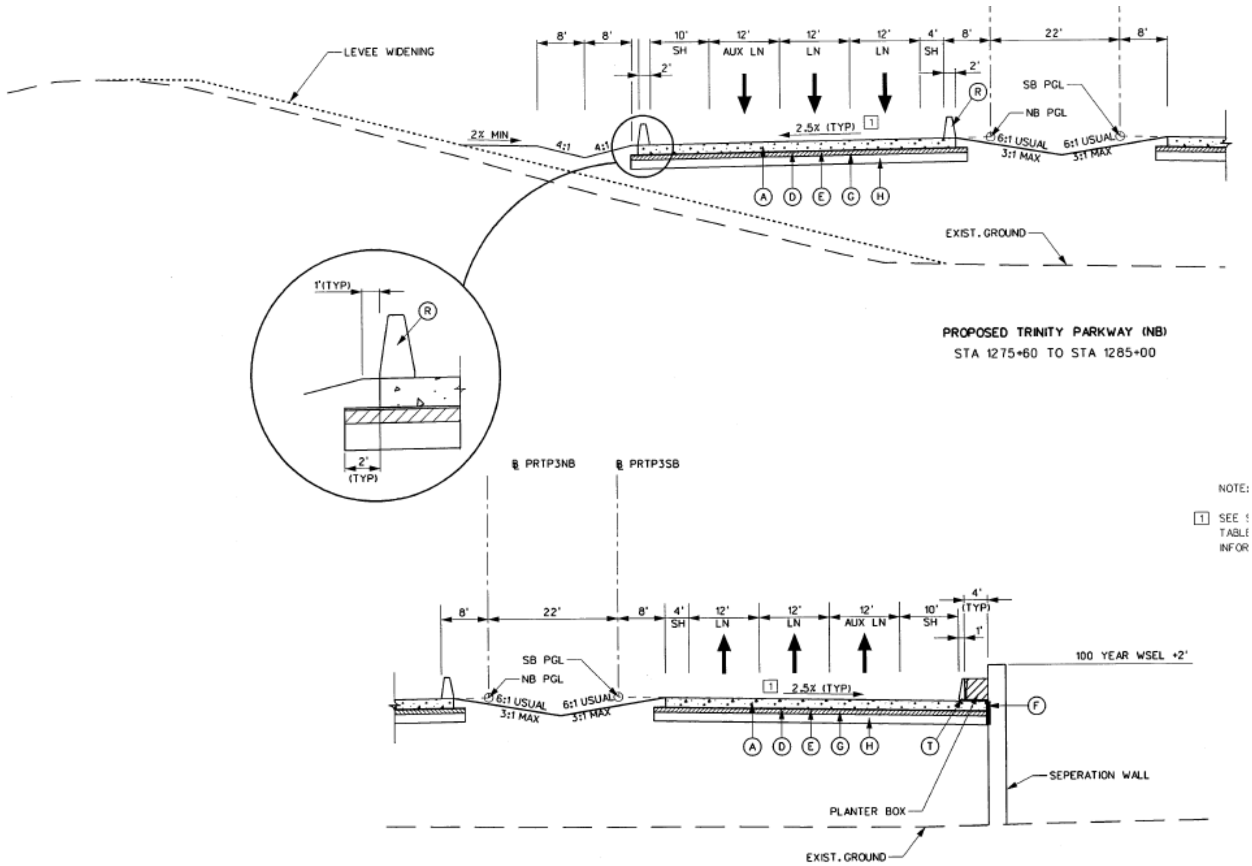

Figure 6. Typical cross-section of a Trinity Parkway roadway embankment.

The proposed roadway embankment will be approximately half the levee height. At the roadway elevation, the combined width of the levee and additional roadway embankment will be significantly larger in cross-section (3 to 8 times as wide as the original levee width). Moderate flood events with a maximum river level of 50% of the levee height will require collapse of the levee over the entire crest of the roadway embankment for full breach to occur. As a result, levee section with the roadway embankment in place will be significantly more robust for flood heights up to 50% of the levee height. Although collapse of the roadway is not required for inundation during for larger storm events, there may be a reduction in consequences. The RET determined that the presence of the roadway would reduce risk for floods that are 75% but not 100% of the levee height.

The placement of the Trinity Parkway roadway embankment will likely induce settlement on the order of several inches to over a foot in some areas. This could cause cracking of the existing embankment on the riverside of the levee embankment, but the cracking would likely be longitudinal and would not present an open pathway to the advancement of an upstream to downstream (riverside to landside) failure. This settlement is expected to occur in the clay layers overlying the basal sand layers. Embankment placement is not expected to affect the ability of the sand layers to conduct seepage.

Risk Assessment – Failure Modes

In order to determine how the overall risk of the Dallas Floodway would change with the implementation of the BVP and the Trinity Parkway, the RET examined each failure mode that was identified as being potentially affected by implementation of components of the Comprehensive Plan. As detailed in the previous section, those elements included the relocation of the current river alignment to introduce meanders within the floodway and the placement of the Trinity Parkway roadway embankment.

PFM 2, Overtopping of the East Levee Embankment

As was described earlier, the Trinity Parkway roadway embankment will be placed from approximately station 300+00 on the east levee alignment and continue downstream to the current study limits of the Dallas Floodway. The roadway embankment will be built to approximately half of the levee height and be up to 8 times as wide as a typical levee embankment at that height. Placement of this additional embankment will not affect the likelihood of the levee to overtop during a large hydrologic event since the crest elevation of the levee will not be changed. The wider embankment section, however, may prevent the levee from eroding completely down to its base. This would decrease the likelihood that the levee would fully breach during an overtopping event. To address this potential outcome, the RET added a node to the event tree for overtopping titled “Levee Fully Breaches”. All other nodes in the event tree remain the same as what was estimated for the BCRA.

The following events would need to occur in order for the levee to breach as a result of this failure mode. The events shown in red italicized text were re-estimated by the RET in order to address changes due to the implementation of the BVP and the placement of the Trinity Parkway. The events in black text use the original nodal estimates from the BCRA.

1. A flood event occurs, increasing the river elevation above that of the levee crest
2. Intervention fails
3. Erosion initiates and leads to a breach of the levee
4. *Levee fully breaches to the base of the embankment*

The BCRA report divided overtopping up into major and minor overtopping events: minor overtopping was defined on the east alignment as flow over the levee crest that is 2.2 ft deep or less. Deeper overtopping depths were considered major overtopping. Minor overtopping duration was considered to be 24 hours on the east alignment. Major overtopping duration was considered to be 40 hours.

Node 4, Levee Fully Breaches

For minor overtopping, the RET estimated that water would overtop the crest level at a depth of 2.2 ft for 24 hours but could be flowing over the Trinity Parkway roadway embankment for days at significant depths (approximately 7 ft). Some sources of uncertainty in the estimate are listed below.

- It is anticipated that tailwater will rise but the time is not well understood
- There may be additional opportunity for intervention to close the partial breach before it fully breaches
- Breach width could vary
- Performance of roadway as a hindrance to erosion is not well understood
- Nature of the roadway fill material is questionable but will likely come from the floodway floor (CH)
- The soils beneath these embankments will densify due to the weight of the additional fill, increasing soil resistance to erosion

For major overtopping, water is expected to overtop the crest level at a depth greater than 2.2 ft for 40 hours but could be flowing over the Trinity roadway for days at significant depths (7 to 10 ft or so). The team considered the uncertainty to be similar to minor overtopping.

Case	Min	Mode	Median	Mean	Standard Deviation	Max
East Alignment Minor OT	0.5	0.5	0.55	0.57	0.10	0.8
East Alignment Major OT	0.75	0.9	0.8	0.83	0.06	0.9

These estimates for major and minor overtopping assume the flow in the floodway for the threshold event (the point at which overtopping is expected to initiate) is 232,000 cfs, the same as it was for the BCRA. The city is planning to raise the levee to contain a flood event with a flow of 277,000 cfs as a result of the alternatives analysis. Should the height of the leveebe increased to contain a 277,000 cfs event, only the estimates for minor overtopping would be pertinent.

Based on the most recent crest elevation survey, the BCRA concludes the critical area for overtopping on the east levee alignment extends from Continental Blvd to Hampton Pump Station. This is the location for which the risk due to overtopping was estimated during the BCRA. Therefore, the impact on overtopping from the proposed Trinity Parkway was evaluated along this levee reach for the purpose of this study.

Consequences

Should a failure occur in the enlarged embankment section from implementation of the Trinity Parkway, the time required to completely erode the embankment will be much longer because the section is much wider than the original section. Consequently, it will take much longer to reach the same inundation depths in the leveed area than if a breach occurred in the unaltered embankment. With a compacted clay embankment, this difference in time could be on the order of 10 days. As a result, inundation depths for the leveed areas of the city would be approximately 3 ft lower and there will be more opportunity to warn the population at risk so life loss could be significantly less than the without-project case (see Figure 7).

A consequence model run using HEC-FIA was made for the case of major overtopping for the east levee alignment (shown in bold in the table below). The consequence estimate assumes an approximately 12 hour warning that would be issued due to the presence of high water on the levees and the expectation that the levees were most likely going to overtop. Because a component of the total risk for overtopping includes minor overtopping, a consequence estimate had to be made for failure due to minor overtopping (shown in italics in the table below). To do this, the same relationship between the major overtopping BCRA estimate (Major OT_{BCRA}) and the estimate accounting for the presence of the Trinity Parkway (Major OT_{TP}) was used to estimate the consequences for minor overtopping. This relationship used to estimate the consequences for minor overtopping that accounts for the Trinity Parkway (Minor OT_{TP}) is defined below.

$$Minor\ OT_{TP} = \left(\frac{Major\ OT_{TP}}{Major\ OT_{BCRA}}\right) * Minor\ OT_{BCRA}$$

The original BCRA consequence estimates and the consequence estimates for overtopping with the Trinity Parkway in place are given in the following table. Should levee raises be implemented in the future that are designed to contain an event of 277,000 cfs within the confines of the floodway, the estimates for minor overtopping would be the only pertinent consequence estimates.

Overtopping Event	Trinity Parkway?	Best Case Expected LoL	Most Likely Expected LoL	Worst Case Expected LoL
Minor	No, BCRA	28	103	784
Minor	*Yes, TP*	*20*	*83*	*434*
Major	No, BCRA	40	134	1274
Major	**Yes, TP**	**29**	**109**	**705**

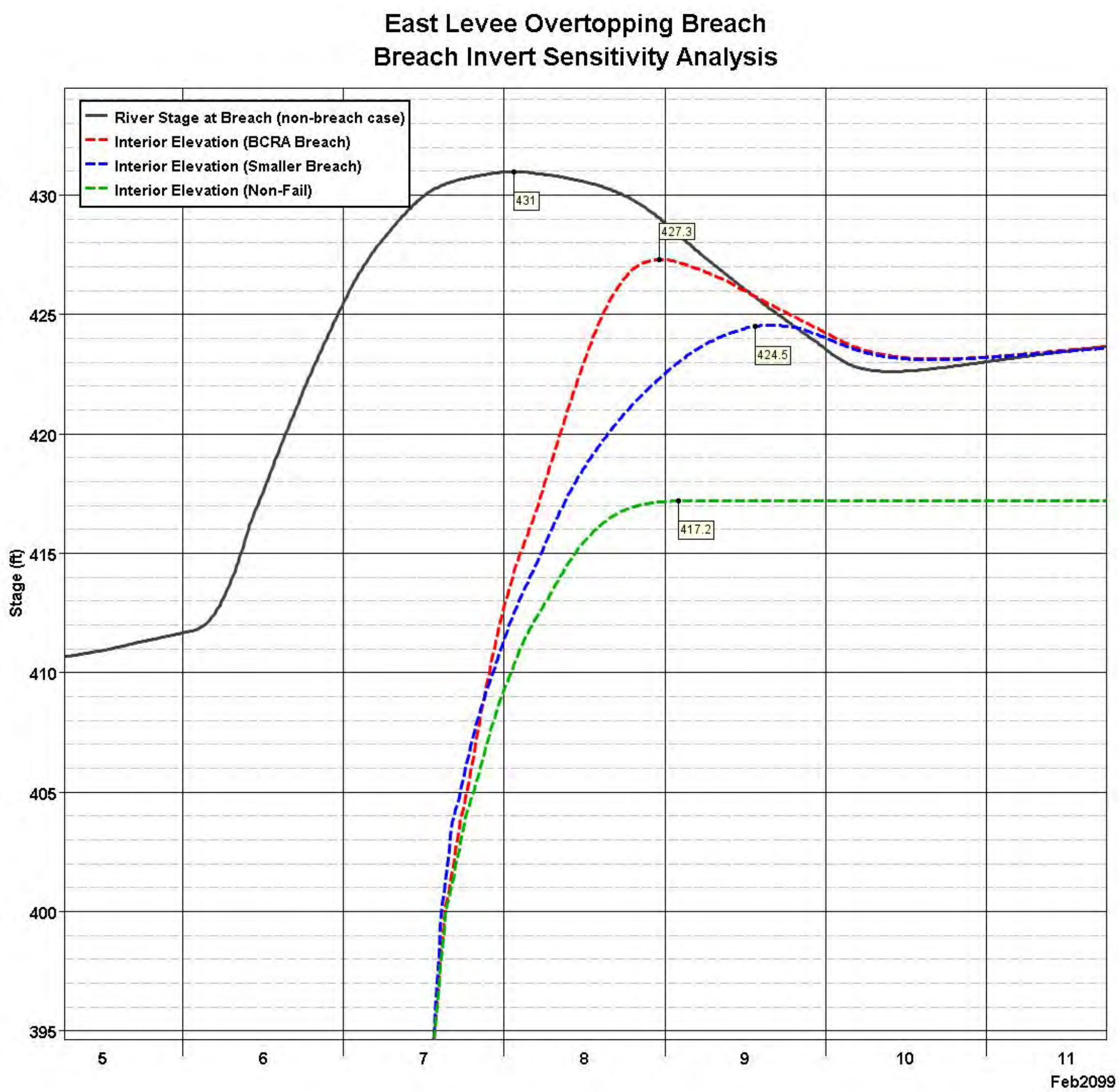

Figure 7. View of internal inundation depths. Green line is OT without failure, red is OT with failure down to toe elevation, and blue is OT with failure down to the elevation of the Trinity Parkway roadway.

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.

This change in risk will only affect the area on the east levee alignment where the Trinity Parkway roadway embankment is placed. In all other areas, including the entire west levee alignment, the risk from overtopping will not be changed from the original estimates made during the BCRA.

PFM 2, Condition	Annualized Failure Probability	Annualized Life Loss
With Trinity Parkway in place	1.66E-04	2.78E-02

PFM 7, Internal Erosion through the Foundation

The BCRA indicates that the levee system is susceptible to this failure mode on both the west alignment and east alignment.

East Levee Alignment

Of all the areas examined, the critical area for heave on the east levee was one of the few where all critical factors considered were present. The basal sand layer exited on a free face on the land side, as interpreted by available exploration data. This reach also had the shortest seepage path for a shallow subsurface sand layer, with a deep landside sump (ditch). Considering these factors, risk estimates for other areas would be significantly lower.

The City of Dallas recently placed a soil-bentonite seepage cutoff wall along the riverside toe of the east levee. The RET anticipates that placement of the cutoff wall in this area has interrupted the continuity of the sand layer beneath the levee in this area, which is captured in Node 1 in the event tree for this failure mode. In addition, the cutoff wall would reduce the gradient during a storm event making it less likely for erosion of the sand exposed on the ground surface at the landside toe of the levee to initiate. This initiation is captured in Node 3 in the event tree. Had this cutoff wall not been placed, the risk of internal erosion would have likely increased with implementation of the BVP. However, the RET determined that the total risk with the BVP in place will be below the BCRA estimate due to the recent addition of the cutoff wall.

West Levee Alignment

The cross-section, materials, and location of the critical levee section on the west levee alignment were compared to that of the critical section on the east alignment during the BCRA. It was found that there was no significant difference between the levee sections so the same probability estimates were used.

The BCRA estimated that the risk of this failure mode occurring on the west levee was above USACE’s tolerable risk guidelines for dams. However, implementation of the BVP would move the river further from the west levee in this location so it’s possible that risk may be slightly lower due to the fact that the seepage path would be lengthened. Any improvement in risk is expected to be minimal, however. The overall risk in this reach will remain unchanged from the BCRA estimate with respect to PFM 7.

Consequences

There will be no change in consequences for PFM 7, internal erosion failure, due to the implementation of the BVP or the placement of the Trinity Parkway. There may be a decrease in warning time, however, if the levee is raised to address overtopping. This is detailed in the report of the Dallas Floodway Alternatives Analysis dated 2 November, 2012.

PFM 8, Heave at the Landside Toe of the Levee, East Levee Section

As described in the BCRA report, there are locations on the East and West levee systems where a pervious basal sand layer exists above the foundation rock and is overlain by an impervious clay cap. In those locations, 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. The results of the BCRA indicate there is an area on the east alignment that is above USACE tolerable risk guidelines for dams. The other area thought to be critical for this failure mode on the west levee alignment was estimated to be below guidelines.

The following events would need to occur in order for the levee to breach as a result of this failure mode. The events shown in red italicized text were re-estimated by the RET in order to evaluate the change in flood risk due to the implementation of the BVP and the placement of the Trinity Parkway. The events in black text use the original nodal estimates from the BCRA.

1. *There is a continuous sand layer connected to the river that allows water pressures to build up in the foundation*
2. A flood event occurs
3. *The foundation pressures exceed the weight of the impervious cap causing the foundation to heave on the land side*
4. Early intervention is unsuccessful if the heave is observed
5. *The gradient is sufficient to move the basal sands*
6. A roof forms and allows erosion to progress under the levee section
7. Heroic intervention fails
8. Erosion progresses and leads to a breach of the levee
9. *Levee fully breaches to the base of the embankment*

The RET assessed the impact of implementing the two critical components of the Comprehensive Plan on the risk from this failure mode separately.

With River Meanders

With the implementation of the BVP, the river would be relocated within the floodway to create meanders in the alignment. At the critical locations, the river would be moved closer to the levee. This would shorten the seepage path from the point where flood waters could enter the sand layer to where they would exit during a failure, at the landside toe of the levee. This would be a potential increase in the risk of failure. As a result, the RET re-assessed the estimates for Nodes 1 (Sand transmits seepage), 3 (Heave occurs at the toe), and 5 (Gradient sufficient to erode sand).

With Trinity Parkway

As was discussed in the treatment of PFM 2, Overtopping, the placement of the Trinity Parkway roadway embankment could prevent or significantly slow development of a full breach of the levee. To address this change in the likelihood of failure, the RET added Node 9 to the end of the original event ("Levee Fully Breaches") to address the effect of the addition of the roadway embankment will have on overtopping.

Node 1, Sand Transmits Seepage

During the BCRA, this node was deposed into three separate nodes in order to make that risk estimating team's estimate more rational and simpler to understand. Those nodes were "continuity of sand layer," "sand layer is clean," and "sand layer is connected to river." The RET for this assessment, however, was able to provide this estimate as one node.

The maximum estimate for this node from the BCRA was 0.73. The maximum nodal estimate from the RET for this study happened to be lower despite the basal sand layers being more likely to transmit seepage due to a shorter seepage path. For consistency, the RET considered using the BCRA estimate as the maximum instead of the lower RET estimate. The RET decided not to substitute the BCRA value, but realized the range in estimates may be inherently smaller for this study than the BCRA report due to the smaller group size with fewer estimates.

Risk estimates for this node in the BCRA were informed by seepage and stability analyses performed on a levee section at Station 220+00. Seepage and stability analyses of the section at Station 247+00 were performed by HNTB in support of this project under contract with the City of Dallas. The RET considered the embankment and foundation conditions to have enough in common to make the results of models at the two locations reasonably comparable.

By relocating the river from the centerline of the floodway, the seepage path would be decreased from 1,600 ft to 900 ft. Analyses at Station 247+00 were performed with the river in the existing location and with the proposed location closer to the embankment. These analyses were provided to the RMC prior to the risk assessment and the RMC performed additional sensitivity analyses with varying permeability values. The HNTB analyses and the RMC sensitivity analyses indicate that moving the river closer to the levee section generally decreases the overall performance of the embankment. However, the team feels that the uncertainty in several factors including:

- Continuity of the sand layers
- Permeability of the sand and clay layers
- The consistency and thickness of the impermeable layer at the landside embankment toe and in the area of the landside sump

- Historical pore pressure regimes during previous storm events
- 3-dimensional movement of seepage in permeable foundation layers

likely outweigh the differences in results seen in the seepage and stability models that investigate the change in river alignment. The highly variable foundation conditions of the point-bar depositional environment and the repeated modifications to the area as a result of 100 (+) years of ongoing development within the floodway lead the RET to believe it may not be appropriate to rely solely on the seepage and stability models to accurately portray the effects of moving the river closer to the levee. Therefore, the team evaluated the flood risk in this area using these analyses as a tool to inform their engineering judgment and expert elicitation.

The largest historical event that has taken place in the Dallas Floodway resulted in Trinity River levels that were about 2 ft below the 100-yr event. Performance during this event was good and suggests a higher silt content and a correspondingly lower permeability in the foundation sand layers. There was no observation of poor performance on the landside toe during this event in the critical area for this failure mode. Poor performance would most likely occur in the bottom of the landside sump, in which nothing was observed or reported. It should be noted, however, that poor performance in this area would be difficult to observe.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
Independent	0.25	0.3	0.3	0.29	0.05	0.4

Node 3, Foundation Pressures Cause Heave at the Toe

As mentioned in the BCRA report, one of the reasons this particular area of the east levee alignment 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 as well as seepage path length.

Seepage modeling done on critical sections indicates there is a head loss across the basal sand layer in the range of 36 to 20 ft. This range corresponds to full dissipation of the river head (36 ft, where the piezometric pressure at the landside toe of the levee would be low) to only about half of the reservoir head (20 ft, where there would still be an appreciable amount of piezometric head at the landside toe).

The RET anticipates that the gradient will be essentially doubled by moving the river closer to the levee (from 1,600 to 900 ft). Under worst case scenario permeability estimates, the gradient will go from 3 to 4 (if the critical gradient is considered to be on the order of 1, then the factor of safety against heave is on the order of 1/3 to ¼). Therefore, the RET anticipates the addition of the BVP will increase the nodal estimate of performance because the pressure at the toe will increase.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
100% Height	0.9	0.9	0.9	0.92	0.03	0.95
75% Height	0.5	0.65	0.68	0.68	0.09	0.8
50% Height	0.1	0.4	0.38	0.33	0.13	0.45

Node 5, Gradient Sufficient to Erode Sand

At this point in the event tree, it is assumed that the impermeable layer at the landside toe of the levee has heaved and seepage can exit. There is no build of piezometric pressure near the landside ground surface up associated with a clay aquatard. For the failure mode to progress, enough gradient must exist to begin to move sand particles out of the basal sand layer vertically upward from the horizontal face of the landside ground surface. 0.6 was considered to be the critical gradient to move sand horizontally during the BCRA. This is not considered enough to cause initiation in this case (which would be erosion vertically upward from a horizontal face), but it is enough for progression of the failure mode as the erosion pipe moves upstream. Based largely on the fines content of the basal sand layers which decrease permeability, it is anticipated that a critical gradient would need to be higher, approximately 1.0, to initiate vertical particle movement.

The gradient will approximately double as a result of the river moving closer to the levee (from 1,600 to 900 ft). After some deliberation, the RET estimated this value would double from the BCRA estimate for this node.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
100% Height	0.1	0.2	0.2	0.20	0.05	0.25
75% Height	0.05	0.1	0.10	0.11	0.04	0.15
50% Height	0.01	0.05	0.05	0.04	0.02	0.05

Node 9, Levee Fully Breaches

As described at the beginning of this section, this node was added to the original event tree to address the thickened levee section provided by the placement of the Trinity Parkway roadway embankment.

The proposed roadway embankment is built to approximately half of the levee height. At this elevation, the width of the levee due to the addition of the roadway will be significantly longer in cross-section (3 to 8 times as wide). In the event of heave that progresses to a backward erosion piping failure during a storm event with a maximum river level that’s 50% of the levee height, collapse of the levee over the entire crest of the roadway embankment would be required for full breach to occur. As a result, levee section with the roadway embankment in place will be significantly more robust for flood heights up to 50% of the levee height. This will not be the case for larger storm events, though there may be a reduction in the likelihood that a full levee breach would occur.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
100% Height	0.2	0.5	0.4	0.40	0.11	0.5
75% Height	0.05	0.15	0.15	0.16	0.07	0.25
50% Height	0.001	0.01	0.01	0.02	0.03	0.1

Consequences

The RET anticipates that the consequences for a heave failure on the east alignment after the river meanders have been put in place will be no different from those estimated in during the BCRA. If the failure occurs after the Trinity Parkway has been placed, it is estimated that there will be a small decrease in the consequences form what was originally estimated. The RET anticipates that the thickened levee section will slow down or even arrest the failure, resulting in a longer warning time and a reduction in consequences.

An estimate of consequences was made for PFM 8 that accounts for the presence of the Trinity Parkway roadway embankment on the east levee alignment using HEC-FIA. The model run assumed that the river level was equal to the crest elevation of the levee (100% height). The consequence estimate assumes an approximately 12 hour warning that would be issued due to the presence of high water on the levees or the observation of seepage emerging on the landside of the levee. Because components of the total risk for heave include river heights at 75% and 50% of the levee crest elevation, an estimate of consequences at the other river levels had to be made (shown in italics in the table below). To do this, the same relationship between the BCRA estimate with the river level at 100% of the levee crest elevation ($100\%_{BCRA}$) and the estimate accounting for the presence of the Trinity Parkway at the same river level ($100\%_{TP}$) was used to estimate the consequences that account for the Trinity Parkway at river levels 75% and 50% of the levee crest. This relationship used to estimate the consequences that accounts for the Trinity Parkway (Minor OT_{TP}) is defined below.

$$75\%_{TP} = \left(\frac{100\%_{TP}}{100\%_{BCRA}}\right) * 75\%_{BCRA}$$

The original BCRA consequence estimates and the consequence estimates for heave with the Trinity Parkway in place are given in the following table.

River Stage	*BCRA Consequences*	*Scaled Consequences for Trinity*

					Parkway	
	Best Case Expected LoL	**Most Likely Expected LoL**	**Worst Case Expected LoL**	**Best Case Expected LoL**	**Most Likely Expected LoL**	**Worst Case Expected LoL**
100% Height	12	66	748	4	8	368
75% Height	45	124	2521	*15*	*15*	*1240*
50% Height	1	5	18	*0*	*1*	*9*

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.

PFM 8, Condition	Annualized Failure Probability	Annualized Life Loss
With BVP (river meanders) only	3.35E-06	1.13E-03
With BVP & Trinity Parkway	8.63E-07	9.58E-05

PFM 8, Heave at the Landside Toe of the Levee with Cutoff Wall in Place, East Levee Section

As described earlier, the BCRA report identifies the critical area on the east levee alignment that is the most likely to fail from heave. The RET anticipates that moving the river closer to the east levee alignment as part of the BVP increase the likelihood of breach. As part of this risk analysis, the RET considered extending the recently placed cutoff wall to mitigate this increase in risk.

The area where the river channel will be relocated closer to the east levee alignment is from the approximate location of Continental Road Bridge (about Station 170+00) to Station 285+00. The cutoff wall recently placed by the City of Dallas extends from Station 285+00 to 442+00 on the east levee alignment. If this cutoff wall were extended downstream to the approximate location of Continental Bridge (a linear extension of about 6,500 ft), the wall would be extended past the area where the river channel will be moved closer to the levee as a consequence of implementing the BVP.

This location is also where the Trinity Parkway is currently proposed. A portion of the Trinity Parkway will utilize a pile-supported floodwall that will likely include some sort of cutoff. It is anticipated, however, that the Trinity Parkway may potentially incorporate a floodwall at Continental Road Bridge that would continue downstream in the area where existing roadway bridges pass over the Dallas Floodway. This is not the area where the risk of a heave failure is increased by the relocation of the river channel. Therefore, the cutoff wall that may be associated with the Trinity Parkway will not offset the effect of river realignment in this reach.

An impermeable blanket placed on the floor of the floodway was proposed as a potential alternative remediation measure to a cutoff wall. A cutoff wall placed 10 to 30 ft in front of the riverside toe was considered a better alternative, however, because:

- Placing an impermeable liner in the new river meander may be more difficult and not as reliable as placing a soil-bentonite cutoff wall at the riverside toe of the levee
- A channel blanket would require more fill material and likely scour protection (rip-rap in the channel bottom)
- Use of a cutoff wall will provide a positive engineered cutoff of the permeable basal sand layer; this will allow unhindered excavation of the floodway floor without significantly impacting the stability of the levee embankments
- A cutoff wall would extend from 20 to 40 ft in depth from the ground surface and not be difficult to construct
- A compacted clay river bottom may not be environmentally desirable

In order estimate the risk reduction provided by the placement of a cutoff wall, the RET re-estimated Node 1 (Sand transmits seepage) and Node 3 (Heave occurs at the toe) of the event tree for PFM 8, which are shown below in red italicized text. Since the relocation of the river channel affects more than just Nodes 1 and 3 (see the section detailing PFM 8 without the benefit of the cutoff wall), there are two nodes that remain changed from the BCRA that were not affected by the placement of the cutoff wall. They are Node 5 (Gradient sufficient to erode sand) and 9 (Levee fully breaches). Given the conditional nature of nodal estimates in the event tree, Node 5 includes the condition that enough gradient is present to heave the toe of the levee (Node 3). Therefore, the estimates for Node 5 from PFM 8 with the new meander in place (closer than the existing river alignment) were used. Node 9 addresses the placement of the Trinity Parkway roadway embankment. The remaining events in the event tree use the original nodal estimates from the BCRA.

1. *There is a continuous sand layer connected to the river that allows water pressures to build up in the foundation*
2. A flood event occurs
3. *The foundation pressures exceed the weight of the impervious cap causing the foundation to heave on the land side*
4. Early intervention is unsuccessful if the heave is observed
5. *The gradient is sufficient to move the basal sands*
6. A roof forms and allows erosion to progress under the levee section
7. Heroic intervention fails
8. Erosion progresses and leads to a breach of the levee
9. Levee fully breaches to the base of the embankment

Node 1, Sand Transmits Seepage

This nodal estimate was reduced by approximately one order of magnitude based on the presence of the cutoff wall. HNTB, responsible for construction quality control on the cutoff wall that was recently constructed by the City of Dallas, has indicated there was an employee on site at all times during the construction process that is solely dedicated to quality control. In the team's opinion, this lends some confidence to the integrity of the cutoff wall.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
Independent	0.001	0.001	0.005	0.02	0.03	0.1

Node 3, Foundation Pressures Cause Heave at the Toe

Each node in an event tree is conditional upon previous nodes being satisfied. Therefore, the probability estimates for Node 3 assume the sand layer is continuous. Since the cutoff wall is in place, it was assumed that there would have to be a defect in the cutoff wall that is transmitting seepage. The RET team decided not discuss what this defect would specifically look like. It was decided that it would be up to the individual risk estimators to consider what this defect looks like and how it will affect the potential to pass seepage to the landside toe of the levee. The team members' judgment of the size of the defect, shape, severity, origination, and ability to pass seepage has all been rolled into their nodal estimates.

River Stage	Min	Mode	Median	Mean	Standard Deviation	Max
100% Height	0.01	0.01	0.01	0.017	0.014	0.05
75% Height	0.001	0.005	0.005	0.004	0.002	0.005
50% Height	0.001	0.001	0.001	0.001	0.000	0.001

Consequences

Placement of a soil-bentonite cutoff wall does not change the consequences from the estimates given in the previous section that were made for PFM 8 when the BVP and Trinity Parkway have been implemented.

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.

PFM 8, Condition	Annualized Failure Probability	Annualized Life Loss
With BVP (river meanders) & Soil-Bentonite Cutoff Wall	4.48E-09	1.07E-06
With BVP, Cutoff Wall & Trinity Parkway	1.46E-09	1.19E-07

Uncertainty

Uncertainty was modeled the in the same way as the BCRA report, using distributions contained in @Risk. For the hydrologic loading, the 5th, 50th, and 95th curves were input using a lognormal distribution. After a comparison of several distribution types, the log-normal distribution was selected for hydrologic loading because it best matched the median, mean, and range of uncertainty in the data.

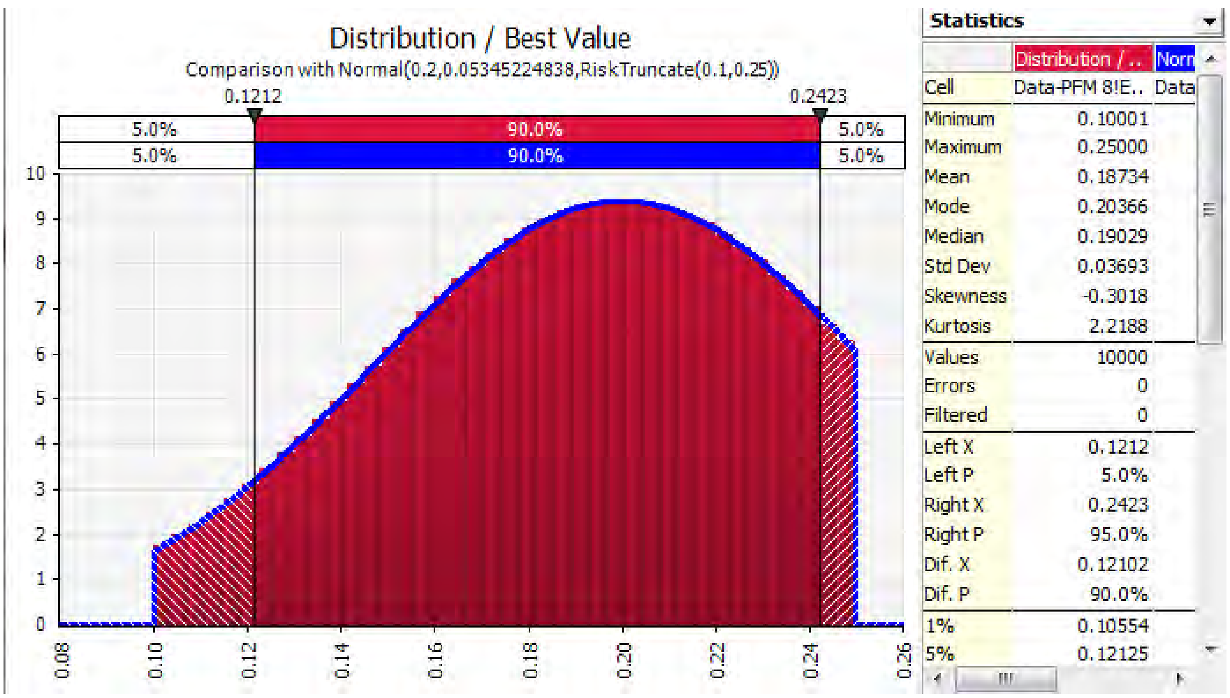

Figure 8 - Example truncated normal distribution for the Sufficient Gradient to Erode Sand node.

For each event tree branch, the team estimates were used to quantify the uncertainty. For nearly all distributions a normal distribution was selected and then truncated at the team's minimum and maximum estimate. Figure 8 shows an example of what that distribution looks like for a single event tree branch. These distributions were used and included in a Monte Carlo simulation with 10,000 runs per failure mode.

Major Findings and Understandings

The results of the risk assessment can be seen on the f-N plots shown at this end of this section. In addition, major findings and understandings about the individual components of the Comprehensive Plan can be found below.

The f-N plot of PFM 2, overtopping, indicates that the risk of failure from overtopping on the east levee alignment is decreased from the BCRA condition but is still above tolerable risk guidelines for dams with the Trinity Parkway in place. This is due to the fact that the thickened levee section of the Trinity Parkway will take longer to erode all the way to the base of the embankment and the leveed area will therefore take longer to become inundated. This has the effect of decreasing the expected consequences.

The f-N plot of PFM 8, heave on the east levee indicates the risk of failure increases above the tolerable risk guidelines with the placement of the river meanders in the Trinity River channel. However, placement of the roadway embankment decreases the risk to at or below guidelines, while the placement of a cutoff wall puts the estimated risk well below guidelines. Placement of both the cutoff wall and Trinity Parkway roadway embankment will, of course, lower the risk of failure even further.

Trinity River Meanders

The BVP proposes to change the alignment of the Trinity River to include more naturally looking river meanders as opposed to the existing straight-line river alignment that is currently located in the center of the floodway for the majority of the study area. As a result, the river will be closer to the levees in some locations. Currently, the main channel of the river penetrates the clay blanket layer that acts as an aquatard over the floor of the floodway and provides a path for floodwaters to permeate into the basal sands and potentially increase the piezometric pressure to a critical point under the landside toe of the levees. It is anticipated that the existing river bottom elevations will be maintained as the meanders are placed. As a result, wherever the meanders move closer to the levees, floodwaters moving through the basal sand layers have a shorter path to reach the levees. The shorter path results in a higher gradient at the land-side toe which increases the likelihood that the levees could become destabilized (either through heave of a clay blanket at the land-side toe and subsequent initiation of backwards erosion piping of sandy materials or just initiation of backward erosion piping where there is no landside clay blanket).

Since the proposed river meanders are sinuous and move closer to the levees in multiple locations, the impacts on the ability of the Dallas Floodway to reduce the risk of flooding will be described in sections with similar levee profiles.

West Levee, Station 3+00 to 29+00

Risks posed by the currently existing channel alignment were estimated to be high during the BCRA. As a result, the City of Dallas placed a soil-bentonite cutoff wall at the river-side toe of the levee from Station 3+00 to 29+00. Had this cutoff wall not been in place, the risk would have increased with implementation of the BVP due to the addition of Oxbow Lake (the river will largely stay in its existing location in this area). However, the risk with the BVP in place will be below the BCRA estimate due to the recent addition of the cutoff wall.

West Levee, Station 29+00 to 165+00

The river is proposed to move closer to the levee in this area. The impact on the overall risk at this section of levee is expected to be minimal, however, due to the presence of a thick land-side clay blanket that will prevent high piezometric pressures from developing near the ground surface. There is also a relatively high land-side ground surface and no sump in this location.

West Levee, Station 165+00 to 250+00

Risks were estimated to be near guidelines in this area during the BCRA. However, there should not be a change in the risk with implementation of the BVP because there are no lakes proposed in this area and the river is not proposed to move closer to the levee.

West Levee, Station 250+00 to the Upstream End

There is not expected to be a change in the risk in this area with implementation of the BVP because the river is not proposed to move closer to the levee and the West Dallas Lake is not expected to impact the performance of the protective impermeable clay blanket.

East Levee, Downstream end to Continental Avenue (Station 170+00)

The impact on the overall risk at this section of levee is expected to be minimal due to the presence of a thick land-side clay blanket that will prevent high piezometric pressures from developing near the ground surface.

East Levee, Continental Avenue to Station 285+00

The risk of flood inundation will increase 1 to 1 ½ orders of magnitude because the Trinity River will be moved closer to the levee in this location. The result of moving the river closer to the flood protection in this area essentially doubles the average gradient, increasing the likelihood that the land-side toe of the levee could heave and the underlying sandy material would begin to erode during a flood event.

East Levee, Station 285+00 to Station 442+00

Risks posed by the currently existing channel alignment were estimated to be high during the BCRA. As a result, the City of Dallas placed a soil-bentonite cutoff wall at the river-side toe of the levee from Station 442+00 to 285+00. Had this cutoff wall not been in place, the risk would have increased with implementation of the BVP. However, the total risk with the BVP in place will be below the BCRA estimate due to the recent addition of the cutoff wall.

Cutoff Wall Remediation

The ability of the Dallas Floodway to reduce the risk of flooding the leveed area will be negatively impacted by implementation of the BVP in the area of Continental Avenue to Station 285+00. As a result, the RMC recommends that the existing cutoff wall be extended or an equivalent risk reduction alternative be implemented.

The current City of Dallas cutoff wall extends from Station 442+00 to 285+00 on the east alignment. An additional cutoff wall placed in the subject area would tie into this cutoff wall at Station 285+00 and extend downstream to the approximate location of Continental Bridge (approximately Station 170+00). This will extend the wall past the proposed meander at approximate station 200+00.

An impermeable clay blanket in the proposed river channel bottom was proposed as an alternative potential remediation measure during our discussions. However, a cutoff wall placed 10 to 30 ft in front of the river-side toe (the current City of Dallas cutoff wall configuration) was considered a better alternative because a channel blanket would likely require more material and likely scour protection (taking the form of rip rap in the channel bottom). In addition, a compacted clay river bottom may not be environmentally desirable. Use of a cutoff wall will provide a positive engineered cutoff that would extend from 20 to 40 ft in depth and not be difficult to construct. This also has the benefit of allowing excavation of the floodway floor without significantly impacting levee stability.

Trinity Parkway

Hydrologic analysis performed by the Fort Worth District indicates the placement of the Trinity Parkway embankment and floodwall will not significantly change flood heights, duration, or flood frequency within the Dallas Floodway when all of the Balanced Vision Plan Features are considered. Excavations made for the BVP lakes and removal of the ATSF Bridge generally offset any effects from the addition of the Parkway embankment.

The proposed roadway embankment is currently planned to be built to an elevation that is equal to approximately half of the levee height in order to keep it above the 100-year flood plane from approximately Station 285+00 to where the roadway passes under Continental Avenue. At this elevation, the overall width of the levee due to the addition of the roadway embankment will be significantly longer in cross-section (3 to 8 times as wide). In the event of a backwards erosion piping failure during a storm event (PFM 7, 8) with a maximum river level that's 50% of the levee height, the collapse of the crest into the erosion pipe would need to occur over the entire crest of the improved levee cross-section. This makes the levee much more robust against catastrophic failure for flood heights equal to 50% of the levee height. For flood heights that are greater than 50% of the levee height, the team thinks the addition of the Trinity Parkway embankment could delay or even prevent a full levee breach during a failure (from internal erosion or overtopping) should it occur from Station 285+00 to Continental Avenue, the location where the full roadway embankment will be placed. A partial breach would result in lower inundation depths and longer arrival times than a full breach, which would result in more time to evacuate and an overall reduction in estimated life loss consequences.

Lake Excavations

Performance of the clay aquatard beneath the proposed lakes is based on the current understanding of the geologic conditions of the Dallas Floodway. The Comprehensive Plan in its current form indicates that the proposed excavations for the lakes will not fully penetrate the clay layers overlying the basal sand layers except in the case of Oxbow Lake. There is an existing soil-bentonite cutoff wall at the riverside toe of the west levee There is, however, a potential for unknown and unexpected conditions to be encountered during construction of these lakes. If there is a clean sand layer encountered during excavation activities, a more robust remediation measure beyond the currently proposed 30-inch clay layer may need to be implemented (such as an additional cutoff wall at the river-side toe of the nearby levees.). Should any sands that are encountered contain a significant amount of fines, the proposed 30-inch clay cap may be acceptable.

Bridge Pier Penetrations

The BCRA results indicate that penetration of foundation elements into the levee section would not significantly increase risk. However, prudent design of structures that utilize such elements should include the use of granular blankets and concrete cover over the ground surface surrounding such peirs.

Utilities under the Proposed Lakes

There are old gravity-feed sewage lines that cross the area where there are lakes being proposed. Water and gas have good records. The location of fiber optic lines, however, may be available but there may be some question as to the actual number of lines in the ground. There are also jet fuel lines in the area of which location data is spread out over several agencies. These utility line crossings need to be handled individually during the design process of the lakes in order to avoid increasing risk.

Tunnel

There are ongoing concerns having to do with the recent tunnel collapse that were not addressed in this assessment of risk. Issues associated with the remediation of the collapse, future work on the tunnel should be addressed separately. Should any components of the Comprehensive Plan be put in place in the vicinity of the tunnel, the risk of failure in that area could be negatively impacted.

Closing

The risks estimated during the investigation of the Comprehensive Plan could change if the currently proposed river or lake alignments change. It should also be understood that river has the potential to migrate on its own after the BVP has been implemented. If this occurs, it could impact risk in a similar manner.

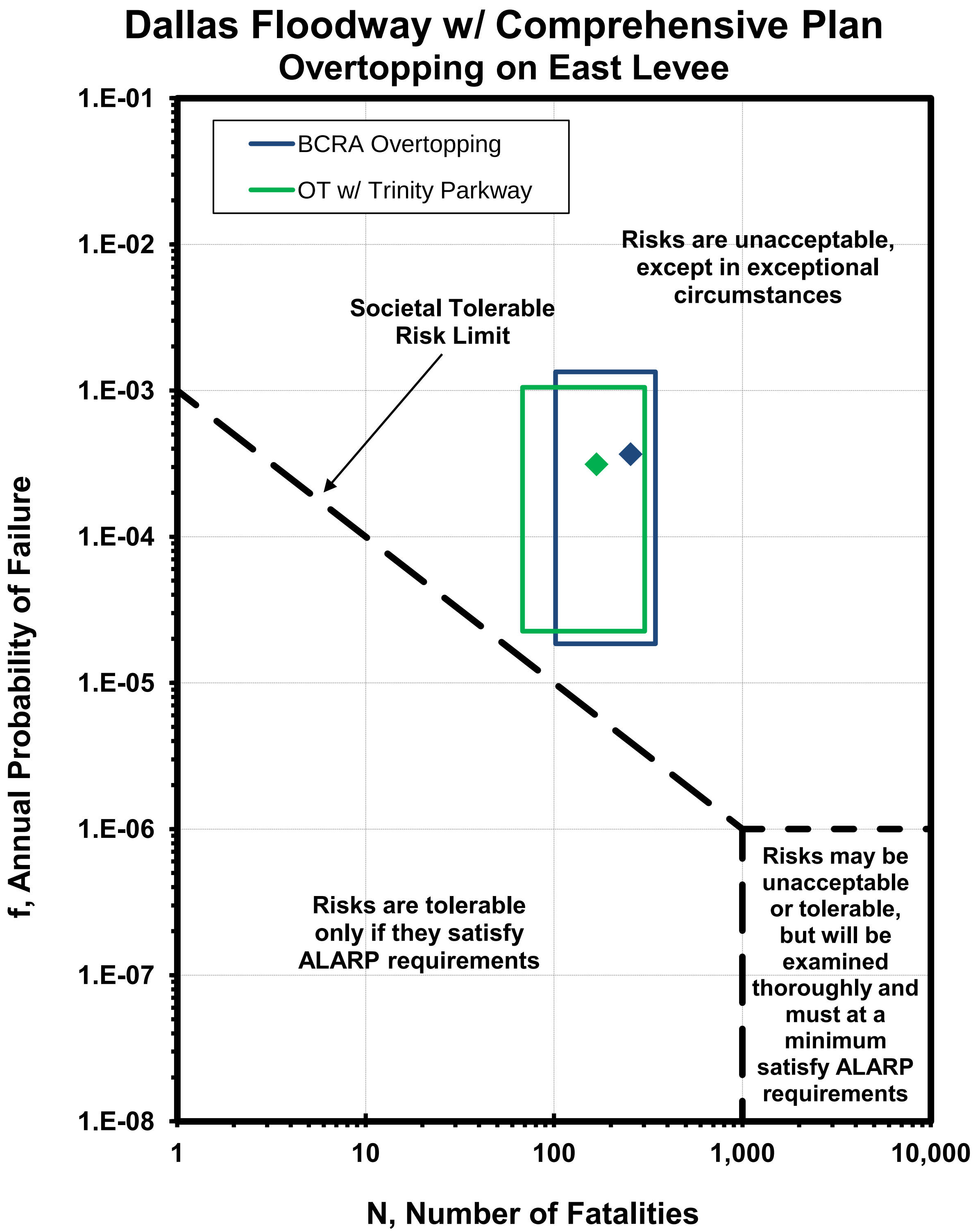

Figure 9. f-N plot for PFM 2, Overtopping, on the east levee alignment. Original BCRA and with-Trinity Parkway conditions shown.

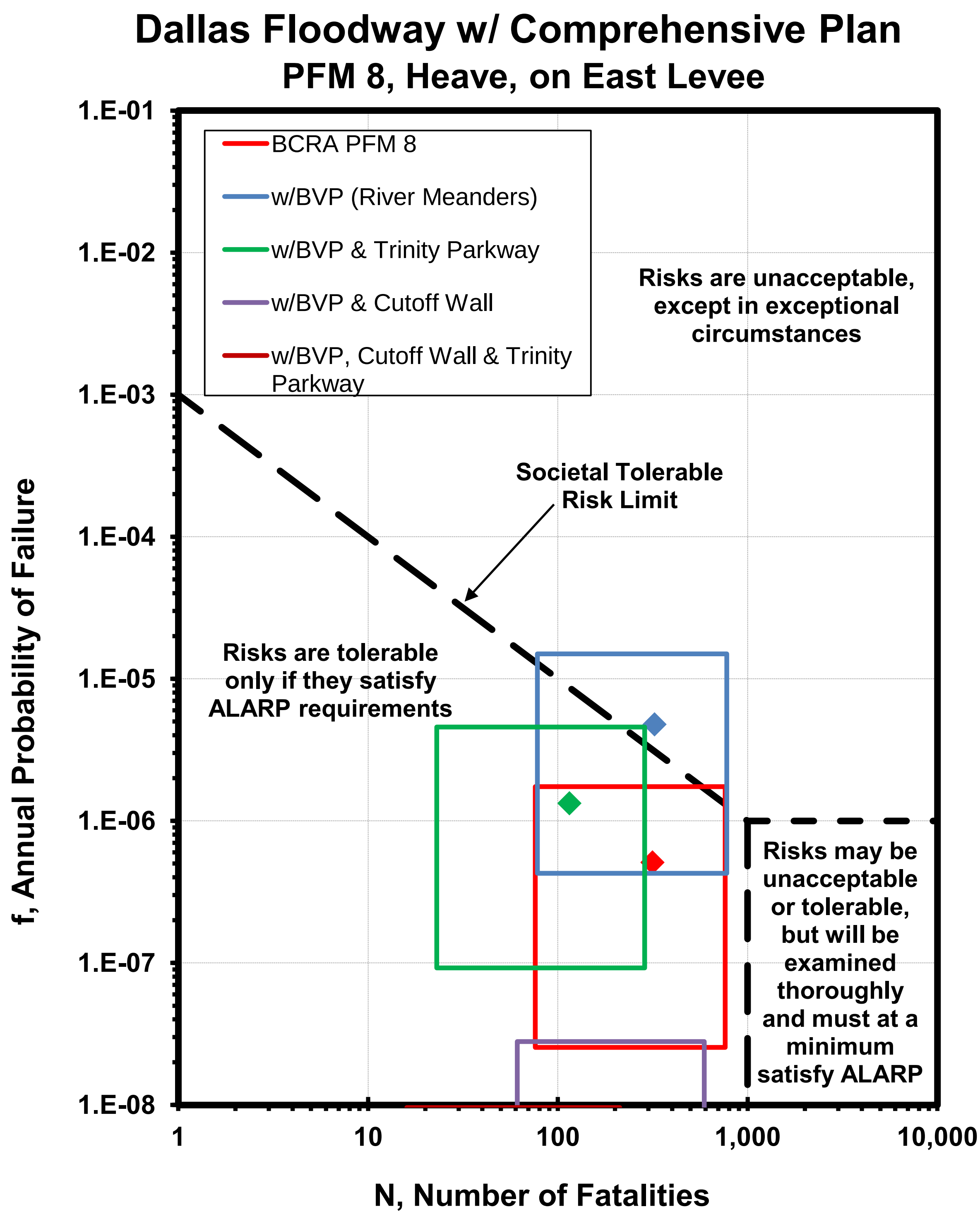

Figure 10. f-N plot for PFM 8, Heave, on the east levee alignment. Shown are the (1) original BCRA condition, (2) with BVP (river meanders) in place, (3) with BVP and Trinity Parkway, and (4) with BVP and cutoff wall. The risk of Heave occurring with the BVP, cutoff wall, and Trinity Parkway in place is below 1E-8 and is not shown on the graph.

Appendix A – References

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US Army Corps of Engineers, Risk Management Center. *Base Condition Risk Assessment, Trinity River Corridor, Dallas Floodway near Dallas, TX*, 6 April 2012.

US Army Corps of Engineers, Risk Management Center. *Risk Assessment of Proposed Remediation Methods (Alternatives Analysis), Trinity River Corridor, Dallas Floodway*, 2 November 2012.

Appendix B – Risk Estimating Team (RET) Participants

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