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Paper Title: Breach Analyses of High Hazard Dams in Williamson County

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Author’s signature (in ink) John R. King, Blaine Laechelin, Kim Potak
Date _____________________
TEXAS SECTION-ASCE
CIVIL ENGINEERING SESSIONS
PRESENTER INTRODUCTION

Paper title: BREAK ANALYSIS OF HIGH HAZARD DAMS IN WILLIAMSON COUNTY

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Presenter: CO-PRESENTER KIM PATAK

Is presenter: ☑ Principal author ☑ ASCE member ☒ Texas Section member ☒ Under age 35**

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Awards, Special Recognition, Professional Societies: ASCE & FEMA MEMBER

* Only the Power Point projector will be provided. The presenters need to provide their own laptop computers. If a Power Point presentation is planned, we highly recommend that a backup of slides, overhead transparencies, and/or handouts be available due to the possibility of projector malfunction.

** For consideration for Younger Member Award
Paper title: Breach Analyses of High Hazard Dams in Williamson County

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Awards, Special Recognition, Professional Societies:

* Only the Power Point projector will be provided. The presenters need to provide their own laptop computers. If a Power Point presentation is planned, we highly recommend that a backup of slides, overhead transparencies, and/or handouts be available due to the possibility of projector malfunction.

** For consideration for Younger Member Award
Breach Analyses of High Hazard Dams in Williamson County

John R. King¹, P.E., Kim Patak², P.E., Blaine Laechelin³, E.I.T.

Abstract

The Upper Brushy Creek WCID operates 23 dams in the Upper Brushy Creek Watershed in Williamson County, one of the fastest growing counties in Texas. A variance to the state’s dam safety criteria, approved by the TCEQ in 2003, requires the District to perform dam breach analyses and develop flood inundation mappings as part of the emergency action plans for its high hazard dams. The methodology for development of the breach parameters, performance of the breach and flood routing analyses, and mappings of the breach flood waves are discussed for four District high hazard dams.

Introduction

When the 23 dams in the Upper Brushy Creek Watershed in southern Williamson County were built by the U.S. Soil Conservation Service in the 1960s, they met all dam safety standards for earthen dams in sparsely populated areas. Now the same dams provide flood control for one of Central Texas’ fastest growing areas. The Upper Brushy Creek Water Control and Improvement District (District) is addressing the dilemma that these low-to-high hazard dams were built to standards designed for rural populations, but are now located in urban areas with rapid development. Figure 1 shows the District’s boundaries and the location of its 23 dams.

Current state dam safety criteria in the Texas Administrative Code (TAC), Title 30, Chapter 299 “DAMS AND RESERVOIRS”, requires all “High” hazard potential classified dams to safely pass 100% of the probable maximum flood (PMF). With respect to the District’s dams, Freese and Nichols, Inc. (FNI) has interpreted “safely pass” to mean that the dam does not fail. A 2000 study performed by FNI, on behalf of the original WCID, estimated the cost to modernize the dams, or to bring them into compliance with state dam safety regulations (i.e. safely passing 100% of the PMF) was cost prohibitive. However, if approved by the TCEQ as a variance, implementing the “cafeteria plan” approach would result in modernization costs that could be funded by a reasonable ad valorem property tax. This approach was recommended by the TCEQ (formerly TNRCC) Executive Director’s Task Force on Dam Safety in 1998. The TCEQ approved the variance in 2003 allowing the WCID to implement the cafeteria plan for all its 23 dams. That same year the WCID watershed voters approved a 2 cents ad valorem tax to fund the dam modernization program.

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The TCEQ’s approved variance did not specifically detail the design criteria for the cafeteria plan. FNI, through a 50/50 grant from the Texas Water Development Board (TWDB), conducted a detailed study of the requirements to implement the cafeteria plan specifically for the District’s dam modernization program. The results of the study were approved by the TWDB and the TCEQ, and the final report “Programming Study for the upper Brushy Creek WCID Dam Modernization Program, Williamson County, Texas” was issued in September of 2005.

One of the central tenets of the cafeteria plan is to allow a “high” hazard potential dam to safely pass between 50% to 100% of the PMF, rather than 100% of the PMF. However, if the hydraulic capacity of the dam is less than 100% of the PMF, then a range (i.e. menu) of non-structural options are to be implemented to mitigate the potential loss of life if the dam breaches, including:

- The District must install an early warning system (EWS) at the dam.
- The District must develop an emergency action plan (EAP) for the dam and the downstream floodplain.
- The District must establish an operation and maintenance program (OMP) for the dam.
- The District must develop an instrumentation and monitoring program (IMP) for the dam.
- The District must develop a formal dam safety inspection program for the dam.
- The District must implement a records keeping system.
- The District must develop a public information program.

A key component of the cafeteria plan for a dam that does not safely pass 100% of the PMF is development of the EWS in conjunction with the EAP, in order to provide effective evacuation of the public below the dam. An effective EWS and EAP provides a direct warning to evacuate the public at least 1½ hours before the breach wave reaches the area per U.S. Bureau of Reclamation research. For the EWS and EAP to be effective, one critical element is to provide, as accurately as possible, inundation mapping of the dam breach’s floodplain to identify the residential areas that are to be evacuated, the roads to be closed, and the critical infrastructure and service to be protected. The remainder of this paper documents the method used to develop the breach parameters to comply with the TCEQ’s variance and with the TWDB’s programming study criteria for implementation of the cafeteria plan, and the development of the breach mapping for the PMF and sunny-day events.

**Document Source for Breach Parameters**

The following two technical sources were used as the basis of deriving the dam breach parameters:


**General Embankment Breach Parameters**

The embankment overtopping breach condition has four primary input parameters:

- average width of the ultimate breach configuration
- top and bottom elevations of the ultimate breach configuration
- breach side slopes
- time of breach formation

The embankment piping breach condition has five primary input parameters:

- average breach width of the ultimate breach configuration
- top elevation of the ultimate breach configuration
- breach side slopes
- centerline elevation of initial piping location
- time of breach formation

**Specific Embankment Overtopping Breach Parameters**

**General.** Tony Wahl (Wahl) used a database of 108 documented dam failures to assess the statistical uncertainty of using four standard methods to develop the average breach width and time of breach formation parameters:

- Bureau of Reclamation (1998)
- MacDonald & Langridge-Monopolis (1984)
- Von Thun & Gillette (1990)
- Froehlich (1995a)

Table 1 in the ASCE Hydraulics Journal article summarizes the average breach width equations and the failure time equations of these methods. Table 1 also summarizes the 95% prediction interval around the hypothetical predicted value of 1.0 for these methods. Wahl found:

- The Bureau of Reclamation’s (1988) average breach width equation \( B_{\text{avg}} = 3h_w \) tends to significantly underestimate the observed breach widths and provides the smallest breach widths of the four methods he assessed. The Bureau of Reclamation’s failure time equation \( t_F = 0.011B_{\text{avg}} \) (metric), which is roughly 1.7 ft/min of height) also tends to significantly underestimate the breach formation time. Wahl states in his 1998 technical article:
“Reclamation (1988) provided guidance for selecting ultimate breach width and time of failure to be used in hazard classification studies using the SMPDBK model. The suggested values are not intended to yield accurate predictions of peak breach outflows, but rather are intended to produce conservative, upper bound values that will introduce a factor of safety into the hazard classification procedure.”

In the case study presented by Wahl in the 2004 ASCE Hydraulics Journal article, the Bureau of Reclamation chose not to use its own breach parameters. Because of the data presented by Wahl, FNI chose not to use the Bureau of Reclamations’ equations for average breach width. In lieu of the Bureau of Reclamations’ equations, FNI used research developed by Wahl to develop the breach parameters.

**Average Breach Width Parameter.** Wahl found that the Von Thun & Gillette (1990) and the Froehlich (1995a) methods provided the most reasonable predictors of average breach widths with the reservoir at its maximum pool level. The MacDonald & Langridge-Monopolis average breach width equation resulted in unreasonably large breach widths with the reservoir at the top-of-flood space scenario. For breach widths less than 30 meters (approx. 100 feet), Wahl found the Von Thun & Gillette (1990) method provided the best predictor of average breach width.

FNI chose to average the result of the Von Thun & Gillette (1990) equation with the Froehlich (1995a) equation to obtain the average breach width for overtopping breach conditions at the tallest section of the dam. The Von Thun & Gillette (1990) equation was used to calculate average breach widths at the abutments, because the average abutment breach widths were generally less than 100 feet.

The Von Thun & Gillette (1990) equation requires only the determination of the hydraulic head, \( h_a \), over the final breach bottom elevation (which equals the height of the breach, \( h_b \), under barely overtopping conditions) and \( C_b \), which is based upon total reservoir storage between the top and bottom of the breach section.

The Froehlich (1995a) equation requires calculation of the volume of water that is expected to be discharged through the breach section, per the following relationship:

\[
\text{Initial volume stored in reservoir above base of breach section} + \text{runoff volume} = \text{volume discharged through breach section} + \text{volume discharge through emergency spillway} + \text{volume discharged through the principal spillway}
\]

Up to the point of overtopping failure, all volume discharged from the reservoir is through the two spillways. After failure is initiated, the remaining volume of inflow will be discharged through the two spillways and the breach section. After an overtopping breach is initiated there will be three discharge regimes:
(1) With reservoir levels dropping from the top of dam (i.e. immediately upon initiation of an overtopping breach) down to the emergency spillway crest, both spillways and the breach will be discharging.

(2) With reservoir levels dropping from the emergency spillway crest down to the bottom of the final breach section, only the breach section will be discharging.

(3) With reservoir levels dropping from the principal spillway crest down to the bottom of the final breach section, only the breach section will be discharging.

FNI made initial assumptions to calculate the flow volumes through the breach section:

(1) It was assumed there was relatively little inflow by the time the reservoir level dropped from the principal spillway crest down to the base of the breach; therefore, the breach section discharged 100% of the flood storage volume within that reservoir zone. The volume discharged through the breach section was obtained from the reservoir’s stage-storage data; therefore, the configuration of the breach section was not as important.

(2) It was assumed there was relatively little inflow by the time the reservoir level dropped from the emergency spillway crest down to the principal spillway crest; therefore, the principal spillway and the breach section discharged 100% of the flood storage volume within that reservoir zone. The configuration of the breach section was roughly calculated using the Von Thun & Gillette equation for average breach width, and its discharge capacity was based upon the broad-crested weir equation \( Q = c h^{\frac{3}{2}} \). The discharge from the principal spillway was based upon the reservoir’s discharge rating curve.

(3) It was assumed that all the remaining inflow volume was discharged through the two spillways and the breach section as the reservoir dropped from the top of dam down to the crest of the emergency spillway. The rough configuration of the breach section, calculated for the previous flow regime was used to calculate the discharge capacity of the breach section at the top of dam. The discharge capacities of the two spillways were from the reservoir’s stage-discharge rating data.

As a cross-check of the assumptions discussed above, the volume of water discharged through the breach section \( V_w \) for Dam #13A was calculated using the hydraulic model’s final breach discharge hydrograph. The average breach width, \( \bar{b} \), was recalculated using Froehlich’s equation using the re-calculated \( V_w \). The re-calculated \( \bar{b} \) was within 96% of the \( \bar{b} \) calculated using \( V_w \) derived under the initial assumptions. All further calculations of \( \bar{b} \) per Froehlich’s equation used the stated assumptions to calculate \( V_w \).

**Breach Failure Time Parameter.** In Wahl’s 2004 ASCE Hydraulics Journal article he noted that all breach failure time equations tend to conservatively underestimate actual failure times.

FNI chose for the first two dams assessed (i.e. Dams #13A and #9) to use the breach failure time calculated by using the Von Thun & Gillette (1990) breach failure time equation based upon height, because its value fell close to the average of the failure time values determined by using the Bureau of Reclamation (1988), MacDonald & Langridge-Monopolis (1984), and the Von
Thun & Gillette (1990) equations. The Froehlich (1995a) equation gave a failure time greatly out of proportion to the other equations, so its value was not used in the assessment of failure time.

After the first 2 dams, FNI chose to calculate the breach failure times by averaging the values calculated by the Bureau of Reclamation (1988), MacDonald & Langridge-Monopolis (1984), and the Von Thun & Gillette (1990) equations.

The Von Thun & Gillette (1990) and the Bureau of Reclamation (1988) equations require only the determination of the hydraulic head, \( h_w \), over the final breach bottom elevation (which equals the height of the breach, \( h_b \), under barely overtopping conditions). The MacDonald & Langridge-Monopolis (1984) equation requires the additional calculation of the volume of water, \( V_w \), discharging through the breach section, which is the same value of \( V_w \) used in the Froehlich (1955a) equation to calculate average breach width \( \bar{b} \), as discussed above.

Side Slopes Parameter. In Wahl’s 2004 ASCE Hydraulics Journal article, he presents the Von Thun & Gillette (1990) suggestion of using breach side slope of 1:1, except for cases of dams with very thick zones of cohesive materials where side slopes of 0.5H:1V or 0.33H:1V might be appropriate.

FNI chose to use 0.33H:1V side slopes for all breach overtopping configurations; however, a flatter 1:1 side slope was used in the sensitivity analysis for the configuration having the largest average breach width and the longest breach failure time. Because of the short heights of the abutment breach sections (i.e. generally less than 10 feet in height), FNI assumed vertical side slopes for the abutment breaches.

Abutment Overtopping Breach Parameters: For abutment overtopping breach parameters, the base elevation of the final abutment breach configuration was set to equal the crest grade of the emergency spillway. The Von Thun & Gillette (1990) equation (i.e. \( \bar{b} = 2.5h_w + C_s \)) was used to calculate the average breach width, and the Von Thun & Gillette (1990) equation for a highly erodible embankment (i.e. \( t_F = 0.020h_w + 0.25 \)) was used to calculate the abutment breach failure time. Vertical side slopes were assumed.

Sunny Day Breach Parameters: According to the Tony Wahl (1998) technical article, Froehlich’s (1995a) equation for average breach width is the best predictor for the cases with observed breach widths less than 50 meters (i.e. 164 feet). Therefore, Froehlich’s (1995a) equations for both average breach width, \( \bar{b} \), and for breach failure time, \( t_F \), were used to calculate the “sunny-day” breach parameters, assuming a piping failure.

Froehlich’s breach failure time equation was used for the sunny-day piping condition because it gives a longer failure time than the other methods. According to Wahl’s 2004 ASCE Hydraulics Journal article, seepage-erosion failures can take a great deal of time to develop. As shown in Tables 1 through 4 of the article, the Froehlich equation results in the longest failure time for the overtopping failure conditions.
The other two sunny-day breach parameters are the centerline elevation of the beginning of piping and the breach side slope. The centerline of the beginning of piping was taken as the centerline elevations of discharge pipe of the principal spillway at the downstream toe of the dam, because piping was considered most likely to occur around and along the discharge pipe. In Wahl’s 2004 ASCE Hydraulic Journal article, he mentions that Froehlich (1995a) assumed side slopes of 0.9:1 (horizontal: vertical) for piping failures. FNI, therefore, used a side slope of 0.9H to 1.0V for the sunny-day piping breach failure.

**Sensitivity Analyses.** In Wahl’s 2004 ASCE Hydraulics Journal article he recommended a range of breach widths and a range of failure times be calculated using Table 1 “Prediction Interval Around Hypothetical Predicted Value of 1.0”. Tables 2 and 3 in his technical article illustrate the use of the prediction intervals in a case study.

FNI chose to use the prediction interval values in Table 1 of Wahl’s 2004 ASCE Hydraulics Journal article to calculate a range of breach parameters (within the 95% prediction interval) for sensitivity analysis of the overtopping breach at the tallest embankment section. This was done to determine the dam breach’s sensitivity to changes in breach failure time, while keeping the parameters within the 95% prediction interval. Once the average breach widths, $\bar{b}$, and the breach time of failures, $t_F$, were calculated by each method, Wahl’s 95% prediction intervals (given in Table 1 of his 2004 ASCE Hydraulics Journal article) were used to calculate a range of values for sensitivity analyses.

**Breach Parameters Results.** Results of the average breach widths, breach failure times, and sensitivity analyses used for analyzing the breach of Dams # 13A, # 9, # 1, and # 6 are summarized respectively in Tables 1 through 4 below.
Table 1 – Dam # 13A Breach Parameters

The following table summarizes the PMF breach parameters using all four methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>PMF Breach Parameters at Maximum Height of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\bar{b}$ (ft)</td>
</tr>
<tr>
<td>USBR</td>
<td>94’</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (height)</td>
<td>178’</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette ($\bar{b}$)</td>
<td>178’</td>
</tr>
<tr>
<td>Froehlich</td>
<td>184’</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis</td>
<td>N/A</td>
</tr>
</tbody>
</table>

For the PMF breach analysis of Dam # 13A, FNI selected the following average breach parameters for analyzing the breach at the maximum height of dam:

\[
\bar{b} = 180’ \\
\bar{t}_p = 0.441 \text{ hrs} \\
\text{Slide Slope} = 0.33H \text{ to } 1.0V \\
\text{Top Breach Elevation} = \text{el. 850.3} \\
\text{Breach Bottom Elevation} = \text{el. 819.0}
\]

The following configurations were used as a sensitivity analysis for analyzing the PMF breach at the maximum height of dam:

<table>
<thead>
<tr>
<th>Config.</th>
<th>$\bar{b}$ (ft)</th>
<th>$t_p$ (hrs)</th>
<th>Side Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>66’</td>
<td>0.200 hrs</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>B</td>
<td>42’</td>
<td>0.08 hrs</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>C</td>
<td>441’</td>
<td>9.2 hrs</td>
<td>1.0H to 1.0V</td>
</tr>
</tbody>
</table>

The following parameters were selected to analyze the abutment breach:

\[
\bar{b} = 80’ \\
\bar{t}_p = 0.300 \text{ hrs} \\
\text{Side Slope} = \text{vertical} \\
\text{Top Breach Elevation} = \text{el. 850.3} \\
\text{Breach Bottom Elevation} = \text{el. 841.8}
\]

For the sunny-day breach analysis of Dam #13A, FNI selected the following average breach parameters for analyzing the breach at normal pool height of dam:

\[
\text{Centerline elevation of pipe} = \text{el. 817.0} \\
\bar{b} = 33’ \\
\text{Side Slope} = 0.9H \text{ to } 1.0V \\
\text{Breach formation time} = 0.39 \text{ hrs}
\]

The following configurations were used as a sensitivity analysis for analyzing the sunny-day breach at normal pool height of dam:

<table>
<thead>
<tr>
<th>Config.</th>
<th>Centerline of Pipe</th>
<th>$\bar{b}$ (ft)</th>
<th>Side Slope</th>
<th>Breach formation time (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>el. 817.0</td>
<td>13’</td>
<td>0.9H to 1.0V</td>
<td>0.15 hrs</td>
</tr>
<tr>
<td>B</td>
<td>el. 817.0</td>
<td>80’</td>
<td>0.9H to 1.0V</td>
<td>2.85 hrs</td>
</tr>
</tbody>
</table>
Table 2 – Dam # 9 Breach Parameters

The following table summarizes the PMF breach parameters using all four methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>$\bar{b}$ (ft)</th>
<th>95% Prediction Interval (ft)</th>
<th>$t_F$ (hrs)</th>
<th>95% Prediction Interval (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>USBR</td>
<td>108’</td>
<td>48’ to 357’</td>
<td>0.362 hr</td>
<td>0.087 to 9.78 hr</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (height)</td>
<td>230’</td>
<td>85’ to 414’</td>
<td>0.469 hr</td>
<td>0.230 to 18.78 hr</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette ($\bar{b}$)</td>
<td>230’</td>
<td>85’ to 414’</td>
<td>0.668 hr</td>
<td>0.234 to 11.36 hr</td>
</tr>
<tr>
<td>Froehlich</td>
<td>N/A</td>
<td>N/A</td>
<td>0.86 hr</td>
<td>0.205 to 9.40 hr</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the PMF breach analysis of Dam # 9, FNI selected the following average breach parameters for analyzing the breach at the maximum height of dam:

- $\bar{b} = 230’$
- $t_F = 0.668$ hrs
- Slide Slope = 0.33H to 1.0V
- Top Breach Elevation = el. 792.0
- Breach Bottom Elevation = el. 756.0

The following configurations were used as a sensitivity analysis for analyzing the PMF breach at the maximum height of dam:

<table>
<thead>
<tr>
<th>Config.</th>
<th>$\bar{b}$ (ft)</th>
<th>$t_F$ (hrs)</th>
<th>Side Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Config. A</td>
<td>85’</td>
<td>0.23 hrs</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. B</td>
<td>48’</td>
<td>0.185 hrs</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. C</td>
<td>552’</td>
<td>11.24 hrs</td>
<td>1.0H to 1.0V</td>
</tr>
</tbody>
</table>

The following parameters were selected to analyze the abutment breach:

- $\bar{b} = 77’$
- $t_F = 0.29$ hrs
- Slide Slope = vertical
- Top Breach Elevation = el. 792.0
- Breach Bottom Elevation = el. 785.4

Dam #9 is dry during normal pool conditions because of Karst features along the creek channel. Therefore, sunny-day breach parameters were not developed for Dam #9.
Table 3 – Dam # 1 Breach Parameters

The following table summarizes the PMF breach parameters using all four methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>PMF Breach Parameters at Maximum Height of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\overline{b}$ (ft)</td>
</tr>
<tr>
<td>USBR</td>
<td>116'</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (height)</td>
<td>237’</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette ($\overline{b}$)</td>
<td>237’</td>
</tr>
<tr>
<td>Froehlich</td>
<td>241’</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis</td>
<td>N/A</td>
</tr>
</tbody>
</table>

For the PMF breach analysis of Dam # 1, FNI selected the following average breach parameters for analyzing the breach at the maximum height of dam:

$\overline{b}$ = 239’
$t_F$ = 0.52 hrs
Slide Slope = 0.33H to 1.0V
Top Breach Elevation = el. 1036.6
Breach Bottom Elevation = el. 998

The following configurations were used as a sensitivity analysis for analyzing the PMF breach at the maximum height of dam:

<table>
<thead>
<tr>
<th>Config. A</th>
<th>$\overline{b}$ (ft)</th>
<th>$t_F$ (hrs)</th>
<th>Side Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>52’</td>
<td>0.09</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. B</td>
<td>88’</td>
<td>0.23</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. C</td>
<td>600’</td>
<td>13.85</td>
<td>1.0H to 1.0V</td>
</tr>
</tbody>
</table>

The following parameters were selected to analyze the abutment breach:

$\overline{b}$ = 63’
$t_F$ = 0.29 hrs
Side Slope = vertical
Top Breach Elevation = el. 1036.6
Breach Bottom Elevation = el. 1025.9

For the sunny-day breach analysis of Dam # 1, FNI selected the following average breach parameters for analyzing the breach at normal pool height of dam:

Centerline elevation of pipe = el. 1000.5
$\overline{b}$ = 40’
Side Slope = 0.9H to 1.0V
Breach formation time = 0.63 hrs

The following configurations were used as a sensitivity analysis for analyzing the sunny-day breach at normal pool height of dam:

<table>
<thead>
<tr>
<th>Config. A</th>
<th>Centerline of Pipe</th>
<th>$\overline{b}$ (ft)</th>
<th>Side Slope</th>
<th>Breach formation time (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>el. 1000.5</td>
<td>16’</td>
<td>0.9H to 1.0V</td>
<td>0.24 hr</td>
</tr>
<tr>
<td>Config. B</td>
<td>el. 1000.5</td>
<td>96’</td>
<td>0.9H to 1.0V</td>
<td>4.6 hr</td>
</tr>
</tbody>
</table>
Table 4 – Dam # 6 Breach Parameters

The following table summarizes the PMF breach parameters using all four methods:

<table>
<thead>
<tr>
<th>Method</th>
<th>PMF Breach Parameters at Maximum Height of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PMF Breach Parameters at Maximum Height of Dam</td>
</tr>
<tr>
<td></td>
<td>( \bar{b} ) (ft)</td>
</tr>
<tr>
<td>USBR</td>
<td>167'</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (height)</td>
<td>279'</td>
</tr>
<tr>
<td>Von Thun &amp; Gillette (( \bar{b} ))</td>
<td>279'</td>
</tr>
<tr>
<td>Froehlich</td>
<td>274’</td>
</tr>
<tr>
<td>MacDonald &amp; Langridge-Monopolis</td>
<td>N/A</td>
</tr>
</tbody>
</table>

For the PMF breach analysis of Dam # 6, FNI selected the following average breach parameters for analyzing the breach at the maximum height of dam:

\[
\bar{b} = 277' \\
\begin{align*}
t_f &= 0.60 \text{ hrs} \\
\text{Slide Slope} &= 0.33\text{H to } 1.0\text{V} \\
\text{Top Breach Elevation} &= \text{el. } 919.8 \\
\text{Breach Bottom Elevation} &= \text{el. } 864.2
\end{align*}
\]

The following configurations were used as a sensitivity analysis for analyzing the PMF breach at the maximum height of dam:

<table>
<thead>
<tr>
<th></th>
<th>( \bar{b} ) (ft)</th>
<th>( t_f ) (hrs)</th>
<th>Side Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Config. A</td>
<td>75’</td>
<td>0.13 hr</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. B</td>
<td>109’</td>
<td>0.46 hr</td>
<td>0.33H to 1.0V</td>
</tr>
<tr>
<td>Config. C</td>
<td>656’</td>
<td>8.75 hr</td>
<td>1.0H to 1.0V</td>
</tr>
</tbody>
</table>

The following parameters were selected to analyze the abutment breach:

\[
\bar{b} = 92' \\
\begin{align*}
t_f &= 0.35 \text{ hrs} \\
\text{Side Slope} &= \text{vertical} \\
\text{Top Breach Elevation} &= \text{el. } 919.8 \\
\text{Breach Bottom Elevation} &= \text{el. } 907.2
\end{align*}
\]

For the sunny-day breach analysis of Dam # 6, FNI selected the following average breach parameters for analyzing the breach at normal pool height of dam:

\[
\bar{b} = 45' \\
\begin{align*}
\text{Centerline elevation of pipe} &= \text{el. } 867.1 \\
\text{Side Slope} &= 0.9\text{H to } 1.0\text{V} \\
\text{Breach formation time} &= 0.34 \text{ hrs}
\end{align*}
\]

The following configurations were used as a sensitivity analysis for analyzing the sunny-day breach at normal pool height of dam:

<table>
<thead>
<tr>
<th></th>
<th>Centerline of Pipe</th>
<th>( \bar{b} ) (ft)</th>
<th>Side Slope</th>
<th>Breach formation time (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Config. A</td>
<td>el. 867.1</td>
<td>18’</td>
<td>0.9H to 1.0V</td>
<td>0.13 hr</td>
</tr>
<tr>
<td>Config. B</td>
<td>el. 867.1</td>
<td>108’</td>
<td>0.9H to 1.0V</td>
<td>2.47 hr</td>
</tr>
</tbody>
</table>
Breach Analyses

The U.S. Army Corps of Engineers’ HEC-RAS unsteady flow model was used to perform the breach analyses and to route the flows downstream of the dam. The floodplain conveyance characteristics were defined by cross-sectional data and Manning’s “N” data. Sections were cut using the Capital Area Planning Council’s (CAPCO) 2002 2-foot topography and aerial photography. The GEO-RAS program was used to cut the sections. The Manning’s “N” values were developed based upon field observations and then increased by 25% to account for turbulence expected under PMF and breach flow conditions, as recommended in the TCEQ’s 2006 dam safety design guidelines. Road crossing and bridge data was obtained from available construction plans and supplemented with field measurements.

The upstream boundary conditions were defined by the pre-breach discharge hydrographs developed by the HEC-1 program. The downstream boundary conditions were defined by tailwater conditions at the confluence of each dam’s floodplain with the main channel of Brushy Creek. Two downstream boundary conditions were analyzed to determine the most conservative breach floodplain. These conditions include: (1) normal depth flows at the confluence under the assumption that the water surface levels within the main Brushy Creek channel did not create a backwater effect, and (2) the basin-wide PMF flood flows on the main Brushy Creek channel would cause a backwater effect. The second downstream boundary condition created the highest water surface levels at the lower reaches of each dam’s floodplain and therefore was considered adequately conservative for flood evacuation mapping purposes.

The basin-wide PMF hydrologic model was also used to determine local runoff discharging laterally into the floodplains downstream of the dams.

The unsteady flow breach routine of the HEC-RAS program was used to assess the peak discharge rates of the “average” breach parameters and to perform breach sensitivity analyses to identify any necessary changes to the breach parameters before routing the discharge hydrographs downstream of the dams. Tables 5, 6 and 7 summarize the sensitivity analyses of the breach parameters performed for Dams 13A, 1, and 6, respectively.

Table 5

<table>
<thead>
<tr>
<th>Breach Parameter*</th>
<th>Peak Discharge Rate (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMF Breach at Maximum Height of Dam</td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>48,875</td>
</tr>
<tr>
<td>Configuration A</td>
<td>24,380</td>
</tr>
<tr>
<td>Configuration B</td>
<td>19,280</td>
</tr>
<tr>
<td>Configuration C</td>
<td>9,633</td>
</tr>
<tr>
<td>Sunny-Day Breach</td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>680</td>
</tr>
<tr>
<td>Configuration A</td>
<td>593</td>
</tr>
<tr>
<td>Configuration B</td>
<td>1,534</td>
</tr>
<tr>
<td>* Note: See Table 1 for configuration of Dam #13A breach parameters.</td>
<td></td>
</tr>
</tbody>
</table>
### Table 6
**Dam #1 Breach Parameters Sensitivity Analyses**

<table>
<thead>
<tr>
<th>Breach Parameter*</th>
<th>Peak Discharge Rate (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PMF Breach at Maximum Height of Dam</strong></td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>85,899</td>
</tr>
<tr>
<td>Configuration A</td>
<td>22,247</td>
</tr>
<tr>
<td>Configuration B</td>
<td>37,352</td>
</tr>
<tr>
<td>Configuration C</td>
<td>10,578</td>
</tr>
<tr>
<td><strong>Sunny-Day Breach</strong></td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>2,317</td>
</tr>
<tr>
<td>Configuration A</td>
<td>1,299</td>
</tr>
<tr>
<td>Configuration B</td>
<td>2,167</td>
</tr>
</tbody>
</table>

* Note: See Table 3 for configuration of Dam #1 breach parameters.

### Table 7
**Dam #6 Breach Parameters Sensitivity Analyses**

<table>
<thead>
<tr>
<th>Breach Parameter*</th>
<th>Peak Discharge Rate (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PMF Breach at Maximum Height of Dam</strong></td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>142,524</td>
</tr>
<tr>
<td>Configuration A</td>
<td>52,259</td>
</tr>
<tr>
<td>Configuration B</td>
<td>71,214</td>
</tr>
<tr>
<td>Configuration C</td>
<td>17,784</td>
</tr>
<tr>
<td><strong>Sunny-Day Breach</strong></td>
<td></td>
</tr>
<tr>
<td>Average Breach Parameter</td>
<td>5,352</td>
</tr>
<tr>
<td>Configuration A</td>
<td>2,384</td>
</tr>
<tr>
<td>Configuration B</td>
<td>3,723</td>
</tr>
</tbody>
</table>

* Note: See Table 4 for configuration of Dam #6 breach parameters.

The breach parameter sensitivity analyses resulted in the following findings:

- The “average” breach parameters for the PMF breach had the highest peak discharge rate when compared to the other configurations. This configuration also produced the highest downstream water surface depths. Therefore, this configuration was chosen to map the PMF breach floodplain for flood evacuation purposes.
- The average or the configuration “B” sunny-day breach parameters had the highest peak discharge rate when compared to the other configurations. The configuration that produced the highest downstream water surface depths was chosen to map the sunny-day breach floodplain for flood evacuation purposes.

The pre-breach and the breach discharge hydrographs were routed below each dam to the location where the breach water surface level was within one foot of the pre-breach water surface level. The derivation of the breach discharge hydrograph and the routings of the pre-breach and
breach discharge hydrographs were performed using the unsteady flow routine of the HEC-RAS program. Table 8 summarizes the channel distances below two of the dams within which the breach flood levels have attenuated to within 1 foot of the pre-breach flood levels. Table 8 also summarizes the time it takes the leading edge of the breach wave to reach the end of the attenuation zone. These times indicate that the public will have very little time to react within the breach inundation zones once the dams breach.

<table>
<thead>
<tr>
<th>Dam</th>
<th>Attenuation Length (ft)</th>
<th>Time for Breach Wave to Travel Attenuation Length (min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13A</td>
<td>9,825</td>
<td>45</td>
</tr>
<tr>
<td>1</td>
<td>57,267</td>
<td>120</td>
</tr>
</tbody>
</table>

The zones of inundation produced by the sunny-day and PMF breaches were mapped using the Geo-RAS software. The breach inundation mappings confirmed the “high” hazard potential classifications of the dams. The breach inundation mappings will be included within the emergency action plan (EAP) for each dam to aid local emergency response officials in evacuating the public from at risk areas, in defining safe evacuation routes, and in identifying sections of roadways that are to be closed. Figure 2 provides the breach inundation map for Dam # 13A.

**Summary Remarks**

Breach analyses were performed on four high hazard potential dams of the Upper Brushy Creek WCID in southern Williamson County. The breach parameters used to perform the PMF breach analyses and the sunny-day breach analyses were based upon the recommendations of Tony Wahl with the U.S. Bureau of Reclamation’s Dam Safety Office. Up to five methodologies were used to derive breach parameters, and sensitivity analyses were performed to determine the most critical combinations of average breach widths, breach formation times, and breach side slopes.

The U.S. Army Corps of Engineers’ HEC-RAS unsteady flow model was used to model the sunny-day and PMF breaches and to route the pre-breach and breach discharge hydrographs along the floodplains downstream of the dams to the locations where the breach flow depths attenuated to within 1 foot of the pre-breach flow depths. The assumed downstream boundary condition of PMF backwater from the main Brushy Creek channel created higher breach flood levels than assuming a free flow into the main creek channel. The breach inundation levels impacted by the main channel PMF backwater were determined to be the most appropriate conservative conditions for emergency flood/breach evacuation purposes.

The breach waves were found to travel at velocities between 3.7 to 8.0 feet per second, thereby not providing sufficient reaction or evacuation time within the breach inundation zones after dam breach occurs.
Figure 2
References


